Eleventh International Symposium on
Water Management and Hydraulic Engineering

International Symposium on
Water Management and Hydraulic Engineering

Editors:
Cvetanka Popovska
Milorad Jovanovski

Cover Design:
Igor Popovski

Published by:
Faculty of Civil Engineering, Ss. Cyril and Methodius University, Skopje

Printed by:
JOFI Sken

Web Site:
http://wmhe.gf.ukim.edu.mk

CIP-Каталогизација во публикација
Национална и универзитетска библиотека "Св. Климент Охридски", Скопје

556:551.58(062)
626/628(062)
502.51(062)

SYMPOSIUM on Water Management and Hydraulic Engineering (11th ; 2009 ; Ohrid)
Proceedings / Eleventh international symposium on water management and hydraulic engineering ; edited by Cvetanka Popovska and Milorad Jovanovski. – Skopje : Faculty of civil engineering , 2009. – 2 св. (592; 593-940) стр. ; 24 см : илустр.

Conclusion кон трудовите. – Библиографија кон трудовите

ISBN 978-9989-2469-6-8 (св. 1)
ISBN 978-9989-2469-7-5 (св. 2)
ISBN 978-9989-2469-8-2 (пов. св.)

1. Popovska, Cvetanka [уредник] 2. Јовановски, Милорад [уредник]
а) Хидраулика - Собири б) Водни еко-системи - Собири в) Водостопанство-Собири г) Животна средина - Собири
COBISS.MK - ID 79029514

NOTE: The volumes of the Proceedings contains original authors papers reviewed and accepted by the Advisory Scientific Committee

Eleventh International Symposium on

Water Management and Hydraulic Engineering

PROCEEDINGS

VOLUME I

Edited by

Cvetanka Popovska

and

Milorad Jovanovski

University Ss. Cyril and Methodius, Faculty of Civil Engineering, Skopje, Macedonia

Published by

University Ss. Cyril and Methodius, Faculty of Civil Engineering, Skopje, Macedonia

Skopje, 2009

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Preface

Water Management and Hydraulic Engineering (WMHE) conferences were established mainly to preserve the quality of the water resources and serving the water requirements of the society. The meetings of scientists and professionals in water sector have started as an event and after years of successful outcomes have become a process. The main goal of the conferences is to achieve on long term perspective the integrated approach in water management. Transboundary shared waters in Europe are a challenge for application of the well known concepts of Integrated Water Resources Management in a context of regional climate changes and anthropogenic pressures on the environment. The transboundary water systems are part of a multi-disciplinary, multi-objective, multi-purpose set of critical challenges that exemplify engineering, ecological, geopolitical, organizational and methodological dimensions.

The key objectives of WMHE are to improve the scientific knowledge and networking between scientists, to favor multi-disciplinary approach and to disseminate reliable information on new technologies, practices and strategies to the end-users in order to improve the education and to preserve the water resources and the biodiversity of the aquatic ecosystems.

This conference tries to contribute to the key goals by inviting the experts from Central Europe, from older and new EU member states as well as from South-East European candidate countries. Exchange of experiences in the field of water management and hydraulic engineering has contributed a lot to the European water sector.

The Water Framework Directive (WFD) is a far reaching document with a major institutional component for water management, comparability in procedures and measurements, and with special provisions for coordination and regulation in shared river basins. So, the new challenge of WMHE now and in future should be introducing innovative engineering solutions and arrangements pertaining to transboundary water resources. Essentially, in a volatile and increasingly vulnerable environment we must develop coherent "vigilance" strategies in the form of contingency planning and management. The resultant treaties, exchanges, and compacts would combine lessons of the past (historical continuity); understanding of present conditions (modeling and critical variables); and a vision of things to come, in term of both probable and preferable futures.

This conference is the eleventh in the series of similar conferences that were started in 1976. The first one was a bilateral activity between the faculties of Gdańsk University (Poland) and Zagreb University (Croatia). Since 1998, the Slovak University of

Technology, the University Ss. Cyril and Methodius from Skopje (Macedonia) and the BOKU University of Natural Resources and Applied Life Sciences from Vienna (Austria) have joined this two-annual conference series.

Now, it is for the first time, that the faculty of Civil Engineering in Skopje, which is integrated into the state University Ss. Cyril and Methodius, is organizing this conference from 1-5 September 2009 in Ohrid, in the South-Western part of Macedonia. For the first time this conference was announced widely on Internet by creation its web page: http://wmhe.gf.ukim.edu.mk. Immediately after it was established this URL was installed on world wide portals such as:

UNESCO Water Portal: http://www.semide.net/thematicdirs/events/wmhe-2009-eleventh-international-symposium-water-1

IAHS Info Portal: http://iahs.info/meeetings

BALWOIS Network Portal: http://balwois.com/cms/index.php

This Internet activity provoked a great interest of the scientists and professionals all around the world that can be shown with the following statistics: 112 received abstracts from 19 countries and 94 submitted full papers, out of which about 70 will be presented orally. The proceedings publish selected papers in two volumes.

During the conference the presentation of the papers will be organized in five thematic sessions which refer to:

1. Hydraulic Engineering and Environmental Impacts
2. Sanitary Engineering and Sustainable Water Use
3. Water Resources and Environmental Management
4. Water Bodies Protection and Ecohydrology
5. River Basin Restoration Strategies and Practices

To manage the tasks related to this conference, a Scientific Advisory Committee was set up:

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The organizers of this conference would express their sincere thanks to all of those who have supported and contributed to this meeting.

The members of the Organizing Committee made a great effort on technical preparation of the conference materials which is certainly not less important than the other issues, and the editors would like to express their warm gratitude to the people listed bellow:

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Roman Wichovski, Gdańsk University of Technology, Poland
Adam Bolt, Gdańsk University of Technology, Poland

This conference has the privilege to be organized under the auspices of the International Association of Hydrological Sciences (IAHS). Without strong support of *Pierre Hubert*, the Secretary General of IAHS this would not be achieved. The organizers gratefully acknowledged this support.

Special thanks should go to *Velimir Stojkovski*, the Rector of the University Ss. Cyril and Methodius in Skopje, who has demonstrated understanding and strong support, and helped a lot in achieving relaxing atmosphere during the final preparation of the conference which is usually the most pressurized work.

Editors

Cvetanka Popovska and *Milorad Jovanovski*

Keynote Lectures

International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Keynote lecture

Theoretical and Practical Solving of Hydroengineering Problems

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Abstract. The household speech abounds with many simplified terms and condensed expressions. Usually they do not cause special misunderstandings in every day human contacts, but related to the strict technical problems may bring some negative consequences. The paper contains a discussion of a commonly used opposition "theoretical-practical". The analysis of different aspects of these forms led to the conclusion that from the methodological point of view there exists an opposition "theoretical–empirical", whereas the term "practical" should be understood as a rational and important completion of both these attitudes.

Keywords: Technical problems, solving approach, theoretical, practical and empirical methods; gnoseological, volitional and formal aspects

1 General Remarks

The key word in the title of this paper is "**and**", what should be underlined, as the role of this conjunction is not absolutely clear. Moreover, this word is quite often being replaced by a completely different term, namely "**or**".

The thing is that two words: **theoretical** and **practical** are very often considered as an opposition. The aim of this paper is a discussion of this question and justification of a thesis, that "there is nothing more practical, than a good theory".

2 Terminological Aspects

In a common approach, the term "theoretical" denotes each human activity, or the result of this activity, which is combined with some reasoning. In a context there sounds a suspicion, that such reasoning is not very precise or at least charged by a very high number of simplifying assumptions. From this statement there is only one step to the conviction, that each theory is sophisticated and subtracted from the real life.

Quite different is the current understanding of the activity and/or results, which are described as "practical". They are considered as based on human experience, otherwise–real, positive, connected with life, leading to the solutions, which are of avail.

One has to admit, that there is "a grain of truth" in these statements, but this grain was milled by "the mill of horse sense". It is very surprising, how often so called "common sense" leads to mistaken conclusions. As a classical example let us consider a gravitational downfall of two bodies–geometrically similar, but having apparently different masses. For the question: "Which body will fall down first?", "commonly thinking people" usually answer, that the heavier one, whereas in fact both these bodies will end their motion in the same moment of time, as both are subject to the same gravity acceleration.

Anyhow, both analyzed terms are treated as opposition in every-day speech. As a distinct confirmation of this fact, one can quote some definitions from a linguistic vocabulary [2], where one can read, that the term "theoretical" describes something based on a **concept**, on a **cognition** and **understanding**, but not on a **practice**. The term "practical" in turns means something based on an **experience**, having an **applied** character and also **suitable**, **useful**, **convenient** (what may suggest, that the theory is **useless**, **unprofitable** and **inconvenient**).

In order to decide the problem, or at least to refer to this alarming situation, one has to analyze the structure of the theory of cognition.

3 Gnoseological Aspects

This is worth remembering, that the courses of gathering and presentation of our knowledge have the opposite directions. According to the lapidary Aristotelian expression (IV c. BC), the **cognition** runs from details, towards generalization, whereas **teaching** should start from the general approach towards full particulars. This means, that nobody should start his research from "the general theory of everything". And similarly – presentation of details at the very beginning of the lecture it is not a good start point to the didactic success.

There are no doubts that the main role in cognition is played by the **observation**. Certainly, the rational argumentation is also a very important factor. The admirable concepts of the founders of Ionian natural philosophy [3] can serve as the best evidence of this fact. However, also these eminent thinkers were not completely isolated from the surrounding world.

The effective advantage, yielded by this source, is directly defined by its range. One can distinguish two main situations (Fig. 1). As the first case we should consider comprehensive and widely developed sets of empirical information, acquired and gathered by numerous investigators, differentiated in space and time. Such a system can be called "a *summa* of experience". But there is also a second possibility, according to which one can gather only a limited “block of information”, gained locally, only for one individual problem.

The general “*summa* of experience” can be generalized, what leads to **scientific theories**, usually expressed by **laws of nature**. Each law must be from definition **univocal**. This condition can be easily fulfilled, when the considered phenomena are described by means of **mathematical models**. In the classical and most convenient way, such models are created by **the equations of mathematical physics**.

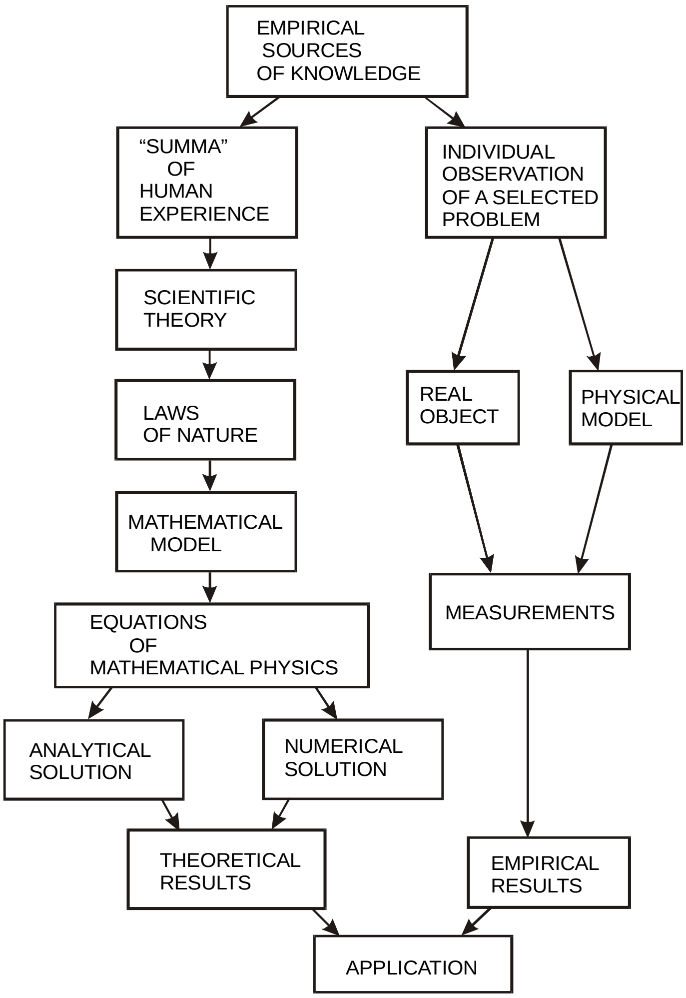

Fig. 1. Structure of cognition process

The alternative forms – **table notation** and **graphical methods** – are less accurate and less convenient, so less popular in consequence, although the former possibility forms the core of **numerical methods**, very useful when the considered equations can not be solved in **analytical** way.

Apparently different character has the advantage, brought by the "local observation" (Fig. 1). There are two possibilities in this way – we can make use of **natural objects**, one or more, already existing, or we can use **physical models**, constructing special objects (in natural dimensions or diminished in some scale), which should fulfill requirements of **theory of similarity** [4, 6].

Final effects, obtained by means of these methods, have different qualities. Theoretical results are general and reliable, whereas these empirical – are confined to the scope of measurements.

So, from the methodological point of view, we can enumerate two attitudes:

- theoretical (when "*summa* of experience" yields a theory, which provides facilities for solving the problem);

- empirical (when "local observation" institute the main tool for the unit problem solving).

As it is seen, these two approaches complement one another and quite often - create a new quality. As a clear example, let us analyze the derivation of Bernoulli's theorem. Taking as a start point the equation of momentum conservation [4]:

$$\rho \frac{D\mathbf{u}}{Dt} = \rho\mathbf{f} + div\left[P\right] \tag{1}$$

After some well known rearranging we can obtain the following expression:

$$\left(\frac{V_{L1}^2}{2g} + \frac{p_1}{\rho} + gz_1\right) = \left(\frac{V_{L2}^2}{2g} + \frac{p_2}{\rho} + gz_2\right) + \int_1^2 \frac{\tau_S}{\rho R_H} dL \tag{2}$$

In order to apply this relation, the shear stress along the stream side surface must be determined. Theoretically we can do it only for some very simple situations, for instance – in the case of Hagen-Poiseuille motion (laminar flow in a horizontal pipe), when:

$$\tau_S = \frac{8\mu v}{D} \tag{3}$$

what leads to a special form of Eq. (2):

$$\Delta p = \frac{32\mu L v}{D^2} \tag{4}$$

Alas, for the overriding turbulent flow it did not succeed in theoretical calculation of this value, so the empirical approach was necessary. Knowing the general relation:

$$f\left(\Delta p, v, L, D, \rho, \gamma, k_S\right) = 0 \tag{5}$$

and making use of the Buckingham's theorem [6], the following relation has been derived:

$$\Delta p = \lambda \frac{L}{D} \frac{\rho v^2}{Z} \tag{6}$$

well known as the **Darcy-Weisbach formula**. Empirical coefficient of hydraulic loss is generally described by the **Nikuradse harp**, or by some equivalent methods.

After these considerations, everybody should admit, that from the methodological point of view there is no place for anything, what could be called as the **distinct practical approach**. Both the theoretical **and** practical descriptions **must have practical features**, in each already presented meanings of this term, because both these methods must be:

- **useful** (even if the method is very simple, it can serve as a scholastic example);
- **supported by the personal experience** of the specialist (what evidently increases advantages, given by this method);
- **convenient** (although the meaning of this term depends on the problem complexity);
- **profitable** (as nobody would take the subject without any final profits).

5 Volitional Aspects

Cognition of reality is strongly influenced by psychological factors, especially – the will of getting knowledge. Individual motivation (curiosity, ambition, welfare, money, personal career…) plays here a very important role.

It is necessary to underline that the real and valuable information must be objective and honest. Alas, taking into account the human weakness, some control and supervision systems are necessary, as the fear for consequences is a very strong stimulator of human behavior. Each investigation should be conducted as deeply, as possible, what is especially important in a modern world, over dominated by "the public relations".

As an example, let us discuss the explanation of the more and more popular term – SPA. It apparently denotes some specific combination of a beauty parlor, a fitness-club and a balneology center.

In the original version, the term "Spa" is a name of an ancient town in Belgium, known for its therapeutic merits since antiquity. Mineral springs in Spa have been discovered in XIV century, and the place was especially popular in XVIII and XIX century. Much so, that its name became **the synonym of the health resort**.

It is worth mentioning, that in Galician part of the Austro-Hungarian Empire the similar role was played by the Italian town – Merano. One could propose more similar examples of situation, when the name of a place or of a producer becomes a synonym of something special (e.g. the terms ELECTROLUX and HOOVER are synonyms of the term DUSTER in Poland and England respectively).

Going back to our topic we can state, that this new sense of the term SPA could be accepted as an intuitive generalization of its original meaning. But some objection arises, when somebody explains this word as an abbreviation: SPA = *Salus Per Aqua*. This explanation is skin-deep and does not show the full taste of the considered word.

6 Formal Aspects

One can draw some useful conclusion, analyzing **the formal structure** of mathematical notation of physical laws. From the nature of things, we can divide different processes into three groups:

- "algebraic" (which are described by **dependent variables**, determined in **each point** of the considered system – e.g. **melting**);

- "differential" (which are described by **local differences** of these variables, determined in **the neighborhood** of each point – e.g. **heat conduction**);

- "integral" (which are described by the **global value** of a proper physical property, determined for **the whole system** – e.g. **economical decision**).

According to the essence of this classification, the particular process can be described by means of algebraic, differential or integral equations respectively.

For simple cases the governing equations can be solved analytically. On the other hand, when these equations are too difficult – numerical methods should be applied.

In this general presentation, there is no place for the detailed lecture on this possibility. However we should pay attention to one important element. Discretization, basic step in a numerical approach, has in this case as if "a secondary character". The idea is that deriving general equations, we express them for the optionally defined system and then we pass into the differential (or in some cases – integral) notation. Applying a chosen numerical method, we have to withdraw in a specific manner, replacing a continuous system by the set of discrete points or elements. Let us show it for a classical Laplace equation:

$$\Delta F = \frac{\partial^2 F}{\partial x^2} + \frac{\partial^2 F}{\partial y^2} = 0 \tag{7}$$

Replacing the second derivatives by the proper differential quotients:

$$\frac{\partial^2 \mathrm{F}}{\partial \mathrm{x}^2} = \frac{\mathrm{F}_{\mathrm{i+1,j}} - 2\mathrm{F}_{\mathrm{ij}} + \mathrm{F}_{\mathrm{i-1,j}}}{\mathrm{h}_2} \tag{8}$$

$$\frac{\partial^2 F}{\partial y^2} = \frac{F_{i,j+1} - 2F_{ij} + F_{i,j-1}}{h^2} \tag{9}$$

we get the following algebraic expression:

$$F_{ij} = 0{,}25\left(F_{i+1,j} + F_{i-1,j} + F_{i,j+1} + F_{i,j-1}\right) \tag{10}$$

which is the working form of this method.

The stage of discretization brings about a specific temptation to relate the laws of nature immediately to these numerical objects (points or elements). This idea is a cornerstone of so=called "computational mechanics". Such attitude can be considered as a kind of "by-pass" in a full system of mathematical description of natural and technical processes (Fig. 2). Surely, it has some important advantages (mainly it simplifies the lecture), but also generates some important problems – making use of this method we make shallow the knowledge about physical aspects of the considered problem. In this proceeding one can read a kind of escape from the effort, that is necessary for the full understanding of physical theories.

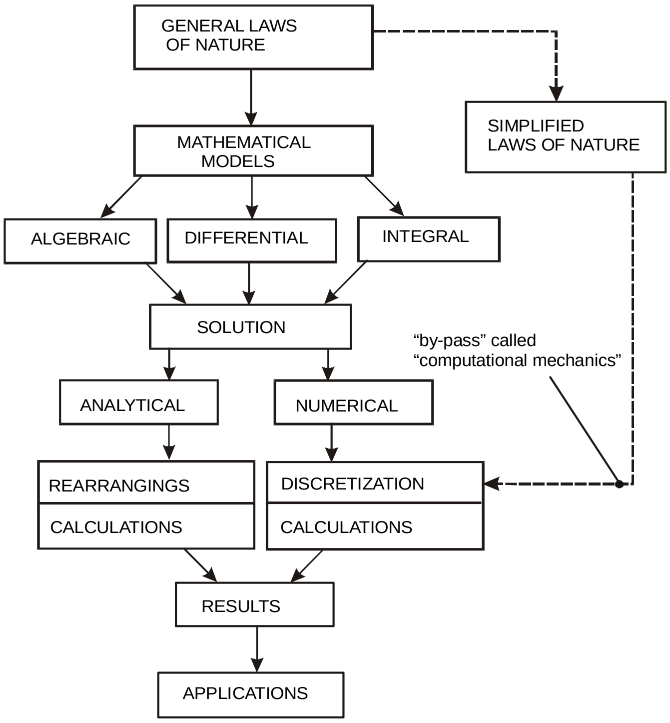

Fig. 2. Categories of mathematical models

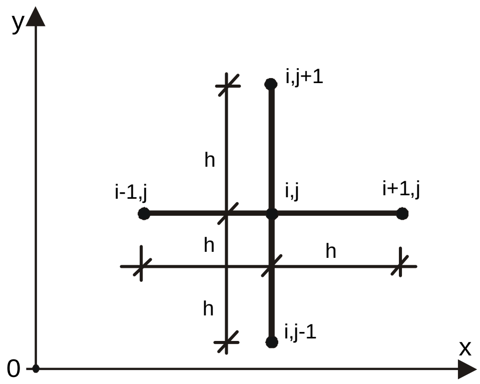

Fig.3. Numerical grind for finite-differences method

7 Conclusions

Reassuming presented considerations one can state, that in scientific and technical activity the word “practical” should not be understood in its household sense, but only in its literal meaning.. We can say that the **method** or **approach** is:

THEORETICAL AND PRACTICAL

or:

EMPIRICAL AND PRACTICAL

The opposition:

THEORETICAL **OR** PRACTICAL

can be rationally used only when:

THEORETICAL means oriented towards SCIENCE
PRACTICAL means oriented towards APPLICATIONS

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009

Keynote lecture

The Role of Cross-border Co-operation Programme for Water Management in Lowland Regions

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Abstract. The first period of entering the Slovak Republic into EU (2004-2007) has been connected with applying and financing of several projects of cross-border co-operation with neighbourhood countries. Some of them have been financed in the field of environment, sustainable development as well as water management in river basins which have cross-border character. This inter-regional initiative was splitted according to countries with common border, i.e. co-operation between Austria and Slovak Republic (AT-SK), between Czech and Slovak Republic (CZ-SK), between Slovak Republic and Poland (SK-PL) and among Hungary, Slovak Republic and Ukraine (HU-SK-UA). The presented contribution is dealing with environmental, water management aspects of some projects which have been solved at the Faculty of Civil Engineering, Slovak University of Technology in Bratislava in frame of the mentioned inter-regional initiative and is continuation of the project presentation at the WMHE 2007 in Šibenik [4].

Keywords: Inter-regional programme, cross-border water management, lowland region, river restoration and revitalisation, technical measures, water management scenarios

1 Introduction

The Department of Hydraulic Engineering has been involved in four inter-regional projects. Three of them where connected with the region close to Hungary and Ukraina – the region among Tisa, Latorica and Bodrog rivers - called in Slovakia Medzibodrožie or in Hungary Bodrogköz region (Fig. 1). As it was mentioned already on WMHE 2007 in Šibenik [4] the main reason for the project proposal was that in this region five flowing rivers were "alive" in this region fifty years ago. Except of the above mentioned Latorica, Bodrog and Tisa rivers there were another two rivers – Tice and Krčava - which were due to water management measures in the years 1946-64 more or less dried out. The reason for that is very simple – it was the construction of protection dykes on the Latorica and Tisa rivers and the consequent groundwater level decrease in the region between these two rivers due to decreased recharge of groundwater from surface flows. The goal of the project is the feasibility study of possibilities and design of possible technical measures for revitalisation of Tice and Krčava rivers.

Next specific points of the project are the already realised water management measures on both sides of the Medzibodrožie region [3, 7]. They are completely other and do not coincide but utilising the proposed water management achieved in the project it could be possible to achieve the symbiosis in water management on both sides.

Since field measurements, analytical and theoretical results of the project have been presented already, this contribution will be limited to technical measures which have been proposed to be undertaken for improvement of the water management in the Medzibodorožie river basin region.

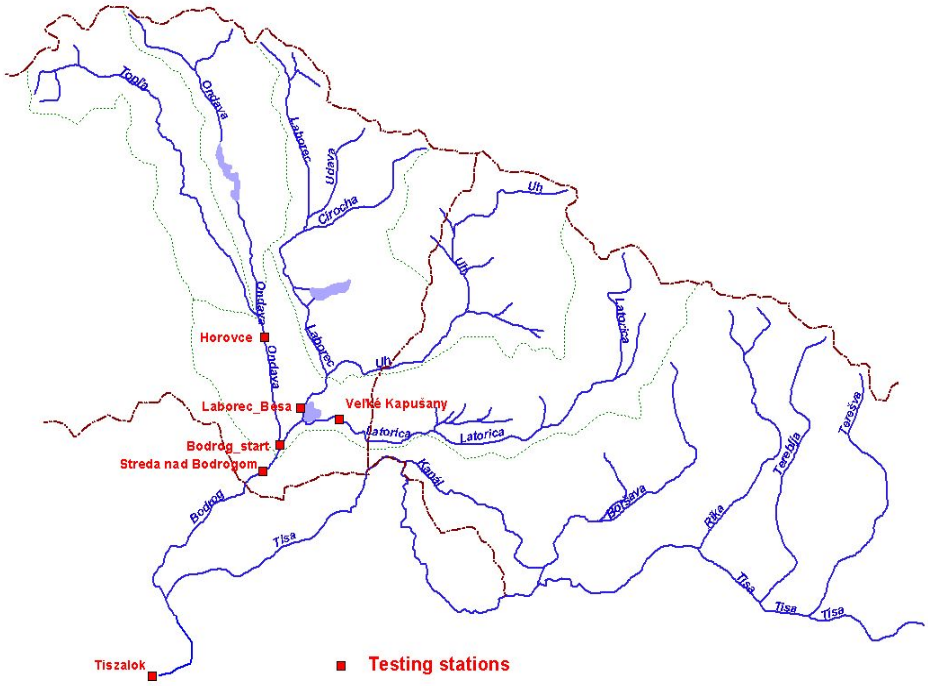

Fig. 1. Illustration of the solved region on the Hungarian – Slovakian and Ukrainian border: The Bodrog River Basin

2 Technical Measures for Improvement of Hydrologic and Water Management Conditions

After measuring, analysing and modelling works most convenient technical measures have been proposed [1, 5] to improve the water management system in the region.

Proposed technical solution has after put into operation to secure in the 44,4 km long Tice River (Fig. 2 and 3) and in some parts of its branch system the creation of a relative steady discharge regime with possibility of certain water level control due to requirements and needs of the ecosystem and population of individual touched villages. There were limited values of hydraulic head on a very long distance, i.e. each centimetre was very important in lowland region from hydraulic point of view. For this reason it was necessary to solve following hydrologic, hydraulic and technical problems:

- hydraulic solution and technical proposal of a gravitational uptake of required water quantity from the Latorica River in such way that even in dry periods will be enough water for creation of convenient water level regime in revitalised Tice River;
- hydraulic proposal and relevant technical solution of necessary measures in the Latorica River for securing the required water take-off into the Tice River;
- hydraulic solution and technical proposal for gravitational discharge take-off from the Tice River into the border Veľká Krčava River;
- flood protection securing in the vicinity of the revitalised Tice River by means of controlled inflow into the Tice River;
- hydraulic solution and technical proposal of measures and objects in revitalised Tice River to enable water control possibility for relatively stable discharges in the Tice River;
- utilisation of existing channel system in the region for solving the inflow into the Tice River and proposal of necessary measures in the channel system and needed gates;
- determination of marginal operation discharges Q_{min} and Q_{max} and determination of minimum and maximum operation water levels in the Tice River and in the channel system, as well as
- review of evaporation impact from water level and seepages into the groundwater on discharge balance in the Tice River and need for water supply from the Latorica River [1, 2].

All these factors have influenced the final proposal of technical measures as they will be presented bellow. The system can work as a natural one without building of any structures but the problem is that it would work just few days in the year and especially in flood periods. Therefore it was reasonable to create a system which would work almost through the whole year with no respect to discharge regime of the attached rivers.

The next purpose for solving the discharge and water level regime was to enable a flowing regime (strictly given by Nature protection agency) in the Tice River. Our first "technical opinion" was corrected several times by environmental agencies to achieve a full harmony between hydraulic and environmental requirements. The solution was divided into two parts – southern and northern part – as it is shown in the Fig. 2 and 3.

Fig. 2. Northern branch of connection of the Latorica water through Leleský channel into the Tice riverbed

Fig. 3. Southern branch of water connection from the Tice through Northern and Southern Radský channel into the Veľká Krčava River (on Slovak - Hungarian border)

3 Description of the System and its Operation

System which will secure during operation the required function of restored Tice flow in the nature consists from relatively separate parts:

- Weir construction in the Latorica River is a component of the technical solution. It is situated in rkm (river kilometer) 21.680 of the Latorica flow, in profile approximately 60 m above the bridge on road connecting Veľké Kapušany – Kráľovský Chlmec towns, which will secure backwater level behind it on operation level 99.5 m a.s.l., whereby conditions for gravitational inflow into the Tice River will be created even at minimum discharges in the Latorica River at consequent securing of hydrodynamic conditions of flood discharges [6].

- Channel 1 is created by using parts of existing channels – Leleský, Kaponský and Bačka with some corrections and completion by a new channel situated from outlet object in the left protection dam of Latorica in rkm 24.751 up to Leleský channel. The total length of channel 1 is 4 617 m and its discharge capacity at most un-convenient hydrodynamic conditions is approx. 1,0 $m^3.s^{-1}$.

- ***Channel 2*** consists of existing channels Pri prameni, Leleský, Veľký les IV and no-named channel. Its total length is 5 800 m and it is connected with Latorica through outlet structure in the left hand-side protection dam in its river chainage rkm 21.750. This channel enables to bring water gravitationally into the Tice River and its discharge capacity at most un-convenient hydrodynamic conditions is approximately 1.25 $m^3. s^{-1}$.

Revitalised Tice River is 44.4 km long and its route is untouched without any corrections in the width and length. Revitalised flow will be supported by Latorica water by means of both mentioned channels. The water level regime will be controlled by sluices on tubes at six damming profiles on the river. These are designed to be situated in following cross-sections:

1. Damming in rkm 0.000 at mouthing into Northern Radský channel;
2. Damming in rkm 4.940 when crossing the field road on the eastern part of the Rad village,
3. Damming in rkm 11.030 at crossing the state road from Zatín to Svinice;
4. Damming in rkm 17.910 at crossing the state road from Kráľovský Chlmec to Boľ on the southern part of the Boľ village;
5. Damming in 26.100 in taper part of the flow in meander northerly from the Kráľovský Chlmec town;
6. Damming in rkm 38.170 at crossing the field road in south-western direction from the Leles village (Fig. 4) [1,5] .

- Channel 3, which secures gravitational water flow from the Tice River down to the Veľká Krčava River, is 8479 m long and is created by connected channel system – Northern Radský, Somotorský and Southern Radský channels. Its discharge capacity is 2.3 $m^3.s^{-1}$.

Fig. 4. Damming profiles on the Tice River

4 Rivers, Channels and Structures

For obtaining the determined goals and for watering of former river bed of the Tice River is necessary to execute in individual rivers and channels following technical measures and improve the operation rules on rivers and their structures.

The Latorica River. On the Latorica River it is necessary to build over the bridge on state road from Veľké Kapušany to Kráľovský Chlmec a weir structure which will secure at even minimum discharges the required water level in the vicinity of two above mentioned outlets in the left hand-side protection dam of the Latorica River. The weir was designed as a bag weir with operation water level on 99.50 m a. s. l. It will be a two-field weir with the width of 12.0 m [6]. The height of the backwater will be 3.30 m. Due to operation on the weir appropriate water level will be secured (Fig. 5).

The Tice River. Recently the Tice River presents a system of wetlands which are connected and sometimes flown through. The river bed is fully grown with water flora and trees. The width of the river varies from 20 to 100 m, locally up to 150 m. The bottom is clogged with mud and several contaminants. The whole head of the river bed bottom is on 44.4 km long river approximately 3,51 m, what presents a mean longitudinal slope of the bottom $i_0 = 0{,}000079 = 0{,}079‰$.

Revitalisation of the river presents at the minimum technical measures of the river bed including cleaning and creation of conditions for securing a permanent discharge in the river and reconstruction as well as building of six damming profiles where the water levels in front of and behind are connected with two pipes (diameter D = 1000 mm) with sluices for the water control. These damming profiles are used for crossing the roads of local importance and field roads, as well.

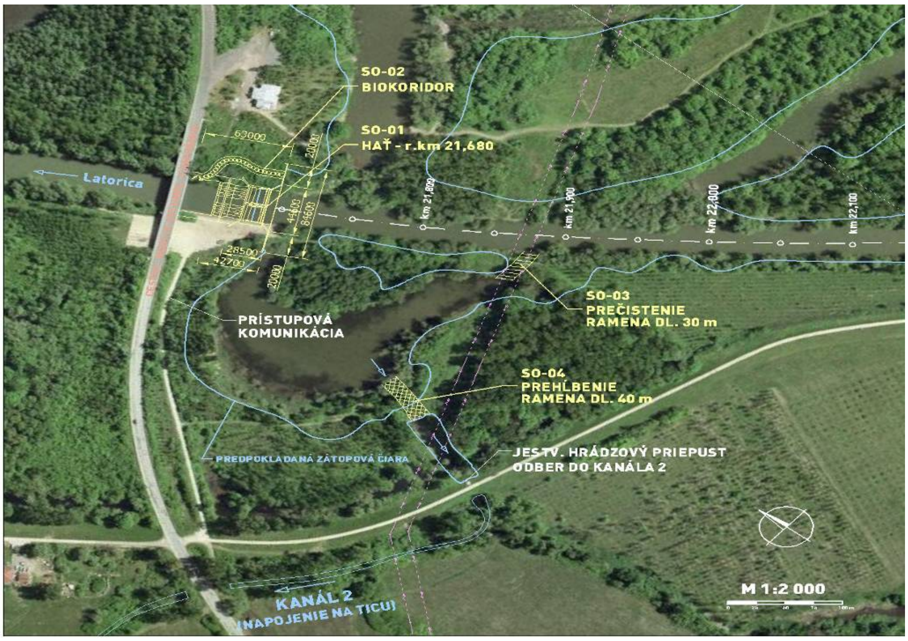

Fig. 5. The scheme of the bag weir on the Latorica River for damming the surface water

Total area of the water level in revitalised 44.4 km long Tice River is approximately 2.8 mil. m^2 and the whole water volume in the Tice River at maximum operation levels in individual reaches is 2.95 mil. m^3 of water.

The discharge regime in revitalised Tice River is given mostly:

- by possibilities of water uptake from the Latorica River in low-water periods and at discharges close to values of minimum discharges when the uptake should not exceed the 0.5 $m^3.s^{-1}$value,

- by discharge capacity of transport channels No.1 and No.2, which should be at most un-convenient hydrodynamic conditions – minimum head at the beginning and at the end of the channel – for channel 1 1.0 $m^3.s^{-1}$ and for channel 2 1.25 $m^3.s^{-1}$,

- at discharge capacity of connecting pipes on individual damming profiles utilising the control capability of sluices on these pipes, which will be used mainly during the first fulfilling of the system as well as during low discharges in the Latorica River.

- at discharge capacity of the channel 3 which is 2.3 $m^3.s^{-1}$.

5 Discharge and Water Level Regime in the System

The discharge regime in the Tice River is detemined by possibilities of the water uptake from the Latorica River and by capacity possibilities of the transport channel No. 3. Discharge in the Tice River will vary in limits from

$$0{,}5\text{m}^3.\text{s}^{-1} \le Q_T \le 2{,}3\text{m}^3.\text{s}^{-1}$$

The mean operation discharge which will be supported from the Latorica River is $Q_{oper,T} = 2.25\ m^3.s^{-1}$.

Water losses from revitalised Tice are given by seepage of the water into groundwater and by evapotranspiration. Determined losses were computed for the maximum evapotranspiration and were evaluated by the value $Q_{evap} = 0.13\ m^3.s^{-1}$.

Losses due to seepage from the Tice river bed into groundwater were calculated by means of mathematical modelling and were determined after watering of the Tice river bed at the value $Q_{seep}= 60\ l.s^{-1} = 0.06\ m^3.s^{-1}$.

Discharge in the Tice River will be after estimation of water losses

$$Q_{prev,Tice} = Q_{prev,T} - Q_{pries} + Q_{výp} = 2{,}25 - 0{,}06 - 0{,}13 = 2{,}06\text{m}^3.\text{s}^{-1}$$

Water level regime for individual reaches of the system was determined by numerical modelling by means of mathematical modelling of steady non-permanent flow.

Fig. 6. The hydro-ecosystem of the Tice River

6 Evaluation of Solution Scenarios

In presented contribution a proposal of water supply into the Tice River and its prospective watering is given using two alternative technical solutions [1].

1. Scenario of the solution considers the construction of weir with operation water level on 99.50 m a.s.l. in the Latorica River (Fig.5) above the bridge on state road from Veľké Kapušany to Kráľovský Chlmec with water transport into the Tice River and its consequent transport into the Veľká Krčava River bed. This variant of the solution enables the watering of the Tice and when controlling the water on required level as well the watering of dead branches of the river. The system can operate during the whole year except of short period when the main – drainage- function of Leleský and Somotorský channels has to be fulfilled.

2. Scenario differs from the first one only by the fact that in the Latorica River a natural surface water level regime will be secured and no weir will be realised. Other elements will be the same as in the first variant. The required operation discharge would be in mean year just in 35 – 40 days secured and in dry years the possibility to improve the water level regime in the Tice will be possible only in 2-5 days per year.

Solving collective **recommended the realisation of Scenario 1** of the solution which enables the renaturation and revitalisation of the Tice River and its dead branches and the creation of conditions for more qualitative life of inhabitants in touched villages.

7 Conclusions

The presented INTERREG IIIA project is dealing with inter-disciplinary and inter-regional problems of water and land-use management in a cross border Medzibodrožie region. It is not the first research project solving the water management in this region but certainly it is the first project which in wide spectrum and cross-border absorbs the water management problems of the Medzibodrožie region on both sides between the Slovak Republic and Hungary. Research works have been performed during 2006-2008 except of Slovak University of Technology in Bratislava by Water Research Institute and Institute of Hydrology, Slovak Academy of Sciences from Slovak side and by Soil and Agrochemical Institute, Hungarian Academy of Sciences, University of Technology in Budapest and by North-Hungarian Environment and Water directorate in Miskolc from Hungarian side.

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009

Keynote lecture

Possible Consequences of Global Change on Water Management and Hydraulic Structures

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Abstract. The objective of this paper is to analyse possible impacts of global change on water resources management. Here, in this paper global change considers impacts of direct human interventions on the water cycle, such as the development of hydraulic structures, land use changes, and second, the impacts of climate change on water resources systems. Based on several studies some general conclusions can be drawn for Central and South-Eastern Europe. The development of reservoirs and hydraulic infrastructure has led in several basins to substantial changes of the seasonal runoff pattern. The channelization of major rivers has shortened the propagation time of floods and the losses in retention capacity along major river courses lead to an increase of flood peaks. The temperature will go up until 2050 by about 2,0-3,0^{0}C and annual rainfall seems to decrease although different models yield different and even contradicting outputs.

Obviously these changes will have severe impacts on water management strategies and on the design of hydraulic structures. The design capacity of reservoirs serving irrigation and drinking water requirements should be revised and this may also hold for protective hydraulic structures, such as spillways and flood levees. However, it should be also considered that there is a large uncertainty in the climate simulation studies which delays the implementation of new design guidelines.

Keywords: Climate change, impact assessment, hydro power

1 Introduction

The paper is structured in the following way. First, the major human interventions in the water cycle are briefly analysed and then the knowledge about climate changes over Central and South-Eastern Europe is summarised. Subsequently, the hydrological consequences are identified and conclusions are drawn with respect to the design of hydraulic structures.

The hydrological characteristics of Alpine basins are subjected to several influences originating from direct human intervention and also from climate change. The human interventions refer to the development of large storage schemes for hydropower together with water abstractions and diversions from many small creeks. Also, winter tourism changed the water balance in small catchments where numerous ponds have been developed to serve water demands for artificial snow production. The land use changes characterised by a decrease of mountain pastures followed by an increase of forested areas contributed also to changes in the runoff regime. Further impacts on the hydrological cycle are to due extension of housing areas, channelization of rivers and other infra-structural development.

At the same time an increase of temperature is observed throughout the Alps over the last decades. This causes a shorter snow cover in the mid altitudes of the mountains and a decrease of glaciers. In some basins a clear trend in the runoff pattern has been already identified. In this paper an overview of the various influencing forces and activities is given and a methodology is outlined to asses the various impacts and to discriminate among them.

1.1 Hydropower

Hydropower plays an essential role in the Austrian energy market. About 65-70% of the electric power is generated by storage and runoff river schemes. Especially the large storage schemes which are located in the high Alpine region modify substantially the runoff pattern. Further, water transfer across catchment boundaries is relevant. Leopold [1] has shown that in winter time the mean monthly runoff of major Austrian rivers like Inn (Fig. 1) and Salzach may be increased by about 30 percent. In early summer, when runoff from snowmelt has its peak, the monthly values are decreased by about 10-15%. Even along the Danube these impacts may be observed but due to the large catchment area the changes in runoff are reduced to about 5 %. Besides changes in the seasonal runoff pattern changes in water temperature, in the flow velocity and in the interaction between surface and groundwater system have been identified [2, 3]. Additionally, impacts on the riverine ecosystems and on the sediment balance have to be considered, [4].

1.2 Winter Tourism

Increasing temperatures, reduced winter precipitation and increasing winter tourism demand for an extension of artificial snow production in the alpine areas. It is estimated [5] that from 93.000 ha of managed skiing areas in the European Alps about 24.000 ha are already equipped with artificial snowing facilities. Although small ponds are used to provide water in the winter period the runoff regime is affected, not only by abstraction but also due to longer snow covered slopes. Impacts on the vegetation and alpine ecosystems are identified [6, 7]. It can be readily assumed that areas put under artificial snowing will continue to increase. Winter tourism generates also a highly fluctuating water demand and waste water discharge. Although the alpine region is in general rich in water resources, shortages have been already observed in some centres.

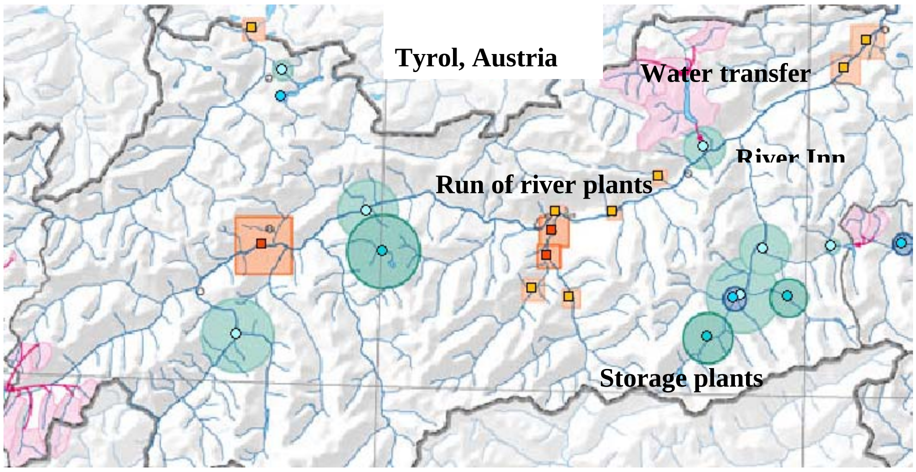


Fig. 1. Map of larger hydropower stations in Tyrol, a western federal state of Austria (from “Hydrologischer Atlas Österreichs”)

1.3 Land Use Changes

Major land use changes are observed in Alpine areas and their impacts on the hydrological response are subject of several research activities [8, 9, 10, 11]. The last reference refers to a recently completed study in which the impacts of land use changes, of infrastructural development including the extension of domestic areas, the development of forest access roads and of river channelization works on the runoff were investigated.

1.4 River Engineering Works

Long impoundment dams along the former river banks prevent from inundation of the former flood plain and contribute to a reduction of storage capacity during floods. Additionally, the flood waves are accelerated in their down stream propagation [12].

A study refers to the Danube [13, 14]. A data set of 85 peak flows of waves (water levels as well as discharge) was used to analyse travel times for the Danube section from Ybbs (stream kilometre 2060) down to Sturova (stream kilometre 1720). The observations cover the period from 1954 to 2002. The analysis indicates a remarkable increase in the flow velocity of flood peaks. While in 1954 the travel time of a flood peak was about 130 h for this 340 km long river section it takes now only 65 to 75 hours. This fact can be explained by the channelization of the Danube, by the impacts of runoff river schemes along this section and by impoundment works.

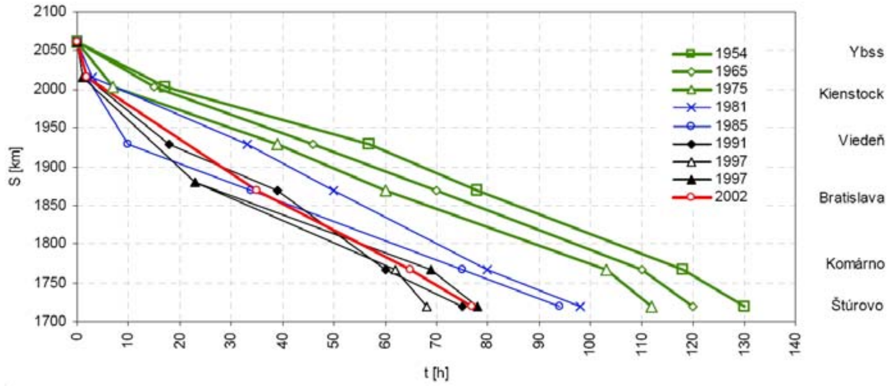


Fig. 2. Travel time of flood peaks in the Danube [14]

1.5 Observed Trends in Hydrological Time Series

Dependent on the selected time period trends in long term precipitation, runoff and temperature become visible. But these trends may change if another time period is being analysed. A recent investigation [15] indicates that in the Southern part of Austria, especially in the region south of the Alps, a long term decrease in precipitation and runoff exists which proofed to be statistically significant.

The analysis is based on the period 1900-2001 showing a decrease in the annual precipitation of about 100-300mm which corresponds to about 15-25% of the long term mean. Especially in the winter period the amount of precipitation decreases while in the summer period opposite characteristics may be observed at some of the stations. The significant decrease is observed in several basins.

Obviously, the runoff shows similar characteristics which may be aggravated by an increase in temperature. The mean annual runoff decreased in a large region by 20-40 % in the last 50 years. In general, this trend holds also for the seasonal pattern but local and temporal deviations are observed. Although winter precipitation decreased substantially the winter runoff is increasing in some catchments due to higher temperatures and resulting snow melt.

An analysis of the trends of the mean annual, the annual maxima and the seasonal runoff pattern for Bavaria and Southern parts of Germany, which has been executed in the framework of [16], does not show any significant changes in the mean values. For subsets of the time series, starting in 1965, an increase in runoff was found for a few stations. Sometimes trends in time series of hydrological extremes are reported but due to the limited observation length the analysis is rather questionable [17].

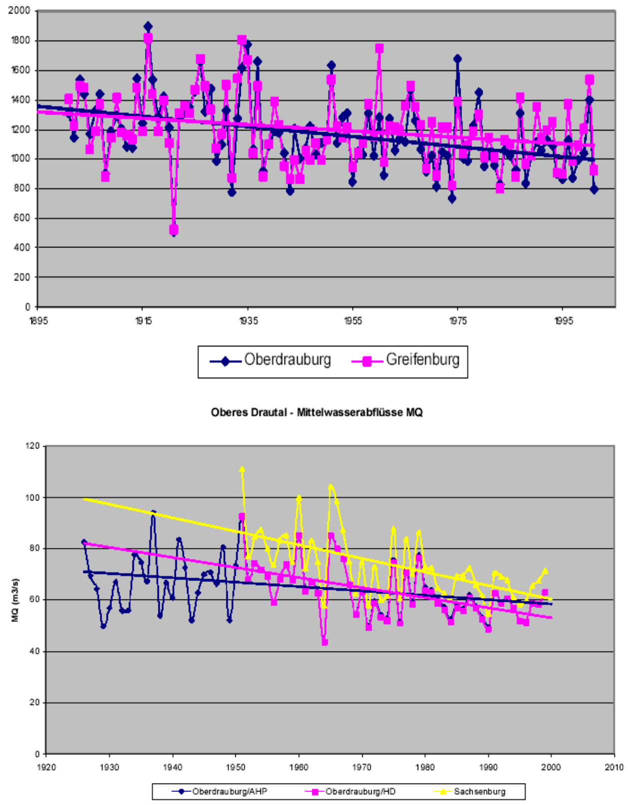


Fig. 3. Time series of annual precipitation in (mm) (left) and mean annual discharge (m3/s) (right) at gauging stations in Southern Austria [15]

2 Expected Climate Changes for Central and South Eastern Europe

Scenarios of future global climate are simulated by global circulation models (GCMs). These physical models, which are able to take into account the effects of radiative forcing caused by changes of the composition of the atmosphere, simulate the global atmospheric circulation and compute variables such as air pressure, air temperature and humidity at various levels in the atmosphere. These transient GCM experiments are based on emission scenarios of greenhouse gases, dependent on different global economic and technological development paths prepared by the Intergovernmental Panel on Climate Change [18].

Very often, the A1B scenario is used that refers to a mix of a strong globally linked technological development with social compatibility. It can be considered as an "optimistic scenario" and it constitutes one of the key scenarios in the IPCC [18] report. The scenario simulations cover a long time period from which the period from 1960-1990, being called the 1*CO_2 case, and the period from 2070-2100, which is named as the 2*CO_2 case, are compared. This terminology refers to the historic greenhouse gas concentration and a doubling of total greenhouse concentration by the end of this century. The climate models used in climate change studies are mostly based on a coarse spatial grid with each element covering several thousands of square kilometres (ECHAM4, ECHAM5, UK-HADCM3). Only a few nested climate simulations are available for Central and South Eastern Europe [19, 20, 21] (PRUDENCE, Ensemble, CIRCE, UBA-REMO) using a spatial resolution of a few hundred square kilometres for a grid element. But even grid cells with an area of 25 x 25 km are insufficient to study hydrological responses in an Alpine environment at the catchment scale.

The general conclusions from these simulations (Fig. 4) indicate that the temperature will substantially increase over Europe until 2050 by about 2,5-3,5^{0}C and by 3,5-4,5^{0}C until 2100. The mean annual precipitation is to increase towards Northern latitudes and to decrease towards the South. Austria is supposed to be located just in a transition zone of these geographical trends and therefore it is difficult to draw any conclusions for this Alpine area.

The decrease of precipitation in the Balkan region is estimated at about 10-15%, together with the more reliably estimated increase of temperature that will substantially increase the evaporative losses, major evaporative losses associated with increasing water stress can be expected.

Therefore their outputs cannot be directly used for local to regional impact studies and downscaling approaches are required to generate local climate features and outputs, such as time series of daily precipitation and temperature. Utilising these time series from different time periods, as indicated above, as an input to hydrological models or to water management models the impacts can be studied at the catchment scale to investigate the impacts on existing water management strategies. In a similar way statistical estimates of quantiles are obtained for recent and future climate conditions which provide the basis for design procedures.

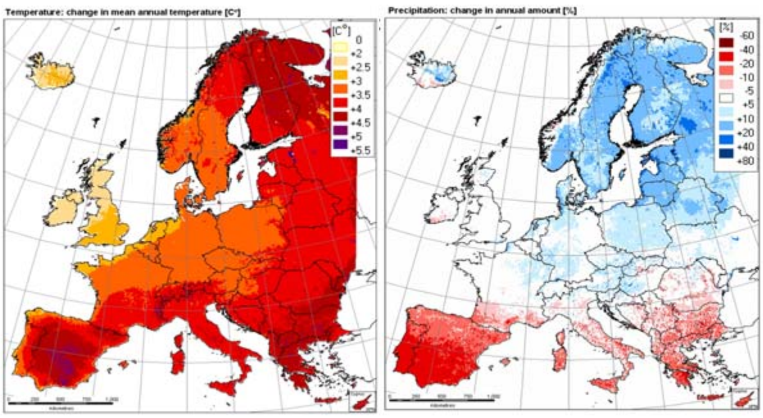


Fig. 4. Changes of mean annual Temperature (left) and mean annual precipitation [19] for 2070-2090

3 Methodological Approach

To discriminate among the consequences originating from human interventions in the water cycle and those induced by climate change some case studies from Austria are used.

A continuous, semi-distributed rainfall-runoff model, COSERO [22, 23], was applied to simulate the runoff patterns under different scenarios. The model requires temperature and rainfall as input data. For the impact assessment of climate change different time series of precipitation and temperature are required for different green house gas levels. To analyse impacts of various human interventions scenarios are run which reflect different land uses and states of the systems.

3.1 Parameterisation of Human Interventions

The catchment was subdivided into homogenous units, in many aspects similar to hydrological response units (HRUs) that were identified by intersections of sub-catchment boundaries, land cover information and elevation bands.

For each unit a set of parameters is required which refers to mean physical characteristics, some of them affected by anthropogenic measures, like interception losses, infiltration capacity, surface roughness, hill slope length and drainage density in each unit.

The hydrological module has several common features with the HBV-model [24] but a different infiltration routine has been applied to model the vegetation layer properly. The vegetation layer is parameterised according to its interception capacity, which may be seasonally variable, and its root depth.

The excess runoff was modelled by a kinematic wave approach to be able to consider the roughness of the surface and the land use changes in each HRU. The kinematic wave model is used to estimate the relation between the hillslope length and length to the outlet of the catchment in streams to the parameters of the hydrological model for surface flow, subsurface flow and stream flow, respectively.

The surface runoff was routed by two parallel reservoirs describing flow in the river and in the flood plain. The flood plain module was activated when the runoff capacity of the main river was exceeded. This approach was necessary to parameterise the hydraulic structures which were implemented in the last decades and which resulted in a channelised river course. The parameters for the two reservoirs were estimated by the help of a 1-D unsteady hydraulic model which used the available cross section data, roughness data and longitudinal profiles for historic and recent conditions. For the historic case it was assumed that the bankful discharge corresponded to a flood with a return period of about one year. For recent conditions the cross section data were used.

The estimates of the model parameters were adjusted to achieve a better fit to the observations but the adjustments were kept to a minimum and it was tried to keep them close to the parameters estimated from each land cover information. Details of the model are given in [11, Nachtnebel and Debene].

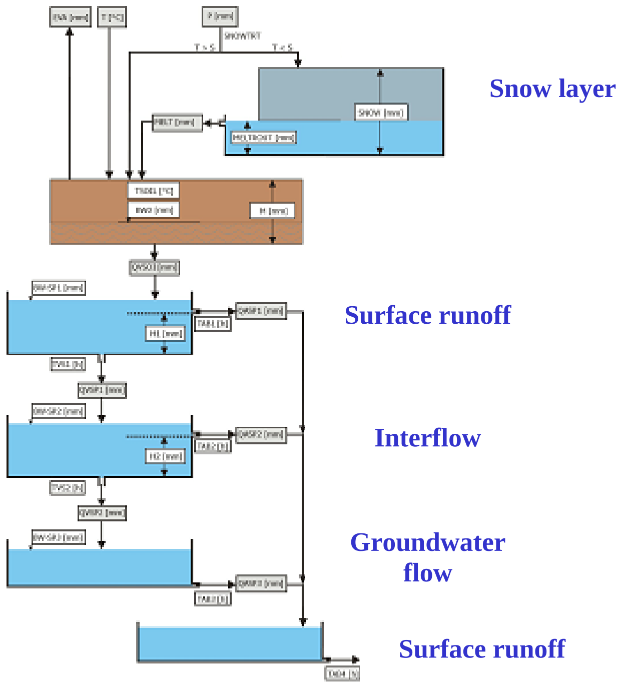


Fig. 5. Hydrological model for a subunit

3.2 Climate Change Impacts

Numerous studies have been published about impacts of climate change on the hydrological cycle [25, 26, 27, 28, 29, 30, 31, 12, 16, 32, 33, 34, 35]. The technique applied in this paper is based on the linkage of large scale meteorological features with local sets of observations. In particular, the air mass movement over Europe characterised by regional indicators of the spatial distribution of the geopotential heights is linked with basin averages of daily rainfall and temperature data from an alpine basin. The spatially averaged values may be disaggregated under consideration of their dependency on the altitude and its regional pattern which is described by a variogram [32]. The downscaled time series of rainfall and temperature serve as an input to the above described hydrological model (Fig. 6) which yields various time series characterising the accumulation and ablation of the snow layer, the storage of water in the soil and in the groundwater system, and finally runoff.

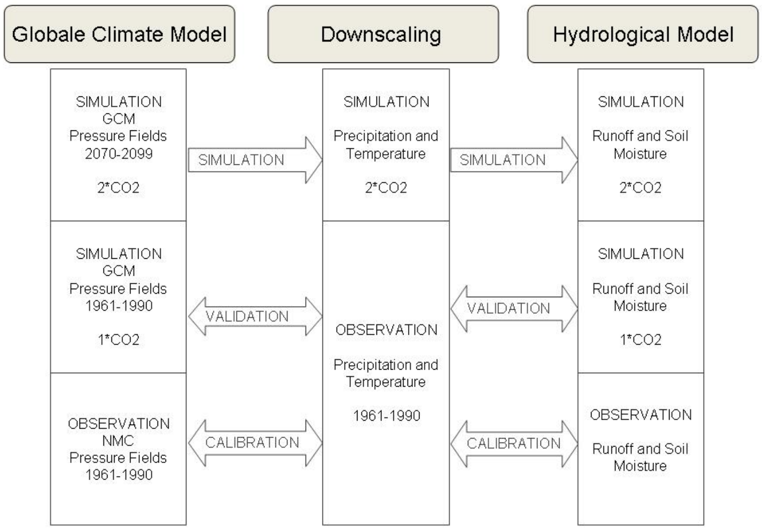
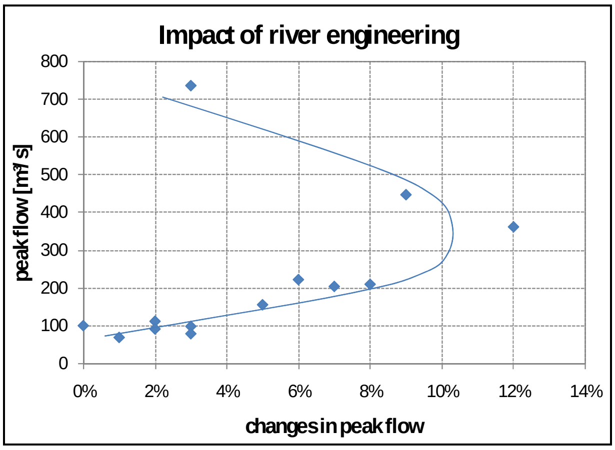


Fig. 6. Sequence of model application

4 Results

4.1 The impacts of Human Interventions on Floods

The hydrological model was run under different scenarios considering independently the various human interventions. Channelizing rivers and the building of dams lead to a decrease of concentration time and an increase of flood peaks of up to ten percent for floods with a recurrence interval of 30 years. The impact decreased with higher recurrence intervals. The impact of river training work is due to losses in inundation areas until the dikes are overtopped and the former flood plain is inundated again. Therefore, a highly nonlinear impact characteristic was identified. Detailed studies were executed in a pre alpine catchment (Traisen River) of about 750 km^2. Some results are presented below.



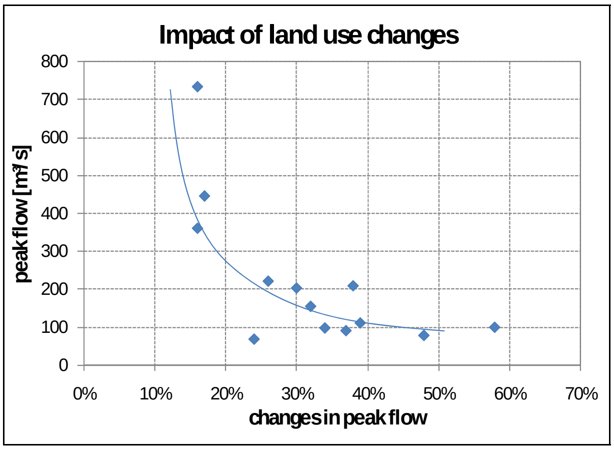


Fig. 7. Percentage of increase in peak flow (x-axis) dependent on its magnitude (y-axis)

Figure 7 demonstrates that flood peaks are increased until they reach the threshold of about 350 m^3/s. Under such conditions the flood levee systems raise the flood by about 10 %. Larger floods which already exceed the design discharge will inundate large areas and therefore the difference between the unengineered state and recent conditions is getting smaller.

The most significant impact results from the modified land use pattern, especially from the increase in forested area from 35% up to 80% of the catchment area in the last 100 years. Due to the higher evaporation rate of forest stands and a deeper root zone compared to grass land the storage capacity of the soil layer is substantially higher under a forest stand. This will lead to a decrease of flood peaks by about 20%. Under extreme rainfall events this effect will be diminuated.

In total, the change in land uses compensates for all the adverse impacts originating from extended housing areas, channelization of rivers, and forest access roads.

4.2 Impacts of Climate Change on Runoff Processes

The simulation runs of the large scale model were used to generate the regional input variables for the hydrological model. All the simulations were based on daily values but here only monthly figures are given.

It can be concluded that there seem to be no major changes in precipitation but due to an increase in temperature the evaporative losses increase and the runoff in the spring and summer time reacts accordingly. Dry periods will increase in intensity and in time but due to the comparable high amounts of precipitation in Alpine environments this will not constitute a major drawback. Obviously, the number of cold winter days is drastically reduced and winter sports in the headwater area will suffer or may even disappear. With respect to floods no major changes are expected. Winter floods may become more frequent. Summer floods don't show an increase in their magnitude.

Similar studies were executed in other alpine river basin [31, 36, 37]. For the period from 2070 to 2100, where we expect a doubling of greenhouse gases, all the simulations show some similar trends but also some regional differences can be identified. It is expected that the temperature will increase until the end of this century between 3,5^{0}C and 4^{0}C. The increase in summer is somewhat higher compared to the changes in winter. With increasing altitude the differences are slightly decreasing. As a consequence the number of frosty days decreases substantially, especially in the valleys by about 50%. The simulations indicate that even in higher mountain ranges the mean daily temperature will not fall below zero between May and October. The number of days with snow cover will decrease between 20-40% in the valleys. In all investigated areas the number of hot days will increase.

Obviously, the change in the temperature has also impacts on glaciers. Recent studies by Kuhn [38] indicate that already between 1960 and 1990 the volume of the Austrian glaciers has decreased from 22 to 17 km^3 and the respective glaciered areas changed from 567 to 471 km^2. This trend will be accelerated and this leads to increasing contribution of glaciers to runoff throughout the next 40 years. Then it is expected that the remaining glaciers are feeding the rivers with a decreasing contribution and until 2100 all of them might have disappeared.

No major changes are found in rainfall data for the Austria Alpine region. Overall, the annual amount has a tendency to decrease but in some regions an opposite trend is found. The late winter to spring precipitation seems to increase while in summer time the rainfall might be somewhat lower. In general it seems that the decrease of precipitation in the South will be somewhat lower than in the central mountain range. In some high alpine catchments in Western Austria some indication were found for increasing precipitation rates.

With respect to intensive rainfalls or floods no indication for an increased frequency or magnitude was found. Droughts and low flow periods will be more frequent and might last longer in the future.

4.3 Consequences for Hydropower

Considering results from historic trend analysis in runoff, considering future climate change scenarios from regional models and our results the following conclusions can be drawn for future changes of runoff and for hydropower generation.

1. Due to the heterogeneity of the Alpine region an analysis at the catchment scale is obligatory to derive reasonable results.
2. Due to an increase of evaporation and minor changes in precipitation the runoff is decreasing in most of the Austrian catchments, especially during summer. The opposite trend is found in winter time because of increasing winter temperature and reduced snow accumulation.
3. There is a gradient from South to North in historic data and in climate change scenarios. In the South the runoff has decreased and will continue to decrease while in high mountain areas and North of the Alps the precipitation is likely to increase but it is uncertain if this increase can compensate evaporative losses. The lower temperatures in the mountains mitigate evaporative losses and the contribution from glacier melt augment the runoff and thus it can be expected that in these catchments the runoff will slightly increase. The contribution from glaciers will have its maximum in about 40 years.

The consequences for hydropower generation were assessed on the basis of annual and seasonal changes in runoff. Further it was considered that an increase of low flows is beneficial for hydropower generation, especially in winter time when the demand is higher. On the basis that the mean annual runoff will decrease until the end of this century between 8-16% in the larger areas of Austria and that in high mountain areas and in Western Austria an increase may be found it can be concluded that the mean annual hydropower generation shall also decrease. Using the flow duration curve and typical design values for hydropower capacity we found for some catchment a decrease of annual output between 5-10%, even a 13% reduction was simulated for some sections. Considering the Western part of Austria it is estimated that the annual output will be decreased by 3-8%. Further losses are expected due to the requirements of the EU- framework directive of water.

The associated economic losses are partly covered by the fact that winter runoff is expected to increase between 3-15% which will result in a better fit of the supply to the demand.

Another climate change aspect relevant for hydropower is in the rise of the permafrost line. This will lead to mobilisation of rock material and as a consequence land slides, rock fall and erosive processes will become more frequent. These processes lead to increased sediment input into reservoirs and endanger the technical infrastructure of hydropower stations. A higher flood risk requiring redesign of spillways and release structures could not be detected by this approach although a higher probability for extreme events is frequently mentioned in the context of climate change.

5 Conclusions

In this paper a review of the various impacts on the hydrology of headwater areas was given. The direct impacts due to human interventions in a catchment were investigated. Especially, the consequences of the increase in housing areas, the channelization of rivers, changes in the land use pattern and the development of the infrastructure were analysed with respect to runoff and peak flow. For that purpose a semi-distributed and mostly physically based hydrological model was applied for a pre alpine region of about 700 km^2. The most critical point was the parameterisation of the various impacts and this was possible by integrating hydraulic with hydrological models.

It can be concluded that in this specific case the adverse impact of river impoundments aggravated the flood probability by about 10%. Above a certain threshold which corresponds to the bankful discharge the impact is smaller. Infrastructural development including forest access roads and extension of housing areas had only a small impact in the range of about 1%, which is to say negligible. The largest effect had the afforestation which took place in the last hundred years and which increased the forest area from 35-80%. As a consequence the storage capacity of forest stands is enlarged and during rainfall events several beneficial effects are observed. The flood peak is reduced and the rising limb of the flood wave is delayed. These impacts are comparable in quantity to those from hydropower operation in the Western part of Austria.

Climate change impacts were studied by the help of the same hydrological model which was driven by downscaled time series of rainfall and precipitation. The downscaling was based on past observations when classified large scale circulation patterns were linked with local observations. The comparison of different large scale models showed a large uncertainty and also the classification procedure showed that the historic frequency of typical circulation patterns did not properly conform to observed ones. In general, the simulations showed an increase in temperature but no clear changes in precipitation. It can be concluded that the runoff pattern will be severely modified, mainly because of the increased evaporation. No major changes in the flood frequency could be detected.

Acknowledgement

This work was partially supported by the following projects: “Hydklima” (GZ 30.610/1-VII/A/3/97) funded by the Austrian Ministry of Science and Education and the former Ministry for Environment, Youth and Family; “Abflussanalyse von Donau und Traisen” funded by the Provincial Government of Lower Austria. Sincere thanks go to DI. A. Debene und M. Fuchs, who have substantially contributed to this work.

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Topic 1

Hydraulic Engineering and Environmental Impacts

International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A03**

Judicial Usefulness of Hydraulics

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Abstract. Hydraulics is a well developed and organized branch of fluid mechanics, which enables solving many technical problems. But specialists in this field should be able to take up also some non-typical tasks. An important and interesting source of such problems is created by judicial controversies and disputes. The paper presents three case studies, illustrating real technical problems, investigated by means of hydraulic methods.

Keywords: Hydraulic systems, pipelines, dimensioning

1 Introduction

Hydraulics is a practically oriented branch of the fluid mechanics. Its history goes back to a period of antiquity (Aristotle 384-322 B.C., Archimedes 287-212 B.C.), and develops as an element of our civilization, especially connected with human needs. Cornerstones of hydraulics were established by Leonardo da Vinci (1623-1662), Isaac Newton (1642-1727), Daniel Bernoulli (1700-1782), Antoine de Chezy (1718-1798), Barre de Saint-Venant (1797-1886), Henri Darcy (1803-1858), Julius Weisbach (1806-1871), Robert Manning (1816-1897), Osborne Reynolds (1842-1926) and many others [2, 5].

As a result of the efforts of these distinguished investigators, a well organized and systematically described system of particular models came into being. This system creates a tool, which gives us possibility to solve many practical and scientific problems. Taking into account technical aspects, one can distinguish here some special fields of hydraulics (dealing with fluid-flow machines, flying or/and floating objects, hydrotechnical constructions, sanitary and environmental engineering…).

From the functional point of view, themes of hydraulic can be divided into some separated categories. Each one sets a peculiar "pattern of hydraulic calculations". One can enumerate here:

- Cubature systems: (reservoirs, reactors);
- Transit systems: (pipelines, open channels, free jets);

- Fluid-flow machinery: (pumps, fans, compressors, turbines, windmills…);
- Connectors and regulators (fittings);
- Measuring instruments and devices.

In parallel analysis one can consider kinematical characteristics of each flow, introducing (for one-dimensional motion):

- Unsteady and non-uniform flow:

$$v = v(x, t) \tag{1}$$

- Steady and non-uniform flow:

$$v = v(x) \tag{2}$$

- Steady and uniform flow:

$$v = \text{const.} \tag{3}$$

Having at our disposal so arranged system of mathematical equations, we can enter upon the considered problem and try to solve it, especially when the question belongs to the family of standard tasks. However such procedure quite often leads to a routine and automatic attitude towards technical problem, what can be dangerous when non-typical questions are considered.

Especially important group of these non-standard technical questions appears when different controversies or disputes have to be decided by a court or the other judicial structure. The paper is devoted to the presentation of some problems, belonging to this category.

2 Theft of the Diesel Oil

Description of the Situation. Just after the reloading of the diesel oil delivery from the railway siding to the fuel tanks, placed on a military area, it was stated, that the actual volume of this medium differs from the expected value. The shortage was equal to M_C = 55 040 kg.

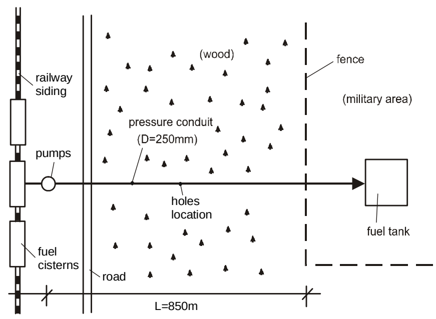

Fig. 1. Situation sketch

After the preliminary examination, prosecuted by the military police, it was found, that in the middle of the pipeline, leading from the siding to the tanks, the conduit was dug out (the diameter of the excavation was close to 1,00 m). Two holes were found in the pipe wall – an upper one (d_1=7 mm) and side one (d_2=10 mm). The ground in the neighborhood was strongly soaked by the fuel.

The time-scale of the fuel reloading looked as follows:

- 6.02., 8:00 – the last control measurement of the fuel volume in tanks before the reloading;
- 6.02., 9:30–15:00–pumping of the first part of delivery (7 cisterns; time of operation – 5,5 hours);
- 6.02., 15:00 – 9.02., 9:30–standstill of pumping station (66,5 hours);

9.02., 9:30–15:00–pumping of the second part of delivery (9 cisterns; time of operation–5,5 hours);

- 9.02., 15:00–final control measurement of the fuel volume (statement of the fuel loss);

In this situation there appeared a question about the possible variants of the fuel outflow through the holes in the pipeline wall.

Determination of the Holes Capacity. As it is shown in Fig. 2, one should consider two different cases:

- Outflow during the fuel reloading, through the both orifices (with the total discharge $Q_P = Q_{P1} + Q_{P2}$),
- Outflow during the pumping station standstill, through the side orifice only (discharge Q_B); the upper whole works as an aerator.

Making use of the classical formula, describing the orifice discharge [3]:

$$Q = \mu_0 \sqrt{2gH_c}\, S \tag{4}$$

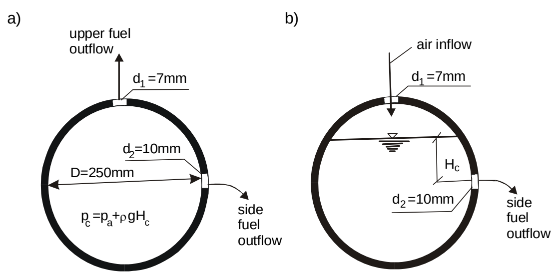

Fig. 2. Variants of the fuel outflow (a – under the pressure, b – gravitational)

One can calculate the characteristic flow intensity for each case. In the first variant it was assumed, that the mean overpressure head in the considered cross-section was equal to H_C=20m H_2O (as according to the information delivered by the pump operators, the initial overpressure head was equal to H_O=40m H_2O). For the coefficient of discharge $\mu_0 = 0.61$ [4], we have:

d_1 =7mm, S_1=8,5 mm^2, Q_{P1} =0,47 l/s,

d_2 =10mm, S_2 =78,5 mm^2, Q_{P2} =0,95 l/s.

The total outflow in this case equals:

$$Q_P = 1{,}42 \text{ l/s} \tag{5}$$

and for the fuel density 845 kg/m^3 gives the following mass discharge:

$$M_P = 1{,}20 \text{ kg/s} \tag{6}$$

For the gravitational outflow in turn, the overpressure head H_C can be expressed by the depth of the fuel layer above the orifice. As the real vertical profile of the pipline was not known, after some considerations it was assumed, that H_C = D/4 = 62,5 mm, what yields:

$$Q_B = 0{,}053 \text{ l/s} \tag{7}$$

$$M_B = 0{,}045 \text{ kg/s} \tag{8}$$

Possible Scenarios of the Outflow. The analysis presented below has a conditional character, because the moment, in which the holes were drilled, is unknown.

As the extreme values of the outflow time one can accept:

$$t_{max} = M_C/M_B = 55040/0{,}045 = 340 \text{ hours} \tag{9}$$

$$t_{min} = M_C/M_P = 55040/1{,}200 = 3 \text{ hours} \tag{10}$$

Assuming that just before the delivery beginning the pipeline was empty, one can calculate the theoretical decrement of the fuel M_T between two characteristic moments – from the first start of pumping station (6.02., 9:30), up to the final control of the fuel mass (9.02., 15:00). Together we have 11 hours of pumps operation and 66.5 hours of standstill, what gives:

$$M_T = (11 \cdot 3600 x 1{,}2) + (66{,}5 \cdot 3600 \cdot 0{,}045) = 47520 + 10773 = 58293 \text{ kg} \tag{11}$$

This value is surprisingly close to the stated fuel loss M_C = 55040 kg.

It is worth underlining, that these two holes have different orientation. This means, that the perpetrator (or perpetrators) of this damage apparently intended to steal the fuel during the standstill, taking the advantage of gravitational outflow.

It looks probably, that this individual (or individuals) was surprised by the sudden pumps operation, and decided to escape, leaving open holes. According to another possibility, these holes could be left after a fuel theft in the past, or prepared for the future activity. These remarks are the more important, as it would be very important and risky to take over about 60 m^3 of the fuel, from the neighborhood of a military object. Anyway, it looks that the immediate reason of the fuel loss was not a criminal action, but a continuous outflow through these orifices into ground.

3 Drowning of Reactors in a Malt-House

System Description. The schematic diagram of the malt production includes:

- washing of barley in vats,
- steeping of barley in vats,
- sprouting of barley and drying of malt in Saladin tanks.

Wash water, strongly polluted, outflows to the industrial sewerage. Wet barley is moved from the steeping vats to the Saladin tanks by the hydraulic installation. It happens that the steeping water, separated from grains through the perforated bottom of these reservoirs, can be discharged to the sewerage system, but in order to reduce the production expenses, this water (relatively pure) can be used once again, as a washing medium (Fig. 3).

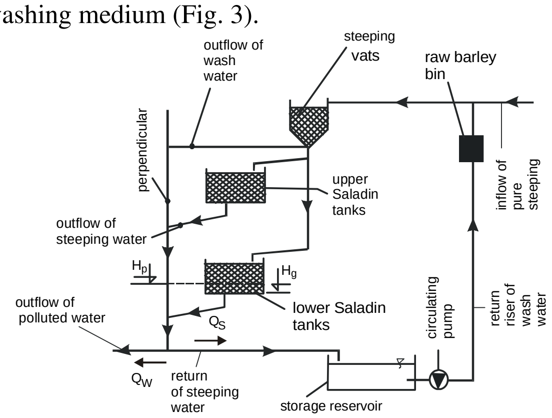

Fig. 3. Malt production system

Such a saving system was designed and performed in some existing malt-house. Unfortunately, during the first trial of exploitation after the modernization it came out, that the steeping water, flowing downwards from the upper set of the Saladin tanks, enters into tanks of the lower set, drowning the sprouting grains and spoiling the final product of the factory.

There appeared a controversy between the owner of the factory and the engineering firm, which has designed and performed the new installation.

Analysis of the Problem. In order to identify the real cause of the Saladin tanks drowning, the hydraulic parameters of the new installation were checked. As a first step, the capacities of perpendiculars were calculated.

The maximal discharge of such a pipe Q_M takes place when the conduit is cross-section is completely filled by water, and in its terminal cross-sections an atmospheric pressure prevails, what means, that the hydraulic slope along this pipe is equal to unity. Making use of the Darcy-Weisbach formula [3], one can write:

$$\frac{\Delta h}{L} = \frac{8\lambda Q_M^2}{gD^5} = 1 \tag{12}$$

According to the technological data, the effective discharge of steeping water Q_S equals:

$$Q_S = 0.0533 \text{ m}^3/\text{s} \tag{13}$$

For two different perpendiculars (D_1=200 mm and D_2=100 mm) we have:

$$Q_{M1}=0{,}239\text{m}^3/\text{s}>Q_S \tag{14}$$

$$Q_{M2}=0{,}139\text{m}^3/\text{s}>Q_S \tag{15}$$

These results mean that the capacity of vertical pipes is sufficient.

In the second order, the hydraulic losses for the horizontal return conduits were calculated. Applying the classical expressions of hydraulic [3] it was stated, that for both variants of the outflow (discharge of the wash water Q_W and return of the steeping water Q_S), this losses were two high. In consequence a damming up of water appears in perpendiculars, exceeding the level of the Saladin tanks bottom ($H_p > H_g$; see Fig. 3):

$$\text{for } Q = Q_{W,}\ H_p – H_g = 0,31\text{m} \quad (16)$$

$$\text{for } Q = Q_{S,}\ H_p – H_g = 0,72 \text{ m} \quad (17)$$

In consequence, the water can move back and drown the bulk of sprouting barley. One should pay an attention for the physical simplicity of this case.

4 Pollution of a River

Case Description. In one river, important because of the high quality of water and valuable landscape, a sudden environmental contamination was stated. One early Saturday morning (27.06., 6:00), an accidental observer noticed an extensive spot of pollution on the river free-surface. The water was turbid, covered by foam; one could feel a strong smell of fermenting matter. Within the compass of this spot, some dead fishes were visible. The total mass of these animals, caught out and utilized later, was equal to about 1.500 kg. Inspection of samples, carried of by specialists, indicated some specific traces of chemical burn, as if these fishes were in contact with some acid.

Inspection of the scene of event was carried out early afternoon of the same day by two policeman and two inspectors of environmental protection office. Walking along the river, they reached the estuary of a small open channel, and walking in this direction – to the open ditch, coming from the neighboring yeast factory.

Chemical analyses of water and ground samples, taken from different places showed, that the characteristics of the polluted water was identical with chemical constitution of the industrial waste water, out flowing from the factory, producing yeast (especially - bacteria *Sphaeroticus Natans* and fungus *Lettomitus Lactens*). Generally, this waste water contained very high load of the organic matter (COD= 6546 mg O_2/l) and nitrogen compounds (N_{total}=367mg/l). The reaction of samples was very low (pH =2.4).

The owner of the factory was accused of resulting in serious ecological harm. However he negated this statement, proving that his factory had been out of operation for the last two months, due to the control and routine repair of equipment. In this situation the problem was examined in more detailed way, by the group of specialists

Case Analysis. From the chemical point of view, the origin of waste water was completely clear. However, in front of the neighboring factory standstill, the actual source of the pollution should be found. The object owner suggested that the pollution could be caused:

- by another factory, placed somewhere in a higher cross-section of this river, or
- by the contaminated run-off from the surrounding fields, which were intensively fertilized by the yeast production wastes.

Both these possibilities were rejected, because of:

- positive results of water quality analyses, carried out by the laboratory of the not far distant water intake (Fig. 4),

- meteorological measurements of precipitation, according to which the last rain in this area was observed two days before the accident.

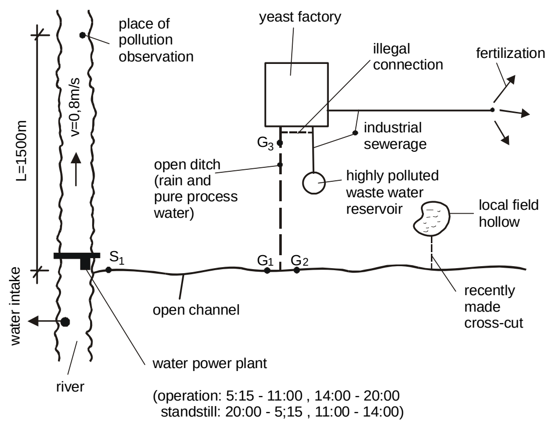

Fig. 4. Site plan

Especially profitable for these considerations was the schedule of operation of a water power plant, placed just below the estuary of the open channel (Fig. 4). Discussion of this time-table led to the conclusion, that the discharged amount of waste water have come to the main river through the open channel between 26.06., 19:30 (Friday evening) and 27.06., 5:15 (Saturday morning).

After a closer examination of the factory neighborhood, a small field hollow was found, several hundred meters from the object. When filled, this hollow could contain about 200 m^3 of water; however during the investigation this place was empty, although their bottoms was strongly wet and between its bank and already mentioned open channel a recently made cross-cut was found.

The chemical composition of samples, taken from this hollow showed that lately it contained some amount of highly polluted water. Analyzing documents from the office of the state environmental control service it became clear, that this hollow was a local natural reservoir, gathering rain water, coming from the run-off, strongly polluted, as the surrounding fields were very intensively fertilized by the yeast production wastes. What more, the owner of the factory was obliged by the environmental authorities to dispose the polluted content of this hollow and then – to liquidate this thread.

So, it became evident, that the hollow was emptied quite recently, on Friday evening (26.06.), and the discharged waste water was the main cause of the investigated accident. However there were two intriguing doubts:

- Why the hollow was emptied in so strange situation (as the owner was in conflict with the environmental authorities)?

- Why the nitrogen compounds concentration in this hollow was rather low and the reaction of water – rather neutral?

The answer was found during the inspection of the yeast factory. Review of the industrial sewerage showed, that there was an illegal connection (Fig. 4) between the system of highly polluted wastes (which officially were gathered in a special reservoir and then – transported in cisterns to the professional sewage treatment plant) and system of relatively pure water (officially discharged through the open ditch and open channel to the river. This junction was found by means of a liquid tracer.

Moreover it was stated, that for cleaning of the factory installation, a concentrated nitric acid was used. In these circumstances one could propose the following hypothetic course of events:

- Friday afternoon – a crew of workers, making the repair of installation, tries to get rid of some cubic meters of highly acidic washing waste water; they introduce this liquid into the illegal sewer (it is also possible, that this outflow was caused by some unexpected event or failure),
- Somebody better informed about the ecological effects of this step, anxious about legal consequences, decides to flush the system, using some relatively "safe" liquid,
- on account of the situation (standstill of the factory), the one and only accessible source of such "flushing fluid" was the content of the local ground hollow – it was strongly polluted, but did not contain acidic components,
- Somebody makes a cross-cut between the hollow and open channel (it is Friday late evening),
- waste water from the hollow moves along the cross-cut and open channel, up to the river, but – due to the closed valves of the water power plant – can not move along the river; close to the water plant it creates a spot of highly polluted water, surrounded by strongly acidic solution, introduced in the first order,
- Saturday morning–the waste water spot moves through the water plant valves and then–along the river; taking into account, that the mean velocity equals here 0.8 m/s, the distance l=1.500 m can be covered during t=30 minutes; this means that about 6:00 AM this spot reaches the place of the pollution identification (Fig. 4); the correlation with the witness observation is almost perfect.

One should underline, that the scenario presented above is strongly hypothetical. This concept will be verified by the court.

5 Conclusions

The paper is devoted to these aspects of practical applications of hydraulics, which are not included in standard and routine set of sanitary engineering problems. Especially interesting examples of this category of technical tasks one can find, analyzing judicial practice. Three case studies, presented in the paper, distinctly show the importance of elastic and differentiated education in this field.

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A06**

Flood Damage Risk Assessment for Hydraulic Structures in River Floodplain

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Abstract. The objective of this study was to work out a method for prediction of the safety and stability of hydraulic structures in river floodplains during multiple floods because of scour. A differential equation of equilibrium of the bed sediment movement for clear water was used and a method for computing the scour development in time at abutments in river floodplain during multiple floods was elaborated. A method for computing the equilibrium stage of local scour at abutments in the river floodplain was also developed. Methods were verified by experimental data. Influence on the equilibrium time of such factors as flow contraction rate, relative flow velocity, Froude number, relative grain size of bed materials, stratified bed conditions, and the time interval duration was also studied. For flood damage assessment the proposed scour calculation methods are used.

Keywords: Multiple floods, scour development in time, equilibrium scour, damage failure of structures, risk factor

1 Introduction

Between 1998 and 2004 more than 100 damaging floods happened, according to the European Commission. These floods caused more than 700 casualties and more than € 25 billion losses. The new EU Floods Directive 2007/60/EC aims to estimate, reduce and manage damaging floods in order to protect population, environment, cultural heritage and economic activity. Flood damage risk estimation and management for engineering structures in rivers in spite of the importance and complexity of the phenomena has not been studied yet.

Transport system infrastructure–roads, bridges, dams, and also water intakes in rivers are under a permanent impact of multiple floods. A lot of engineering structures are destroyed because of the scoured foundations during the floods.

During the past few decades equilibrium and temporal depth of scour at engineering structures has been studied. For computing the equilibrium depth of scour flow parameters at the peak of the flood with unrestricted or restricted duration (some hours or days) was used. However, in the nature the flow load on engineering structures are in the form of a hydrograph and multiple floods form scour holes.

General scour development in time for bridge crossings has been studied earlier [11] as well as the scour development during multiple floods at abutments [2], [7], guide banks [3], [5], [6], and water intakes in river floodplains.

The objective of this study was to work out a method for prediction of safety and stability of hydraulic structures in the river floodplains during multiple floods due to scour. By comparing the current scour depth with the designed scour depth, we could estimate the reliability of a structure in real-time during a flood or estimate necessity of the application of emergency protection measures.

A differential equation of equilibrium of the bed sediment movement for clear water was used and a method for computing the scour development at abutments in a river floodplain during multiple floods was elaborated. A method for computing the equilibrium stage of local scour at abutments in a river floodplain was also developed. Both methods were verified by experimental data.

Using the two mentioned methods the estimation of flood damage risk factor is proposed. The parameters of scour hole computed during multiple floods can be compared with its equilibrium parameters by the equations suggested, and the flood damage risk factor for abutments can be determined as a ratio h_s/h_{equil}, where h_s is a scour depth developed during/after multiple floods, and h_{equil} is an equilibrium scour depth. The closer the current depth value of the scour hole is to the equilibrium value of the scour hole, the more is the flood damage risk factor and the less is the stability of the structure. It is possible to estimate the current stability of the structure in the period of maintenance or to predict/compute it's stability at the design stage using the flood damage risk factor.

For implementation in practices we should have:

- Flood probability forecasting system, by obtaining the flood data;
- The flood damage risk factor for estimation of the structure safety and stability in expected floods;
- The warning system for the population in the flood risk area and to take protection measures for structures depending on flood damage risk assessment.

2 Methods for Scour Depth Determination

An integration of two scour depth estimation methods can be used in order to evaluate flood damage risk because of the scour at the engineering structure foundation.

The first method: for computing the scour depth, width and volume development in time during multiple floods a differential equation for the bed sediment movement for clear-water conditions was used. The calculation methods for scour development at abutments [2], [7], elliptical guide banks [5], [6], and straight guide banks [3] during multiple floods were elaborated and confirmed by experimental data. To determine the scour depth, width and volume development during multiple floods, hydrographs were divided into time steps of durations of 1 or 2 days. Each time step was in turn divided into time intervals up to several hours (Fig. 1).

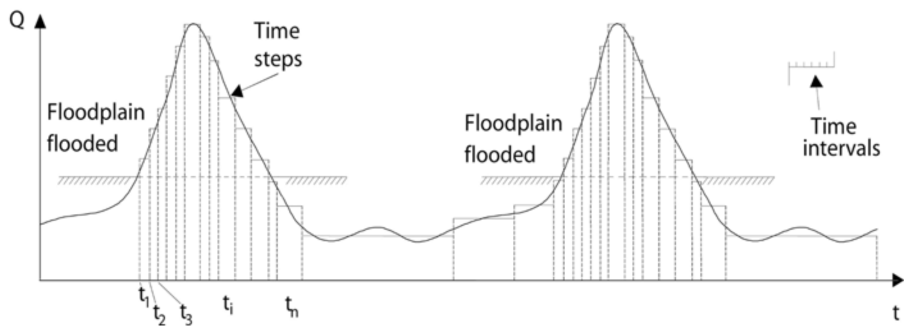

Fig. 1. Hydrograph divided into time steps and time intervals

For each time step, the following parameters must be determined: h_f – water depth in the floodplain; Q/Q_b – flow contraction rate; Δh – maximum backwater; d – grain size; H – thickness of the bed layer with d_i; γ – specific weight of the bed material. As a result, there is V_1 – local velocity, V_0, A, D_i, N_i, N_{i-1}, and h_s at the end of time intervals and finally at the end of the time step.

The scour development during floods can be computed by the formula:

$$N = \frac{t_i}{4D_i h_f^{\,2}} + N_{i-1} \tag{1}$$

where: $N=1/6\ x_i^6 - 1/5\ x_i^5$; h_f – water depth in the floodplain; t_i – time interval; D_i – constant parameter in steady flow time step; x_i – relative depth of scour. Using $N=f(x)$ for calculated N_i, we find x_i and the scour at the end of time interval:

$$h_s = 2h_f\left(x-1\right)\cdot k_m \cdot k_s \cdot k_\alpha \tag{2}$$

where: h_s – depth of scour; k_m – coefficient depending on the side-wall slope of abutment [11]; k_s – coefficient depending on the abutment shape; k_α – coefficient depending on the angle of flow crossing.

Theoretical analysis of the formulas (1) and (2) shows that relative depth of scour is a function of following parameters [4]:

$$\frac{h_S}{h_f} = f\left(\frac{Q}{Q_b};P_K;P_{Kb};\frac{Fr}{i_0};\frac{d_i}{h_f};\frac{V_l}{\beta V_0};\frac{h}{h_f};t_i;N_{i-1};H_{strat};k_m;k_s;k_\alpha;a;f_P;f_S;f_F;f_D\right) \tag{3}$$

where Q/Q_b – flow contraction rate; P_K – kinetic parameter of the open flow; P_{Kb} – kinetic parameter of the flow under the bridge; Fr/i_0 – Froude number referred to the slope; d_i/h_f – relative grain size; $V_l/\beta V_0$ – ratio of the local flow velocity to the velocity at which sediment movement starts; β – coefficient of reduction in the velocity because of vortex structures; h/h_f – relative depth of flow; t_i – time interval; N_{i-1} – parameter to calculate scour formed during the previous time step; H_{strat} – stratified riverbed conditions; k_m – side-wall slope of the structure; k_s – shape of the structure; k_α – alignment of the structure; a – unsteadiness of the flow; f_P – probability of multiple floods; f_S – sequence of multiple floods; f_F – frequency of multiple floods; f_D – duration of multiple floods.

According to the tests and the method suggested, scour development starts when the floodplain is flooded and usually stops at the flood peak or just after it. The scour development depends on the hydraulics of the flow, the river-bed parameters, and the probability, sequence, frequency, and duration of the multiple floods. At the flood peak a scour hole is usually formed, however, the scour process can be continued further, but it stops because the flood is time restricted. Thus the scour time is always less than the flood duration. At the next flood with the same probability, the scour process does not start when the floodplain is flooded, bet at a later time step, closer to the flood peak. This happens because of the scour hole developed in the previous flood. The duration of the scour process at the second and forthcoming flood is less than at the previous floods.

The scour depth, width and volume are thus increasing and are summing up for clear-water conditions during the floods with the same probability. When depth of scour reaches designed value for foundations, structure usually could be destroyed.

The second method: the designed depth of scour is calculated at the equilibrium stage [4], when local flow velocity V_{lt} is equal to the velocity V_{0t} at which sediment movement starts, and $h_s = h_{equil}$, where h_s – developed depth of scour and h_{equil} – equilibrium depth of scour calculated at the 1 % or 0.2 % flood probability.

The equilibrium stage of scour depth can be determined by the formula:

$$h_{equil} = 2h_f \left[\left(\frac{V_l}{k \cdot \beta \cdot V_0} \right)^{0.8} - 1 \right] \cdot k_m \cdot k_S \cdot k_\alpha \tag{4}$$

where: h_{equil} – equilibrium depth of scour; V_l – local flow velocity at the upstream corner of the structure [2]; k – coefficient of discharge changes because of scour [2]; β – reduction coefficient of velocity V_0 because of the flow vortex structures.

3 Influences of New Parameters on Equilibrium Time Value

The equilibrium depth of scour can be reached in equilibrium time. Duration of the clear-water scour development in time can be very long because small grains of the bed material are removed continuously from scour hole. The question is when to stop and accept the equilibrium stage of scour, and which criteria to use for the estimation.

Different criteria are proposed for the estimation of the equilibrium stage of scour. Three different criteria: first – in tests with different duration by measurement confirmation, second – of the pattern of time development of the scour depth, and third – estimation of scour increase in succeeding 24 hours in tests have been proposed [8].

The equilibrium stage of temporal scour under steady flow conditions has been defined [1], [8], [9].

The proposed method for computing the development in time of scour depth, width and volume during multiple floods allows accepting new criteria for estimation of equilibrium time for steady and unsteady flow conditions.

The laboratory tests were made in a flume 3.5 m wide and 21 m long. The tests with the rigid bed were made for different flow contractions and Froude numbers. The sand bed tests was to study the scour process, the changes in velocity at the abutments, the effect of main hydraulic parameters, Froude numbers, flow contraction, the grain size of the bed material. Some of experimental data for open flow conditions are presented in Table 1.

Table 1. Experimental data under open flow conditions in flume

Test	L (cm)	h_f (cm)	V (cm/s)	Q (l/s)	Fr	Re_c	Re_f
L1	350	7	6.47	16.60	0.0780	7500	4390
L2	350	7	8.58	22.70	0.0103	10010	6060
L3	350	7	10.30	23.60	0.1243	12280	7190

The test time was 7 hours and that is not enough to reach the equilibrium stage of scour. For each test developed scour depth after 7 hours was at different stage comparing with the equilibrium stage.

Methods for computing scour development in time during multiple floods were confirmed by different tests, which lasted 7 hours. To reach equilibrium stage of scour we prolonged the test duration by computer modeling of scour process. The influence of time interval duration, flow contraction rate, Froude number, grain size of the bed material, stratified bed conditions, and relative flow velocity on equilibrium time was studied in computer modeling of the scour development for steady flow conditions. In computer modelling same time intervals for the first 7 hours were used as in laboratory measurements, but later different time interval durations from 60 to 1440 min were used in computer modelling.

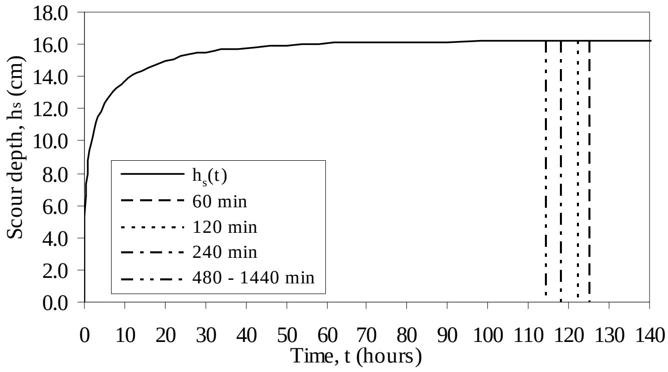

Fig. 2. Scour depth development with different time intervals (test SL1)

The equilibrium depth of scour calculated by formula (4) was compared with equilibrium scour depth computed in computer modeling tests and equilibrium time values were obtained for different time interval durations (Fig. 2). Equilibrium stage of scour in tests was reached when relative velocity $V_{lt}/\beta V_{0t}$ (where V_{lt} - local flow velocity in time t, V_{0t} – velocity at which sediment movement in time t) becomes equal to 1.

The comparison of equilibrium time obtained in the computer modeling tests with calculated equilibrium scour depth with different time intervals after 7 hours gave insignificant differences in the final results (see Fig. 2 and Table 2). The difference in equilibrium time was 3.9% - 11.7%. There was found no connection between time interval duration, equilibrium scour depth and equilibrium time.

The calculations of the scour development in time and the equilibrium depth of scour were made with a different grain of bed material: d_1 = 0.24 mm; d_2 = 0.5 mm; and d_3 = 0.67 mm. The increase of the relative grain size d_i/h_f leads to a decrease of the equilibrium time in order to reach the equilibrium depth of scour (Table 2).

It was found in modeling tests that new accepted parameters as: flow contraction rate Q/Q_b, Froude number for open flow conditions, relative flow velocity $V_l/\beta V_0$, grain size of the bed material, and stratified bed conditions has an influence on the value of the equilibrium time and the scour hole parameters for steady flow conditions. The influence of these parameters on the equilibrium of time has not been studied yet.

Table 2. Modelling results

Tests	Fr	Q/Q_b	$V_l/\beta V_0$ (d_1)	Equilibrium time, hours						
				60 min (d_1)	120 min (d_1)	240 min (d_1)	480 min (d_1)	1440 min (d_1)	60 min (d_2)	60 min (d_3)
SL1	0.075	5.27	2.00	125	122	118	114	114	54	40
SL4	0.075	3.66	1.79	111	104	102	98	98	30	18
SL7	0.075	2.60	1.45	42	40	42	42	42	11	7
SL10	0.075	1.56	1.20	24	22	22	22	22	4	0.33
SL2	0.1037	5.69	2.34	177	174	170	162	170	75	56
SL5	0.1037	3.87	2.15	133	130	126	122	122	50	53
SL8	0.1037	2.69	1.84	77	76	74	74	74	27	19
SL11	0.1037	1.66	1.30	38	36	38	38	38	6	2
SL3	0.1237	5.55	2.53	193	190	186	178	186	113	58
SL6	0.1237	3.78	2.49	172	170	166	162	170	63	48
SL9	0.1237	2.65	2.10	102	100	98	98	98	38	36
SL12	0.1237	1.67	1.41	41	40	38	38	38	11	4

According to the results in Table 2 the equilibrium time is increasing with the increase of the flow contraction rate Q/Q_b, Froude number for open flow conditions Fr, relative flow velocity $V_l/\beta V_0$, and is decreasing with the increase of the relative grain size of the bed materials d_i/h_f.

4 Flood Damage Risk Factor Estimation

The stability of the engineering structures in floods from the aspect of scouring can be evaluated by combining two calculation methods, namely the method of computing the scour development in time during multiple floods of different probability, frequency, sequence, and duration of floods according to Eq. (2) and the method of computing the equilibrium depth of scour by Eq. (4).

The scour-hole parameters calculated during or after multiple floods can be compared with the equilibrium parameters, and thus the stability of the engineering structure can be evaluated. The flood damage risk factor can be calculated for abutments and other hydraulic structures as a ratio between h_s, the scour depth during/after the floods of certain probability, and h_{equil}, the equilibrium scour depth in the floods of the same probability:

$$R = \frac{h_s}{h_{equil}} \cdot 100\% \tag{6}$$

The closer the current depth value of the scour hole is to the equilibrium value, the greater the risk factor and the lower the stability of the construction is. By using the flood damage risk factor, it is possible to estimate the current stability of the structure or predict/compute the time of a safe maintenance at the stage of design. If the developed scour-hole depth is close to the equilibrium conditions, the critical conditions are reached and the emergency scour-protection measures (riprap, rock riprap, sand/cement-filled bags, etc.) should be taken.

5 Conclusions

Transport and hydraulic structures in river flow are under constant impact of multiple floods and can be damaged because of local scour. The safety and stability of these structures ensure the protection of the population and reduce the environmental losses. Safety and stability of engineering structures because of scour during multiple floods has not been studied previously.

A differential equation of equilibrium of the bed sediment movement for clear water was used and a method for computing the scour development at abutments in river floodplain during multiple floods was elaborated. A method for computing the equilibrium stage of local scour at abutments in river floodplains was also developed. Both methods were verified by experimental data.

Influence on the equilibrium time of such factors as flow contraction rate, relative flow velocity, Froude number, relative grain size of bed materials, stratified bed conditions, and the time interval duration was also studied. It was found that there was no connection between time interval duration and equilibrium time. According to the results the equilibrium time is increasing with the increase of the flow contraction rate, Froude number for open flow conditions, relative flow velocity, and is decreasing with the increase of the relative grain size of the bed materials.

Using the two scour calculation methods the evaluation of the possible damage of abutments caused by multiple floods was proposed. The scour depth computed during multiple floods can be compared with the equilibrium stage of scour, and the flood damage risk factor can be determined as a ratio h_s/h_{equil}, where h_s is a scour depth developed during/after multiple floods, and h_{equil} is a equilibrium scour depth. The closer current scour depth value is to the equilibrium stage, the more is the risk factor and the less is the stability of the structure. By risk factor the current stability of the structure in the period of maintenance or in the design stage can be predicted.

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International Symposium on Water Management and Hydraulic Engineering

Ohrid/Macedonia, 1-5 September 2009

Paper: **A15**

Definition of Waterway According to 2D Flow Model

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Abstract. Arrangement of natural watercourses in order to protect against harmful impacts of water and with intention to allow navigation is a complex process which has to comply with the following basic conditions:
- Regulation of bed shall not cause deterioration of the existing eater regime,
- Regulation shall not cause devastation of banks and beds.

Mathematical flow models are very helpful in solving these complex processes. For the entire watercourse, one-dimensional flow model has been prepared on the basis of which two-dimensional models were provided for minor critical sections. The paper presents and defines an optimal solution for the critical watercourse sections on the basis of two-dimensional hydrodynamic mathematical flow model. For establishing of the model, detail surveying of bed geometry was performed as well as measurements of the water flow and the speed at characteristic water levels. On the basis of the obtained data, calibration of the model was done. Following the mathematical model, the effects of various regulation measures, interventions or structures were tested. The results of deepening of bed were modeled with regard to the flow speeds and the water levels. After that, the effects of building the regulation structures, groynes, their frequency and positions, were defined. On the basis of the data obtained by means of the mathematical model, i.e. the water levels and distribution of flows surrounding the regulation structures, the solution for regulation of the respective watercourse sections was provided.

Keywords: Hydrodynamic mathematical model, calibration, calculation, RMA2, regulation measures, waterway, groynes

1 Introduction

On the Sava River is defined the waterway dimensions of 70.0 x 2.5 m in water level of 95% duration. In order to ensure the required dimensions of the waterway on determine sections of the river it is necessary to make the bed excavation. The basic requirement is that after excavation beds cannot get to the differences in water levels greater than 10 cm. One-dimensional mathematical models were created for the entire flow of the Sava River in Croatia. By means of 1D model, the sections of the river and place where is greater difference in water level, has been detected. On these sections were detailed record cross sections of river approximately every 50 meters and the flow and velocity on the input and output cross sections at different water levels were measured. Measurements were made using acoustic electric meter (ADCP). An Acoustic Doppler Current Profiler (ADCP) is a type of sonar that measures and records water current velocities over a range of depths.

2 Applied Methodology

Methodology for the analysis the waterway and design critical sections is defined through the 5 steps as follows:

- Build the mathematical models of flow sections of the Sava River, production network elements to define the bed by GRID finite element method.
- Calibration models based on measured data. Variable that we use to synchronize the water level is Manning's roughness coefficient which after these steps is adopted as a constant value for each section and for each special level of the water face (large, medium or low water).
- Calculation of water levels for 95% duration water level in two-dimensional model.
- Calculation levels from the excavation waterway through the 95% duration water.
- Calculation of levels from the set regulatory structures – groynes, the sections on which the water level drop down more than 10 cm after excavation the waterway, on the 95% duration water.

3 Two-dimensional Mathematical Model of Flow

To create a two-dimensional mathematical model of the flow is used RMA2, one module of hydraulic model SMS (Surface-water Modeling System). RMA2 is a two-dimensional depth averaged finite element hydrodynamic numerical model. It computes water surface elevations and horizontal velocity components for subcritical, free-surface two-dimensional flow fields. RMA2 computes a finite element solution of the Reynolds form of the Navier-Stokes equations for turbulent flows. Friction is calculated with the Manning's equation, and eddy viscosity coefficients are used to define turbulence characteristics. Both steady and unsteady state (dynamic) problems can be analyzed.

The generalized computer program RMA2 solves the depth-integrated equations of fluid mass and momentum conservation in two horizontal directions. The forms of the solved equations are:

$$h\frac{\partial u}{\partial t}+hu\frac{\partial u}{\partial x}+hv\frac{\partial u}{\partial y}-\frac{h}{\rho}\left[E_{xx}\frac{\partial^2 u}{\partial x^2}+E_{xy}\frac{\partial^2 u}{\partial y^2}\right]+gh\left[\frac{\partial a}{\partial x}+\frac{\partial h}{\partial x}\right]$$
$$+\frac{gun^2}{\left(1.486h^{1/6}\right)^2}\left(u^2+v^2\right)^{1/2}-\zeta V_a{}^2\cos\varphi-2hv\omega\sin\phi=0 \tag{1}$$

$$h\frac{\partial v}{\partial t}+hu\frac{\partial v}{\partial x}+hv\frac{\partial v}{\partial y}-\frac{h}{\rho}\left[E_{yx}\frac{\partial^2 v}{\partial x^2}+E_{yy}\frac{\partial^2 v}{\partial y^2}\right]+gh\left[\frac{\partial a}{\partial y}+\frac{\partial h}{\partial y}\right]$$
$$+\frac{gvn^2}{\left(1.486h^{1/6}\right)^2}\left(u^2+v^2\right)^{1/2}-\zeta V_a{}^2\cos\varphi-2hu\omega\sin\phi=0 \tag{2}$$

$$\frac{\partial h}{\partial t}+h\left(\frac{\partial u}{\partial x}+\frac{\partial v}{\partial y}\right)+u\frac{\partial h}{\partial x}+v\frac{\partial h}{\partial y}=0 \tag{3}$$

where: h- water depth, u and v velocities in the Cartesian directions, x,y and t Cartesian coordinates and time, r density of fluid, E eddy viscosity coefficient, xx normal direction on x axis surface, yy normal direction on y axis surface, xy and yx shear direction on each surface, g acceleration due to gravity, a elevation of bottom, n Manning's roughness coefficient, 1.486 conversion from SI (metric) to non-SI units, ξ empirical wind shear coefficient, Va wind speed, ψ wind direction, ω rate of earth's angular rotation, ϕ local latitude.

Equations (1), (2), and (3) are solved by the finite element method using the Galerkin Method of weighted residuals. The elements may be one-dimensional channel reaches, or two-dimensional quadrilaterals or triangles, and may have curved (parabolic) sides. The shape (or basis) functions are quadratic for velocity and linear for depth. Integration in space is performed by Gaussian integration. Derivatives in time are replaced by a nonlinear finite difference approximation. Variables are assumed to vary over each time interval in the form:

$$f(t)=f(t_0)+at+bt^c \quad t_0\le t<t_0+\Delta t\ t0 \tag{4}$$

which is differentiated with respect to time, and cast in finite difference form. Letters a, b, and c are the constants. The solution is fully implicit and the set of simultaneous equations is solved by Newton-Raphson non linear iteration scheme.

4 Model Calibration

Input data for model calibration were measured flow on the upstream and water levels at the downstream cross section of the river sections.

Table 1. Measured data: flow, medium-speed and water level

	Low water		Medium water		High water	
	P551	P567	P551.	P567	P551.	P567
Q [m^3/s]	287,7	275,4	1142,4	1151,3	1627,6	1704,3
V_{sr} [m/s]	0,39	0,80	0,79	1,02	0,85	1,01
H [m a.s.l.]	79,86	80,08	82,67	82,67	84,38	84,42

After the model calibration we get the values of Manning's roughness coefficient for the each area of water.

Table 2. Manning's roughness coefficient for low water

Cross section	Distance	Manning's
P551	110+381,87	0,015
P559	112+029,83	
P560	112+303,30	0,023
P567	113+621,93	

5 Calculations of Water Level

Manning's calibration coefficients obtained for the measured low water is used for calculation of water level for flow 95% duration. Flow 95% duration for the river section is Q_{95} = 297.0 m^3/s, and it is the subject of further calculations. The first calculation is the definition of the initial water level of the natural state.

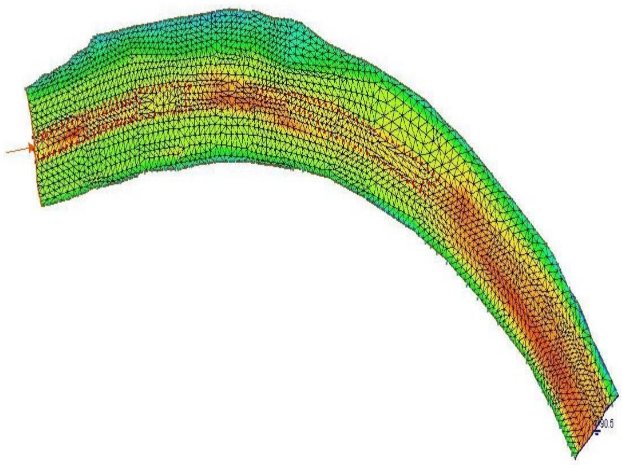

Fig. 1. Model with waterway

After that is done correction of model, excavation waterway (Fig. 1), and repeats the calculation. Analysis of the results detected the falling down of water levels greater than 10 cm. The model is in certain parts of the river bed, setting up regulatory structures, groynes, and was repeated calculation to obtain solutions. All the groynes are modeled so that their crown is on the level 95% water +1.00 m (Fig. 2).

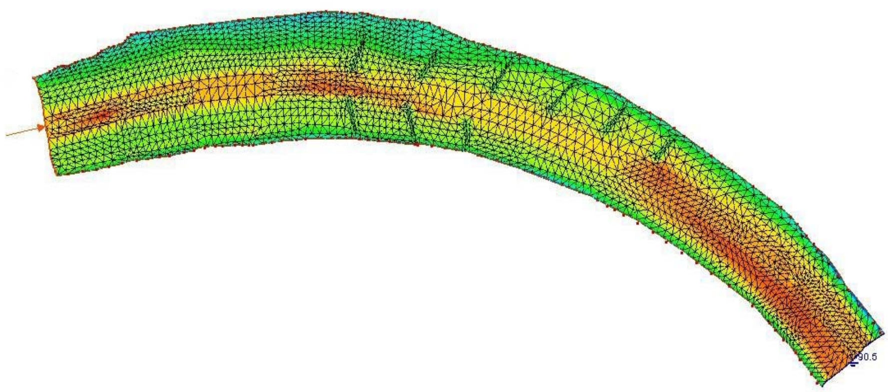

Fig 2. Model with regulatory structures-groynes

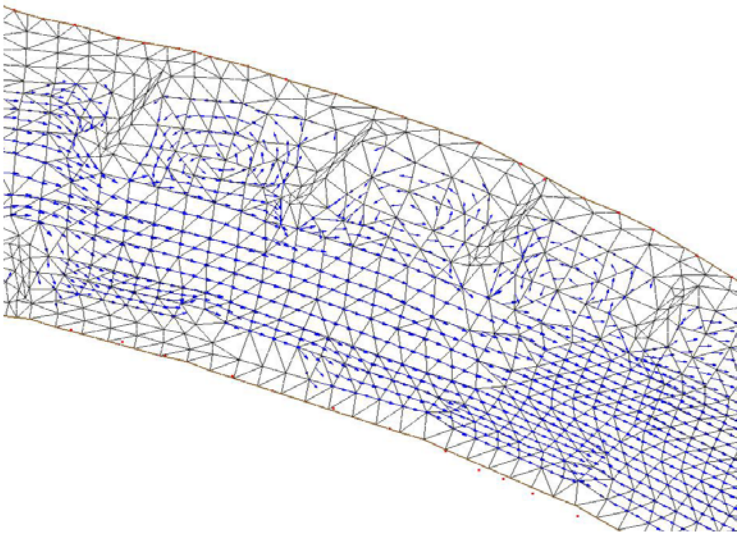

Fig. 3. Computed velocity vectors in the river bed

The solution must satisfy the set condition that the difference between water level in Natural River and in built waterway must not be less than 10 cm (Fig. 4).

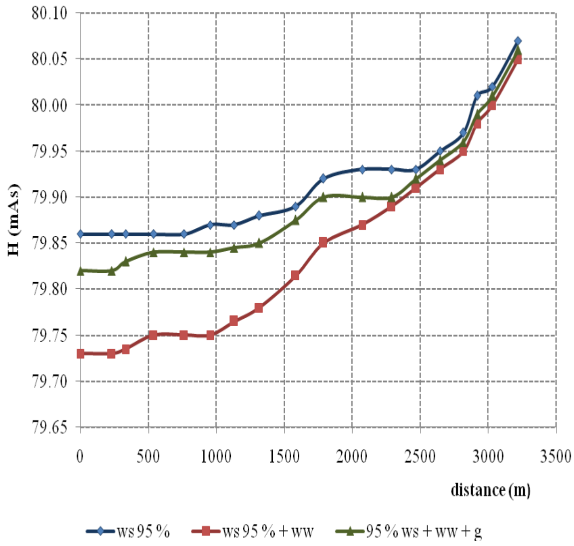

Fig. 4. Results of two-dimensional model – water surface profiles

6 Conclusions

With two-dimensional mathematical models of smaller section of natural watercourses, it is possible to describe the natural state of watercourses. The basic requirement is a quality measurement that allows us to define the input data and calibrate the model. Testing with models it is possible to examine a number of variants of which then selects the best solution in the technical and economic sense. The presented case study on Sava River section in Croatia showed the applicability and accuracy of RMA2 model. Water surface profiles with 95% probability were computed on the basis of which the regulation measures were designed.

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A18**

Balance Error Generated by Numerical Diffusion in the Solution of Muskingum Equation

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Abstract. In the paper the conservative properties of the lumped hydrological models with variable parameters are discussed. It is shown that in the case of the non-linear Muskingum equation the mass balance is not satisfied. The study indicates that the mass balance errors are caused by the improper form of equation and by the numerical diffusion which is generated in the solution. It has been shown that the classical way of derivation of the non-linear Muskingum equation leads to non-conservative form which does not guarantee the correct solution. The balance errors can occur not only in the case of non-conservation form but can occur even for an equation written in the conservative form. Using the modified equation approach it was possible to show why the numerical solutions of equations in the conservative form can suffer from the balance errors.

Keywords: Hydrological models, Muskingum equation, conservative and non-conservative form, accuracy analysis, numerical tests

1 Introduction

The simplified flood routing models as the kinematic wave equation or the Muskingum model are derived from the Saint-Venant equations by introducing additional assumptions. These simplified equations are based on the full continuity equation and adequately reduced dynamic equation [3]. If the inertial and pressures forces are neglected in the dynamic equation, the Saint-Venant model is reduced to the kinematic wave model which can be expressed as:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = 0 \qquad (1)$$

$$A = \alpha Q^m \tag{2}$$

where Q is the discharge; A is the cross-sectional area, α and m are parameters which depend on a formula of steady flow. In Manning case:

$$\alpha = \left(np^{2/3} \big/ s^{1/2} \right)^m ; \quad m = 3\!/\!5 \tag{3}$$

where n is the Manning coefficient; p is the wetted perimeter and s is the bottom slope. The kinematic wave model (1)-(2) is written in the form of the set of two equations, which can be reduced to one transport equation with one dependent variable. In this case we obtain the kinematic wave model in the form of non-linear transport equation:

$$\frac{\partial Q}{\partial t} + C(Q)\frac{\partial Q}{\partial x} = 0 \tag{4}$$

where $C(Q)$ is the kinematic wave celerity:

$$C(Q) = \frac{1}{\alpha m} Q^{(1-m)} \tag{5}$$

If the kinematic wave celerity is not a function of discharge and takes constant value $C(Q) = C =$ constans, then kinemaatic wave equation becomes a linear.

The Muskingum model belongs to the hydrological lumped models. In order to derive this model the continuity equation (1) is integrated over considered channel reach and the dynamic equation (2) is entirely neglected. Finally one obtains the storage equation:

$$\frac{dV}{dt} = Q_{j-1} - Q_j \tag{6}$$

where V is a storage of the channel reach of length Δx, Q_{j-1} and Q_j are inflow and outflow respectively, j is index of cross–section.

Introducing additional relationship between storage, inflow and outflow

$$V = K\left(X{\cdot}Q_{j-1} - (1-X)Q_j \right) \tag{7}$$

into the obtained storage equation (6) leads to the Muskingum model [1]:

$$X\frac{dQ_{j-1}}{dt} + (1-X)\frac{dQ_j}{dt} = \frac{1}{K}\left(Q_{j-1} - Q_j \right) \tag{8}$$

where X and K are constants. This is classical way of derivation of the Muskingum model.

2 Linear and Non-linear Muskingum Model

The Muskingum model can be considered as a semi-discrete form of the kinematic wave equation [7]. In this case the Muskingum model is directly received as a result of the discretization in space of the kinematic wave equation. The approximation is carried out at the grid at the point P (Fig. 1). In result the partial differential equation (4) for C = const is reduced to the ordinary differential equation:

$$\frac{dQ_P}{dt} + C\frac{Q_j - Q_{j-1}}{\Delta x} = 0 \tag{9}$$

where Q_P is the discharge at the point P described by the linear interpolation between the cross-section j and j+1along channel reach of length $\Delta x = x_{j+1}-x_j$:

$$Q_P = XQ_{j-1} + (1-X)Q_j \tag{10}$$

where X is the weighting parameter, which ranges from 0 to 1. It is defined as follows:

$$X = \frac{x_j - x}{x_j - x_{j-1}} \quad \text{for} \quad x_{j-1} \le x \le x_j \tag{11}$$

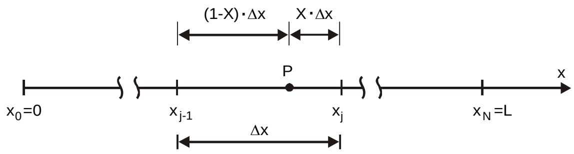

Fig. 1. The discretization in space applied for the kinematic wave

Comparison of Eq. (8) and (9) yields that the Muskingum model is consistent with a semi-discrete form of the kinematic wave linear equation on the following condition [2]:

$$C = \frac{\Delta x}{K} \tag{12}$$

Therefore assuming constant celerity one obtains the Muskingum model in the form of Eq. (8).

Above procedure can be used to derive other forms of the Muskingum model. Semi-discretization of Eq. (4) with the kinematic wave celerity $C(Q)$ as variable leads to the non-linear equation of the Muskingum model:

$$X\frac{dQ_{j-1}}{dt} + (1-X)\frac{dQ_j}{dt} + \frac{\left(XQ_{j-1} + (1-XQ_j\right)^{m-1}}{\alpha \cdot m \cdot \Delta x}\left(Q_j - Q_{j-1}\right) = 0 \tag{13}$$

3 Non-Conservative and Conservative Form of Equation

The Eq. (13) is written in the non–conservative form. The numerical solution of non-linear equation in this form does not guarantee consistency with the physical conservation laws mass or momentum. It is possible to eliminate the balance error if the appropriate conservation form of equation is used. To derive the conservative forms of the Muskingum model, in the first order the kinematic wave equation (4) is reconsidered. This equation can be converted into the equivalent conservative form:

$$\frac{\partial Q}{\partial t} + \frac{1}{\alpha \cdot m \cdot (2-m)} \frac{\partial Q^{2-m}}{\partial x} = 0 \tag{14}$$

Another one can be derived directly from Eqs. (1) and (2). Combining these equations with assumption that α = const, one obtains:

$$\frac{\partial}{\partial t}\left(\alpha Q^m\right) + \frac{\partial Q}{\partial x} = 0 \tag{15}$$

Using the kinematic character of the Muskingum model, the conservation forms of this model are derived from the corresponding equations of the kinematic wave model. The semi–discretization of Eq. (14) yields the first conservative form of the non–linear Muskingum equation:

$$X \frac{dQ_{j-1}}{dt} + (1-X)\frac{dQ_j}{dt} + \frac{1}{\alpha \cdot m \cdot (2-m) \cdot \Delta x}(Q_j^{2-m} - Q_{j-1}^{2-m}) = 0 \tag{16}$$

In similar way the second conservation form can be derived by semi-discretization of Eq. (15):

$$\frac{d}{dt}(X \cdot Q_{j-1} + (1-X)Q_j)^m + \frac{1}{\alpha \cdot \Delta x}(Q_j - Q_{j-1}) = 0 \tag{17}$$

The form of solved equation is not important as long as the smooth solutions are considered. The choice of equation's form becomes an issue when the initial condition contains a discontinuity or/and the approximation of equation introduces numerical errors. In such a case various conservative forms will generate the solutions of various accuracy.

4 Mass Balance for the Non-linear Muskingum Equation

The basic physical laws of the conservation of mass and momentum can be expressed in both differential and integral forms. In order to analyze the conservative properties it is convenient to write the governing equation in the integral form. Let us consider a channel reach of length L in which the unsteady flow described by Eqs. (1) and (2) takes place during the time interval T. It is well known that Eq. (1) represents the mass conservation principles. To obtain the global conservation laws this equation is integrated over the domain of solution, i.e. for $0 \le x \le L$ and $0 \le t \le T$. Assuming that the time interval T is sufficiently long and after passing of the flood wave the discharge comes back to its initial value, the mass balance takes the following form:

$$\int_0^T (Q(0,t) - Q(L,t))dt = 0 \tag{18}$$

Introducing:

$$M_0 = \int_0^T Q\left(0,t\right)dt, \qquad M_L = \int_0^T Q\left(L,t\right)dt \tag{19}$$

the relative mass balance error is defined as follows:

$$\Delta M = \frac{M_L - M_0}{M_0} \cdot 100\% \qquad (20)$$

where ΔM is the mass balance error expressed in %, M_0 and M_L are the total volume of inflow and outflow respectively within the time period of T.

Having three different forms of the non–linear Muskingum equation one can compare their properties. First of all we should explain which conservative quantity is preserved by them. To answer these questions, the Eq. (13), (16) and (17) must be integrated over the solution domain. In this case the integration over a channel reach is not needed since, the Muskingum model is actually the kinematic wave integrated in space – over an interval of length equal to Δx.

Let us integrate the non–linear Muskingum equation, written for one interval – reservoir, in the non–conservative form Eq. (13) over time T:

$$\int_0^T \left(X \frac{dQ_{j-1}}{dt} + (1-X) \frac{dQ_j}{dt} \right) dt = \frac{1}{\Delta x \cdot \alpha \cdot m} \int_0^T (XQ_{j-1} + (1-X)Q_j)^{1-m} (Q_{j-1} - Q_j)\, dt \qquad (21)$$

Since the Muskingum model is applied for a cascade of N reservoirs bounded by cross–section $j-1$ and j having length of Δx, the outflow from the preceding reservoir is the inflow to the next one. Finally, if the time interval T is long enough, the following integral equation is obtained:

$$\int_0^T \left((X \cdot Q_0 + (1-X)Q_1)^{1-m} Q_0 - (X \cdot Q_{N-1} + (1-X) \cdot Q_N)^{1-m} \cdot Q_N \right) dt = \int_0^T R_M \cdot dt \qquad (22)$$

in which

$$R_M = \sum_{j=1}^{N-1} \left[(X \cdot Q_j + (1-X)Q_{j+1})^{1-m} \cdot Q_j - (X \cdot Q_j + (1-X)Q_{j+1})^{1-m} \cdot Q_{j+1} \right] \qquad (23)$$

where N is the total number of reservoirs, Q_0 is the discharge at upstream end, Q_N is the discharge at downstream end.

The term R_M, which appeared in Eq. (22) results from the internal fluxes between subsequent intervals – reservoirs. They are not equal one another and consequently the total flux at the internal nodes cannot be cancelled. Eq. (23) shows, that the Muskingum model written in non–conservative form, Eq. (13), preserves neither the mass conservation nor the momentum conservation law. This fact explains the mass balance error reported by Ponce and Yevjevich [6] and Tang, Knight and Samuels [8], which was noticed after introducing variable parameter K into the linear Muskingum equation. Such approach cannot be successful because it leads to the non–linear equation written in the non–conservative form. Consequently an extra term R_M is generated during integration.

In the same way the global conservation laws for the conservative forms of the non–linear Muskingum equation can be derived. For Eq. (16) one obtains:

$$\int_0^T ((Q_O(t))^{2-m} - (Q_N(t))^{2-m})\, dt = 0 \qquad (24)$$

Where as for Eq. (17) the global conservation law is as follows:

$$\int_0^T (Q_0(t) - Q_N(t))\, dt = 0 \qquad (25)$$

Since the functions $Q_0(t)$ and $Q_N(t)$ denote the hydrographs in the cross-sections $x = 0$ and $x = L$ respectively. Eq. (24) cannot preserve the mass conservation, whereas Eq. (25) represents the mass conservation principle since Eq. (25) coincides with Eq. (18). In this case the total volume of the flood wave inflowing by the upstream end ($x = 0$) should be equal to the total volume of water outflowing by the downstream end ($x = L$). Of course, the linear Muskingum model in form of Eq. (8) also represents mass conservation principles.

While solving both conservative forms of the Muskingum equation one can notice that one of them fulfill perfectly the law of conservation of the transported quantity independently of the accuracy of the numerical method, whereas the other one generates a balance error depending on the values of the weighting parameters involved by the numerical scheme. It suggests that this error is determined by the numerical errors generated by applied method of solution.

5 Accuracy Analyses

The process approximation of differential equation introduces numerical error into the solution. In the case of the linear Muskingum Eq. (8):

$$X\frac{dQ_{j-1}}{dt}+(1-X)\frac{dQ_j}{dt}=\frac{1}{K}\left(Q_{j-1}-Q_j\right) \tag{26}$$

this error can be estimated by means of the modified equation approach. To derive the modified equation all nodal values of Q in Eq. (26) must be replaced by Taylor's expansion around the point of approximation [4]. Including the terms of 2nd order, the following modified equation is obtained [7]:

$$\frac{\partial Q}{\partial t}+\frac{\Delta x}{K}\frac{\partial Q}{\partial x}=\nu_n\frac{\partial^2 Q}{\partial x^2} \tag{27}$$

where ν_n is the coefficient of numerical diffusion given by relation:

$$\nu_n=\left(\frac{1}{2}-X\right)\frac{\Delta x^2}{K} \tag{28}$$

The diffusive term in Eq. (27) causes in the numerical solution an artificial flood wave's attenuation. An additional numerical diffusion can be generated while integrating the ordinary differential Eq. (26) over time by the implicit method which can be expressed as:

$$f_{t+\Delta t}=f_t+\Delta t\left[\left(1-\theta\right)f_t'+\theta f_{t+\Delta t}'\right] \tag{29}$$

where f is a function, f' is a derivatives of function f, Δt is time step, θ is the weighting parameter ranging from 0.5 to 1.

Taking into account the integration over space and time, a numerical diffusion ν_n takes the form:

$$\nu_n=\left(\frac{1}{2}-X\right)\frac{\Delta x^2}{K}+\left(\theta-\frac{1}{2}\right)\frac{\Delta x^2}{K^2}\Delta t \tag{30}$$

Above relation is consistent with the form of coefficient ν_n resulting from accuracy analysis carried out for the approximation of the kinematic wave by the box scheme. In Eq. (30) additional diffusive term contains the coefficient ν_n, which depends on the numerical parameters only. Note that for X = 0.5 and θ = 0.5 numerical diffusion disappears, since one obtains ν_n = 0. Otherwise the scheme generates numerical diffusion, which causes the smoothing of solution.

The relations and conclusions presented for the linear equation can be useful to explain the properties of the non–linear equations of the Muskingum model as well. In the case of Eq. (16) its approximation by the method (29) leads to the following modified equation:

$$\frac{\partial Q}{\partial t}+\frac{\Delta x}{K(Q)}\frac{\partial Q}{\partial x}=\frac{\partial Q}{\partial x}\left(\nu_n\frac{\partial Q}{\partial x}\right) \tag{31}$$

where as for Eq. (17) the modified equation takes the form:

$$\frac{\partial Q^m}{\partial t}+\frac{\Delta x}{K(Q)}\frac{\partial Q^m}{\partial x}=\frac{\partial Q}{\partial x}\left(\nu_n\frac{\partial Q^m}{\partial x}\right) \tag{32}$$

where parameter *K(Q)* takes the form:

$$K(Q)=\Delta x\cdot\alpha\cdot m\cdot\left(Q_P\right)^{m-1} \tag{33}$$

In both equations the coefficient of numerical diffusion ν_n is described by the same formula (30), however in this case the coefficient ν_n is considered as variable. Eqs. (31) and (32) are combined with the numerical diffusivity ν_n and then these equations are rewritten in the following conservative forms:

$$\frac{\partial Q}{\partial t}+\frac{1}{\alpha m(2-m)}\frac{\partial Q^{2-m}}{\partial x}=\frac{\partial}{\partial x}\left(\frac{\nu_n(Q)\cdot K(Q)}{\Delta x}\cdot\frac{1}{\alpha m(2-m)}\frac{\partial Q^{2-m}}{\partial x}\right) \tag{34}$$

$$\alpha\frac{\partial Q^m}{\partial t}+\frac{\partial Q}{\partial x}=\frac{\partial}{\partial x}\left(\frac{\nu_n(Q)\cdot K(Q)}{\Delta x}\cdot\frac{\partial Q}{\partial x}\right) \tag{35}$$

The modified equations (34) and (35) correspond to the governing equations (16) and (17) respectively. According to the condition of consistency the modified equation must tend to the governing one if space interval tends to zero [7]. For $\Delta x \to 0$ Eq. (34) and (35) tend to the kinematic wave equation written in the conservation form (14) and (15) respectively. Equation (14) can be transformed directly to equation (15). However, in the case of the modified equations (34) and (35) such direct transformation is not possible. These equations are not equivalent because of the additional terms containing the derivatives of 2nd order. In order to demonstrate this difference Eq. (34) is reformed into the following equation:

$$\alpha\frac{\partial Q^m}{\partial t}+\frac{\partial Q}{\partial x}=\frac{\partial}{\partial x}\left(\frac{\nu_n(Q)\cdot K(Q)}{\Delta x}\cdot\frac{\partial Q}{\partial x}\right)+S_n \tag{36}$$

where:

$$S_n=\left(\left(\theta-\frac{1}{2}\right)\Delta t\frac{\Delta x}{K(Q)}+\left(\frac{1}{2}-X\right)\Delta x\right)\frac{1}{m}\frac{\partial Q^m}{\partial x}\frac{\partial Q^{1-m}}{\partial x} \tag{37}$$

A comparison of equations (35) and (36) indicates that in (36) an extra term has occurred. This additional source term S_n results from the artificial diffusion, which is generated in the solution by the applied method. Therefore, although both governing equations (16) and (17) are written in the conservative forms, their numerical solution must be different. The additional term S_n indicates that the mass balance error in the numerical solution of the Muskingum equation (16) is generated and an artificial increasing of the mass outflowing from channel will be observed. According to Eq. (37) the balance error depends on the values of the weighting parameters, mesh dimensions and the spatial derivatives of the flow discharge Q.

6 Numerical Tests

To investigate the difference between the various equations of the Muskingum model with respect to the mass conservation, some computational tests were carried out. The flow in a rectangular channel of width B = 50 m and of length 50 km is considered. Its bottom slope is s = 0.0005 whereas the Manning's coefficient is n = 0.025. The channel length was replaced by ten reservoirs (N=10) with length Δx = 5 km between cross-section j and j+1. The initial condition corresponds to the uniform steady flow for discharge q_0 with normal depth h_n. At the upstream end the following hydrograph was imposed:

$$q(t) = q_0 + \left(q_{\max} - q_0\right)\left(\frac{t}{t_{\max}}\right)^2 \exp\left(1 - \left(\frac{t}{t_{\max}}\right)^2\right) \tag{38}$$

where q_0 is the baseflow discharge of the inflow, $q_{\max}$ is a peak discharge of the inflow, $t_{\max}$ is time of the peak flow.

All forms of the Muskingum equation were applied for the same flood wave described by Eq. (38) with q_0 = 100 m^3/s, $q_{\max}$ = 1000 m^3/s, $t_{\max}$ = 5 h and T = 21 h. The numerical integration was carried out using the implicit method (29). The mass balance errors were calculated by Eq. (20) for various values of the weighting parameters θ and X. It was assumed that the time T is long enough, so that at the downstream end the flow reaches an initial steady state after the passing of the wave. The integrals in Eq. (19) were calculated using the trapezoidal method.

As it could be expected, the linear Muskingum equation (with K = const) perfectly satisfies mass conservation laws. The calculated mass balance errors ΔM were always equal to zero regardless of the values of the weighting parameters.

In Fig. 2 the results of the relation $\Delta M = f(\theta, X)$ for the conservation forms (16) and (17) are presented.

It can be seen that the numerical solution of Eq. (17) perfectly fulfils the law of mass conservation. Regardless of the values of weighting parameters the balance error is always equal to zero. However, in the case of equation (16) the balance error differs from zero and the conservation law of mass is not satisfied. The greatest value of mass error ΔM = –4.4% is obtained for X = 0 and θ = 1, whereas for X = 0.5 and θ = 0.5 this error is minimized (ΔM = –0.3%). This fact can be related to the numerical diffusion generated by the applied scheme for time integration and manner of semi-discretization in space. For X = 0.5 and θ = 0.5 the approximation is of 2nd order of accuracy and therefore it is non – dissipative. For X = 0 and θ = 1 the approximation is of 1st order of accuracy with regard to x as well as to t. Consequently it generates strongest numerical diffusion and introduces the balance error into the solution of Eq. (16).

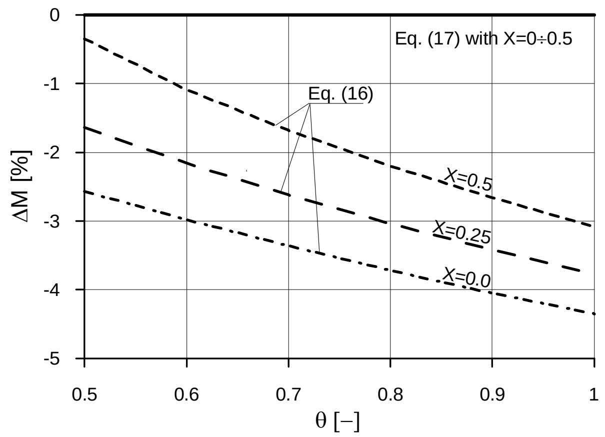

Fig. 2. Mass balance error for the Muskingum model written in the conservative forms for various values of the parameters θ and X (N = 10, Δx = 5 km, Δt = 0.5 h)

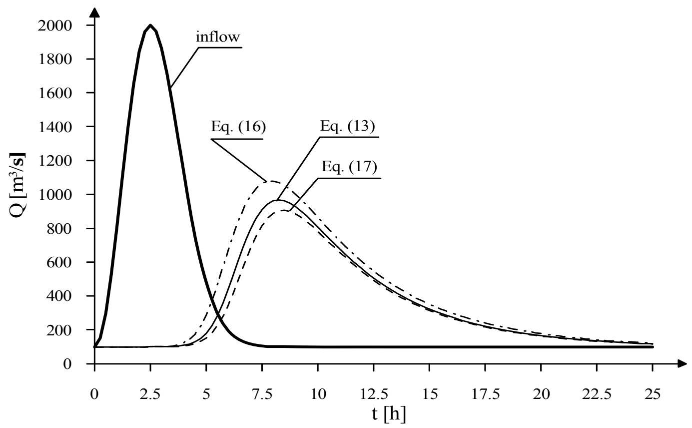

Fig. 3. Numerical solution of the non–linear Muskingum equation written in the conservation and non-conservation form for θ =1 and X = 0 (N =10, Δx = 10 km, Δt = 0.25 h)

The results of flood routing computed at the downstream end are shown in Fig. 3. The significant difference between outflows of the considered forms of the Muskingum equation is observed. The results were obtained for X=0 and θ =1. These values of weighting parameters introduce into the solution a strongest numerical diffusion, which is manifested by an excessive smoothing of the outflow wave. The numerical diffusion results in the mass balance error ΔM=-16.5 % for conservation form (16) and ΔM=-5% for non-conservation form (13). In the case of equation (17) written in the conservation form a mass error is not occurred.

7 Conclusions

An analysis of the mass conservation carried out for the Muskingum model allows us to find out, that the linear Muskingum equation satisfies mass conservation laws.

Using the kinematic character of the Muskingum model, various forms of the Muskingum equation can be derived directly from both continuity and steady uniform flow equation or by semi-discretization of the kinematic wave equation.

It was shown that a non-linear equation in non-conservation form of the Muskingum model does not possess a conservation property. The Muskingum equation can be written in two conservative forms, but only one form satisfies the mass conservation. An accuracy analysis carried out for conservation forms indicates that the balance error in the numerical solution results from the inadequate form of the considered equations and it is directly connected with the source term generated by the numerical diffusion. Consequently the mass error depends on the values of time step Δt, spatial interval Δx and the weighting parameters involved in the applied method of solution.The presented results suggest that for hydrological application the non–linear Muskingum model in the form of Eq. (17) seems to be most suitable, since it satisfies the mass conservation principle. It worth to add, that the results given by various forms of the Muskingum equation completely coincide with those given by the kinematic wave model [5]. This fact additionally confirms the kinematic origin of the Muskingum model.

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A21**

Organization and Technology of Construction of Dams in Function of Sustainable Environment Development

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Abstract. Dams have long been built for providing, flood protection, irrigation and water supply of population. While dams perform important functions, their impacts can be damaging to the environment. Within recent decades, environmental impacts of dams have been considered. Every dam causes some impact on the environment. Whether its purpose is hydroelectricity production, flood control or irrigation, a dam is a barrier across a river and it is a very concrete obstacle to the river's natural course and functions. The typical potential impacts associated with the development of dams include perils to water quality, neighboring biodiversity, nuisances and traffic generated by construction and operation of the dam, resettlement of people and visual and landscape impact of the dam. While wide variations occur from site to site, the environmental impacts of dams can generally fall within two categories: those due to the construction of the dam and existence of the reservoir and those due to the dam operation. Well prepared Project for organization and technology of construction for dam can create conditions for increasing positive and decreasing negative dam's construction impacts on environment. This paper presents environmental aspects that are used into Project for organization and technology of construction for dam making to ensure sustainable environment development in Republic of Macedonia.

Keywords: Organization of construction, technology of construction, dam, environment.

1 Introduction

The construction of the large dams is a complex and long-lasting process with a lot of interconnected activities in phases of preparatory, main and final civil and other works during the construction. The areas of dam profile and dam basin are usually with relatively friendly and favourable environmental conditions. In a present practice, beside the known positive impacts on the society development from one side, the possible negative impacts on the environment must be analyzed in details in order to insure its sustainable development.

Authors believe that the most important possible influences of the construction on the environment can be predicted during the preparation of the Design project for organization and technology of the construction. Because of that, the goal of this paper is to make an overview of possible influences and interaction of the dam construction with environmental aspects that should be analyzed in such Design Projects, in order to insure sustainable development of the environment in R. Macedonia thought some cases from practice.

2 Negative Impacts on the Environment Connected with Organization and Technology for Large Dam Construction

2.1. Types of Negative Impacts

Construction of the large dams is connected with whole system of complex works, and therefore the possible impact on the environment can be numerous.

The term large dams in fact are related to different type of structures, which means that for every different structure, some specific positive and negative influences shall be analyzed for each case. In general, the classification of the impacts, based on different criteria, can be presented in the Table 1.

Table 1. Classification of the impacts using different criteria

Criteria	Impact description
Level of impact	Low level Middle level High level
Time of action	Short term impact action Middle term impact action Long term impact action
Subject of impacts	On the humans Flora and fauna Soil, water, air, climate, Material goods, cultural heritage, landscape etc.
Types of Impact on the human	Sociological Psychological Health etc.

Most important impacts are present if a citizen’s dislocation is necessary, not only for the citizens that are dislocated from the reservoir area, but also for the people upstream and downstream of the reservoir area, as well as for the zone where the dislocated people will have new living spaces.

The properties problems are always present when people have property in the site area or in the near site surrounding, as well as for the population that may have limited access to certain material, cultural-historical and other goods. Those impacts refer to recreation and sport activities, land usage, usage of: roads, picnic places, recreation places etc. The impacts may have influence on people as some health problems because of the change of quality of soil, water, air etc.

Big part of the impacts connected to people is referred to the workers who are employed for execution of the construction work.

Area–construction work for large dams is mostly spread out on large area. That's why the impacts on the area are unavoidable.

There are two types of impacts on the area (3): (a) Visual and (b) Area impacts in the terms of noise, smells, topography, natural characteristics, climate, water, trees losses and vegetation etc.

Impacts on the **soil, water and air** mostly come from the activities related to the construction of structures. The most common impacts are: soil erosion, as a result of increased surface water flow or land movement; land compression, as a result of machines movement, landslide and landfall because of not enough good protection of the banks; pollution of surface and ground water from the leakage and from the used fuels or from the chemical leakage into the warehouses and workshops. Water pollution affects its quality and together with that comes the possibility of its usage on the construction site and possible bad influences on the ecosystems.

Impacts on the flora and fauna originate as a result of the vegetation cutting as well as from the quality change of the air, water and soil. All of that has influence on the disturbing of the ecological equilibrium, on the losses of soil and water residence of plants and animals, and on the residence dividing. The construction activities in some area may result in a change of the pattern and migration paths of certain fauna types. The land compaction from the machines can cause infiltration decreasing of the water into the soil, and some difficulties for the water to get into the roots of the plants. Dust deposition on the plant's leafs may cause lowering the process of the photosynthesis. The influences on the existing life world can occur because of the introduction of some new types of flora and fauna during the renewal of the soil and water residences after finishing the activities connected to the construction of the structure.

During the construction time there is a possibility for damaging the **material[1] and cultural-historical goods**, such as land losses and losses of the natural resources which have economical value, goods which are important for production, development, recreation and living. All of that can have negative influence on the cultural inheritance: settlements, monuments/structures which are valuable for their age, history, beauty and tradition.

Besides the already mentioned impacts, during the execution of the construction works, some combinations of the impacts can also occur as well as some other impacts that are not mentioned above.

2.2. Strategies for Decreasing of the Negative Environmental Impacts

The magnitude and the nature of the environmental impacts during the time of the dam construction depend on the sustainability of the dam location and its near environment, type of the construction works that will be executed as well as on the scheduled time for construction of the structures.

1 Material goods are: physical resources in the nature which have human or natural background (3)

Because of the fact that these structures and the impact factors are unique, it is not possible to determinate all the possible environmental impacts that are resulting from the construction works and to give measures for their reduction, for all types of dams. The concrete measures and the complete estimation of the environmental impacts depend on the materials, work methodology and technology of the construction, which are selected in the Project for organization and technology of construction, and depend on the work of the machines and mechanization used from the contractors. However, during the phase of designing of the Project for organization and technology of construction it is possible to identify the larger part of the potential negative environmental impacts. For reducing the above-mentioned negative impacts, in the project are used three generally adopted strategies: avoiding, reducing and remediation (3):

- Avoiding the possible environmental impacts during the execution of the activities related to the dam construction.

- Reducing is applied in the cases when there are impacts which can not be avoid, that's why it is recommended to adopt measures for reducing the already known or predicted impacts.

- Remediation is a strategy applicable for dealing with impacts that cannot be prevented to cause bad impacts. At big dams there are always intended impacts which are happening according to the Project for organization and technology of construction, after taking certain measures for their reducing. In order to mitigate the negative environmental impacts for impacts that can not be reduced during construction of large dams, some measures can be taken after finishing the dam construction.

With the Project for organization and technology of construction some more characteristic negative impacts can be identified and reduced in order to increase the positive environmental impacts. But, it has to be mentioned that there are always impacts that can not be completely estimated until the beginning of the realization of activities related to construction of the structure.

3 Negative Impacts and Recommendations for Getting Beyond Them

If we analyze the content of the Project for organization and technology of construction the conclusion will be that main reasons for problems, as negative environmental impacts, caused by this project may be:

- Selection and application of improper materials, during the period of designing and execution of construction works.
- Improper research of preliminary works.
- Selection and application of improper technology, organization and mechanization for construction of the structures during the preparatory, main and final works.
- Improper application of the relevant low regulative during designing and realization of the project for organization and technology of construction.

3.1. Selection and Application of Improper Materials During Designing and Executing of Construction Works

During the process of designing and execution of the construction works on dam, it is possible to select and apply materials that can be possible danger to human health or to the environment, because of their nature, quantity, state etc. Usually those are materials that are composed from dangerous substances. The Project for organization and technology of construction will make sure those kinds of materials not to be adopted.

3.2. Improper research of preliminary works

Results from the researches of preliminary works[2] are base for the Project for organization and technology of construction. Improper researches of preliminary works, as well as improper researches in terms of organization and technology of construction, will result in defining activities, measures and solutions that may have negative environmental impacts.

3.3. Selection and application of inappropriate technology, organization and mechanization for construction of the structures during the preparation, main and final works

Selection and application of inappropriate technology, organization and mechanization for construction of the structures during the preparation, main and final works may result as the following, more characteristic, possible sources of bad environmental impacts:

Structure location if the selected locations are in populated areas or in areas with high nature, culture-historical, material and other value. That is why we should avoid places that are declared to be national natural treasures or locations on which is possible to occur any impacts on the archeological findings, monuments, structures of cultural heritage and other structures. In case of vegetation cutting it's necessary to have an expert person who will be responsible for not harming the surrounding area of that structure.

The selected location must be justified in economical, engineering and ecological terms.

Construction settlement and structures from the construction site may produce communal sewage water and hard waste material. According to the Project for organization and technology of construction, and for avoiding them, the following measures should be taken:

- The structures for accommodation and work of the personnel on the construction site should have sanitary equipment for purification of sewage water, designed for the

[2] Preliminary works are: topographic, hydrological-hydraulics, geotechnical etc.

predicted capacity of accommodation, alimentation, and work for the maximal number of workers, parties, and technical stuff.

- The hard communal waste should be gathered in containers and should be regularly eliminated by the contracted services that will be responsible for keeping the hygiene and taking the waste material away from the construction site into previously assigned dump places.

The structures of the construction site[3] may cause negative environmental impacts during the time of construction or during the exploitation period. Some waste materials which will contaminate the environment may occur as a result of maintaining of mechanization and machine equipment (current maintenance, periodical controls, regular servicing and defects repairing). Everything should be washed out from the earth and dust layers that are usually greasy. The parts that should be repaired are also greasy and before the repairing process they should be degreased. Water under pressure, mixed together with some detergents, is used for washing out. Wasted materials may be bad for the environment, especially for the earth and water. The possible polluters are the concrete plants, concrete mixers, mixers and injection stations that during the cleaning process produce technical unclear water. As a possible environment polluter is the oil used by many machines for lubrication and that is periodically changed.

For reducing the possible negative influences on the environment, the Project for organization and technology of construction recommends the following measures:

- Washing and maintaining of the equipment, mechanization, and devices should be carried out on special places that are designated for that purpose. The unclean water from the washing and maintenance should be taken away in a sedimentation pond for sedimentation cleaning. On the conduit parts where vehicles and fuel are stored, separators for oil and greasy earth are adopted. Accumulation of wasted oil should be in special equipment – mostly steel barrels. The recommendation is to empty those barrels according to the authorized communal department which is responsible for safe dislocation out of the construction site, into recycle centers. The refined water outflow should be in the riverbed.

- Provided that the fuel and lubricator depot is not designed as a part of a construction site, the lubricator should be packed in cans and carried to the site in trucks, and fuel should be replenished with mobile plants (auto cisterns), into places that have channels for accumulating of the eventually fuel seepage. During the pouring off and refilling of the fuel the recommendation is to apply all the protective measures in order to prevent the possible fuel outflow, as well as measures for fire protection. Specially trained workers are responsible for the handling.

Activities connected to execution of the preparatory work may cause significant environmental changes that happen mostly because of the chosen work methods and technology. The most common impacts are: emission from the burning gases of ma-

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[3] concrete plants; asphalt plants; services workshops; plants; site petrol stations; grease storehouses; other depots and warehouses; laboratories for concrete, geotechnic, asphalt, material dumps etc.

chines, noise and vibrations during construction, radiation from the substation for distribution of electrical energy, dust, sewage water from the construction site etc.

For reducing the already mentioned impacts, in the Project for organization and technology of construction recommended are the following measures:

- During works in the riverbed, we must provide the biological water minimum in the river, for reducing the impacts of the water flora and fauna. Also, fish outlets must be provided.

- During designing and activities connected to execution of temporary roads we should keep on mind the minimal disturbance of the environmental appearance, and the disarrangements of the local and transit traffic compared to the situation before construction start, should be minimum. The temporary roads may have huge length and that is why we should use the possibility for multi purpose investment during the process of designing. After finishing the dam construction, roads can be used for: settlements connecting; exploitation of farmland and woodland; as roads that lead to picnic places, natural rare places, historical or archeological places etc. The temporary settlement, as well as the other structures from the construction site, if possible, should be designed in a way which will give the opportunity for their usage after the dam is completed, for different purposes: as host and tourist centers; picnic-recreation centers; social, health structures, weekend settlements etc.

- During work execution in the construction site and activities in phase of carrying out work for keeping the location in satisfactory state, some changes may occur in the soil and in geology. Possible impacts are: earth erosion, sliding and land sliding because of unsatisfactory protection of the river banks. Impacts can happen because of some excavations and earth movement and material dumping on inappropriate places. All of that can influence on the quality and quantity of surface and underground water flow, and can change the hydrological conditions. Also during the construction works there is dust and sewage water from the construction site.

The Project for organization and technology of construction gives recommendations for planning and for taking care of those excavations in order to keep the stability of the ground during construction, and to create conditions for stability during further use of the structure. Because of possible erosion and landslides, excavations, if needed, have to be secured which gives protection to people, mechanization, and construction works.

As a measure for dust reduction, which is coming from the separation, the most common case is to adopt sand colanders. For lowering the dust creation it is recommended spraying the roads with water, effective traffic management, as well as certain limitations concerning the time when the excavation work is executed, especially when it comes to mining.

Spare excavation material should be deposit on special places, chosen for that purpose. As an excavation material depot, the dead space of the accumulation can be used, but with appropriate previous preparations in order not to jeopardize the water quality in the following exploitation regime of the dam. The locations for deposited excavation material, during the dam construction, should be with temporary look, and after finishing the structure, they will get their final look, which is close to the basic natural look they used to have.

- During machines and equipment work, if there are some waste substances because of the machine oil leakage, the recommendation is that machine should stop its work until the repair. Good cleaning of the places with random leakages is recommendable in order to prevent the possible pollution of the soil, ground, and surface water. To prevent the unwanted consequences that are coming from the work and maintaining of the machines and equipment, there should be in time checks and maintaining.

- The noise, which during construction, is coming from the equipment, trucks, explosions, excavation etc, as well as the vibrations during construction, can cause temporary dislocation of some animal species from that location and from the location that is near the construction site. The noise can also cause health problems for the workers on the construction site and people who live in the site's neighborhood.

With the Project for organization and technology of construction machines and equipment that generate less noise should be adopt. The recommendations are proper maintaining and providing muffles for noise limiting. As a protection from too loud noise, workers should use protection. As measure that can help in reducing the noise impacts is limiting the civil activities during day when people are actually less vulnerable to noise.

- During the construction of the dam, there is also a hard waste material: excavation material, remaining from the earth works, asphalt remaining etc. The hard waste material should be taken away into previously arranged depot.

- During the dam construction there is a possibility of fire that can easily spread out to the near, mostly forest location. That is why with the Project for organization and technology of construction we can choose an appropriate system for fire protection.

- Besides the above-mentioned environmental impacts from the Project for organization and technology of construction there are other possible environmental impacts, as well as combinations made from the mentioned impacts.

With the Project for organization and technology of construction as a strategy for final works is remediation. After the structure is finished, there should be activities carried out for returning the terrain state on the level that is close to the starter one in order to improve the landscape, such as: removing the temporary structures and installations that were part of the construction site, removing the temporary depots from waste materials, forming the final terrain looks, renewing the water and soil quality, vegetation planting, settling new fauna etc.

From the above-mentioned the conclusion is that with the Project for organization and technology of construction, for executing of the works, are predicted organization, technology, methodology of construction which will eliminate or reduce the possible negative impacts on human's health and quality of the environment.

The project should be designed by personnel who have appropriate knowledge in environment in which the project will be carried out, appropriate experience in the certain type of the project-dam and by personnel who will be able to collaborate with other environmental experts in order to make a good Project for organization and technology of dam construction. We should take into consideration the previous experiences in realization of similar projects.

The dam contractor should fulfill the measures for environmental protection, as it is predict in the project, during the whole construction time. That will help in avoiding the additional change requests of the project, during its realization, which may lead to bigger financial investments compared to the planned ones, and will probably lead to environmental disturbance. Investments made for the environmental protection activities are secondary investments, taken into account in the process of calculating the total finances needed for the realization of the dam design (1).

3.3. Inappropriate Application of the Relevant Low Regulative During Designing and During Realization of the Project for Organization and Technology of Construction

The Project for organization and technology of construction can contribute in environmental protection only if all environmental impacts are considered, as well as impact's combinations, during the designing process, and only when they will be accomplished during construction. All this is possible if during the time of work execution the relevant low regulative is respected and if there is an efficient monitoring. In addition, after the work is finished, we should still respect the low regulative and there should be periodical monitoring, in order to foresee if all required effects are achieved.

4 Conclusions

Due to large scale, complexity and long term for realization of the works, during construction phases of large dams, the impacts on the environment cannot be avoided.

The possible impacts, level and details that should be analyzed in the Project for organization and technology for constructions project, are different for each structures. Having this in mind, the article presents only typical possible impacts on the environments, and the measures that are usually analyzed in the project documentation, in order to insure sustainable development of the affected areas. The measures should respect the relevant law framework, (3), (4), (5) and (6). They should be also in a correlation with Sectorial EIA Guidelines-dams, (2) which is also correlated to the Directive 85/337/EEZ (further improvements as 97/11/ES) for evaluation of the impacts on public/state and private projects on the environment (Law on the Environment), as well as the Directives of EU for evaluation if indirect and cumulative impacts and impact interactions.

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A28**

Semi-quantitative Risk Assessment of Water Structures

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Abstract. Recently, the use of methods of risk-based assessment of water structures (dams, levees, weirs, etc.) has become more frequent. During research into the applicability of risk analysis methods of water structures in the Czech Republic (CR), two approaches have been studied. For the prioritization of financial investment into reconstruction, refurbishment and the conceptual maintenance of water structures in a given portfolio, the Failure Modes Effects and Criticality Analysis (FMECA) can be effectively used. As a part of the research, formalized FMECA analysis was applied for the comparison of two dams (Sance and Mostiste) in the Czech Republic. Moreover, for the Mostiste dam the analysis was done for the state of the scheme before and after extensive reconstruction, carried out during the years 2005/2006. The results of the analysis suggest the high applicability of the method used.

Keywords: Water structures, risk analysis, FMEA, FMECA, dam

1 Introduction

Nowadays, the use of methods of risk assessment of water structures has become more frequent [1]. During the research into the applicability of water structures risk analysis methods in the Czech Republic (CR), two approaches have been studied.

The categorization system of selected water structures has been developed and used in the CR for safety assessment and emergency planning since the seventies. The category (I to IV) is derived from the potential damage and loss in the area downstream of the structure in the event of its total collapse.

For the prioritization of financial investments into reconstruction, refurbishment and the conceptual maintenance of water structures in a given portfolio, the Failure Modes Effects and Criticality Analysis (FMECA) can be effectively used [2]. As a part of the research, formalized FMECA analysis was applied at the comparison of two dams (Sance and Mostiste) in the Czech Republic. Moreover, for the Mostiste dam safety analysis was done for the state of the scheme before and after extensive reconstruction carried out during the years 2005/2006. The results of the analysis suggest the high applicability of the method used.

2 The FMEA and FMECA Methods

In FMEA analysis, firstly the hydro-scheme system must be defined and a complete list of all significant elements must be assembled. The following step is the search for potential failure modes (defects) and their effects on performance of the scheme. Qualitative analysis then summarizes elements of the system (dam, levees, reservoir, and catchment) and identifies potential defects or failures of these elements.

The FMECA process extends the first qualitative step of the analysis with a criticality evaluation, known as semi-quantitative analysis. Within the semi-quantitative assessment the potential for the failure of individual elements is evaluated together with the probability level for such a failure. Finally, the resulting partial probability and consequence scores are summarized in tabular form (e.g. spreadsheet). This method enables detailed assessment of individual element-defect seriousness and the total criticality of different water structures. In a similar way, comparison of various stages for one given water structure can be carried out, e.g. for a state before and after refurbishment, etc.

2.1 Qualitative Analysis

Qualitative analysis is the first step, when checklists of system elements and their possible defects are set up. Here, perfect knowledge about the structure, its components, their technical parameters and behaviour is essential. All available documentation and reports should be assembled and analyzed as a part of the qualitative analysis.

The basic question which has to be answered is: ‘*Which unfavourable events can be expected?’* Attention must be paid to all important parts of the system (elements) and processes which can be involved in creating a serious situation (danger) at the water structure. All processes and the states of elements are time dependent; the defects are assumed to be expected future incidents. During the identification of both elements and dangers, engineering ingenuity, experience and skill must be used.

In our analysis the main parts of the water structure (dam) at the principal classification level were identified as follows (Fig. 1):

1 - The catchment upstream of the dam
2 - Reservoir
3 - Dam body
4 - Sub-base of the dam
5 - Appurtenant works (A - bottom outlet, B - spillway)
6 - Tailwater channel

The main parts of the water structure at the secondary classification level have to be systematically sub-divided into elements at the analysed level, such as e.g.:

3 - Dam body

- 3A - Dam crest
- 3B - Upstream face
- 3C - Upstream shoulder
- 3D - Dam core... etc.

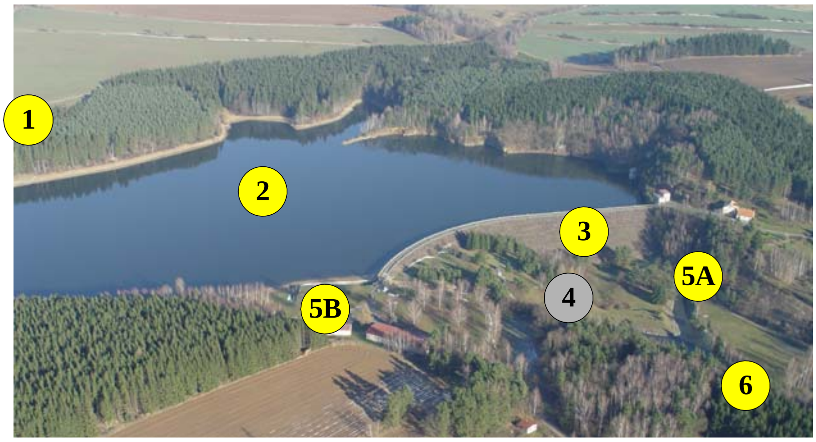

Fig. 1. Location of principle elements of the scheme - the Mostiste dam

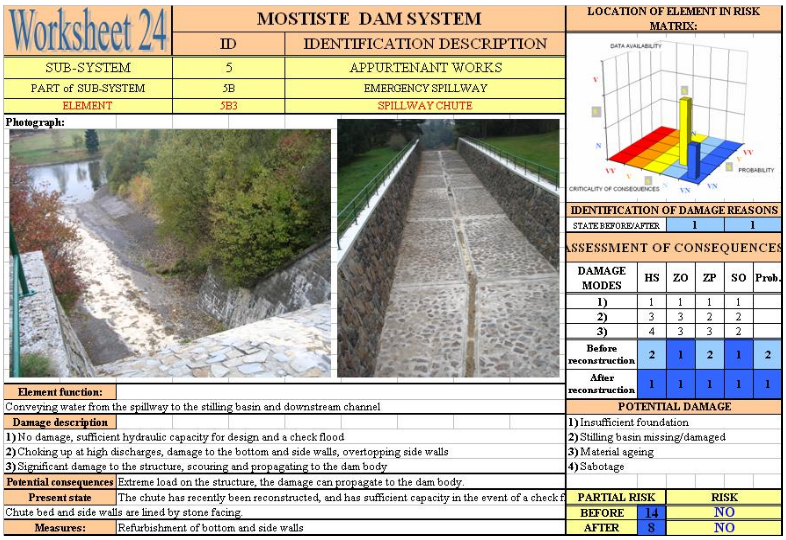

Fig. 2. An example of the Worksheet - No. 24 - Spillway chute

For each element, the potential partial failure modes were identified based on knowledge about the water structure, historical design and construction procedures, and the results of technical-safety surveillance and monitoring. One element can be exposed to more dangers. The reports from both the Czech and International Large Dams Committees are generally useful source of knowledge referring to particular incidents affecting structure parts and elements. A graphical database was set up comprising the verbal description of failure levels regarding the water structure assessed. The database was arranged in the form of "element worksheet", which contains a drawing of the element, its position in the system and the list of possible element defects (Fig. 2).

2.2 Semi-Quantitative Analysis

Risk is traditionally quantified as a product of the probability of an event and its consequences, e.g. losses in monetary units. At the same time, risk based methods for geotechnical, biological and environmental systems are recently in the early stage of development (2) when compared with those for the mechanical and electrotechnical sciences (1).

FMECA represents an alternative to complete quantitative assessment with comprehensive probabilistic analysis and reliable estimation of financial losses due to the failure of the scheme. The probability of defect occurrence and financial loss estimates in FMECA are replaced by criticality assessment, for which scoring by 5 (probability) x 5 (consequences) point scales was used. Criticality is related to individual element defects and their consequences for the system, respectively for the structure and the area below. Such an extension of FMEA is called Failure Modes Effects and Criticality Analysis (FMECA). In our analysis the "risk" (criticality) is evaluated for four consequence classes (subscript *j*):

1) Material losses (ML)
2) Human lives (HL)
3) Environmental losses (EL)
4) Societal consequences (SC)

The risk is expressed as follows:

Partial risk R_{ijk} for a given element (i), consequence class (j) and individual failure mode (k) is given by the formula:

$$R_{ijk} = BP_{ijk} + BC_{ijk} \tag{1}$$

where: BP_{ijk} is a probability score from 1 to 5 according to Table 1, BN_{ijk} is a consequence score from 1 to 5. Guidance for their estimation is given below in more detail. Firstly, the consequence score BC_{ijk} for consequence class j, given element i and individual failure mode k is estimated after careful analysis taking into account the location, function and importance of individual elements and the potential consequences in the event of element defects.

The guidelines for scoring are as follows:

Extreme consequences (E) – 5
- *Material losses* - catastrophic consequences leading to structure failure or destruction, extensive losses downstream of the scheme in thousands of millions EUR, international support and the help necessary.

- *Human lives* - high loss of human life, serious injuries to thousands of inhabitants, mental suffering, “aerial” rescue needed.

- *Environmental consequences* – damage to the natural environment, ecological breakdown, irreversible changes to ecosystems.

- *Societal losses* - extensive evacuation of inhabitants, loss of relatives, long-term interruption of the water, gas and electric current supply

High consequences (H) - 4

- *Material losses* - critical situation, extensive loss to property and structures in tens of millions EUR, governmental financial support necessary.

- *Human lives* - single mortalities, serious injuries to numerous inhabitants.

- *Environmental consequences* - environmental losses of regional extent, long-term regeneration required, ecosystems seriously disrupted.

- *Societal losses* - delivery of drinking water, gas and electric current interrupted (more than ten thousand inhabitants affected).

Medium consequences (M) - 3

- *Material losses* - serious losses of hundred thousand to million EUR, covered by local (regional) sources with governmental participation.

- *Human lives* - single serious injuries and numerous lightly injured.

- *Environmental consequences* - serious environmental losses of local character in the vicinity of the scheme, damage to several species.

- *Societal losses* - local problems with the delivery of products and services, including water, gas, etc.

Low consequences (L) - 2

- *Material losses* - less important incidents mostly to water structure, costing up to a hundred thousand EUR, covered by the structure owner.

- *Human lives* - single small injuries.

- *Environmental consequences* - insignificant damage to the environment, only a single species affected, regeneration in months.

- *Societal losses* - local societal problems, mostly involving difficulties with delivery to single households.

Negligible consequences (N) - 1

- *Material losses* - negligible losses affecting the water structure itself, property of third parties not affected.

- *Human lives* - no injuries at all.

- *Environmental consequences* - negligible local damage, regeneration in days.

- *Societal losses* - almost no consequences, delivery not influenced.

At the same time, the probability rating for element defect/failure has to be added. The basis for probability score estimation is given in Table 1.

The partial risk can be categorized and attached to a color scale according to the resulting score, Table 2.

Table 1. Guidelines for probability scoring

Probability	Probability class	More detailed verbal description
Expected (E)	5	The defect occurs annually
High (H)	4	High probability of occurrence; a return period of 2 to 20 years
Medium (M)	3	Occasional occurrence, once in 20 to 50 years
Low (L)	2	Not probable but a possible phenomenon with a return period of 50 to 100 years
Not expected (NE)	1	Phenomenon has never yet been observed; a very low probability with a frequency lower than once in 100 years

Table 2. Risk matrix with partial risk scaling

		PROBABILITY CLASS				
		Not Expected (NE)	Low (L)	Medium (M)	High (H)	Expected (E)
CONSEQUENCE CLASS	EXTREME (E)	6	7	8	9	10
	HIGH (H)	5	6	7	8	9
	MEDIUM (M)	4	5	6	7	8
	LOW (L)	3	4	5	6	7
	NEGLIGIBLE (N)	2	3	4	5	6

The partial risk R_{ij} for consequence class (j) and the given element (i) is obtained by summarizing the partial risks from (1) over all corresponding failure modes (k):

$$R_{ij} = \sum_{k=1}^{m} R_{ijk} \tag{2}$$

In a similar way, the partial risk R_i for the selected element (i) is obtained by summarizing the partial risks from (1) over all consequence classes (j) and corresponding failure modes (k):

$$R_i = \sum_{j=1}^{4} \sum_{k=1}^{m} R_{ijk} \tag{3}$$

Finally, total water structure risk is expressed as follows:

$$R = \sum_{i=1}^{n} \sum_{j=1}^{4} \sum_{k=1}^{m} R_{ijk} \tag{4}$$

where: n is the number of system elements at the analyzed level, and m is the number of failure modes found for individual elements. This summary has been organized using a spreadsheet. The organized application of Eq. (1) to (4) is shown in Table 3.

Table 3. Evaluation of partial and total water structure risk using a spreadsheet

SYSTEM	SUBSYSTEM_1	SUBSYSTEM_2	ELEMENT	ID	PROBABILITY	CONSEQENCES				PARTIAL RISK R_{ijk}				DATA AVAILABILITY
						MATERIAL LOSSES	HUMAN LIVES	ENVIRONMENTAL LOSSES	SOCIETAL CONSEQUENCES	MATERIAL LOSSES	HUMAN LIVES	ENVIRONMENTAL LOSSES	SOCIETAL CONSEQUENCES	
						ML	HL	EL	SC	ML	HL	EL	SC	
DAM MOSTISTE - before reconstruction	**Catchment area**	-	Catchment area upstream of dam	1	**1**	1	1	1	2	2	2	2	3	1
	Reservoir	-	Bottom of reservoir	2A	**1**	2	1	1	2	3	2	2	3	1
		-	Reservoir banks	2B	**2**	2	1	1	2	4	3	3	4	1
	Dam body	-	Dam crest	3A	**2**	3	1	2	2	5	3	4	4	2
		-	Upstream face	3B	**1**	3	2	2	2	4	3	3	3	1
		-	Upstream shoulder	3C	**2**	2	1	1	2	4	3	3	4	2
		-	Clayey core	3D	**4**	4	3	3	4	8	7	7	8	3
		-	Upstream filter	3E	**2**	4	4	3	4	6	6	5	6	3
		-	Downstream filter	3F	**2**	4	3	3	3	6	5	5	5	3
		-	Downstream shoulder	3G	**2**	4	3	3	3	6	5	5	5	2
		-	Downstream face	3H	**1**	2	1	1	1	3	2	2	2	1
		-	Approach tunnel	3I	**1**	1	1	1	1	2	2	2	2	1
		-	Grouting gallery	3J	**3**	3	2	2	2	6	5	5	5	2
		-	Right-bank abutment	3K	**3**	3	2	2	2	6	5	5	5	2
		-	Left-bank abutment	3L	**1**	2	1	1	2	3	2	2	3	1
	Dam sub-base	-	Sub-base rock	4A	**2**	3	1	1	2	5	3	3	4	3
		-	Grout curtain	4B	**2**	2	2	2	1	4	4	4	3	3
	Appurtenant works	Bottom Outlets	Inlet structure	5A1	**2**	2	1	1	2	4	3	3	4	2
			Pressure tunnels	5A2	**1**	2	1	1	1	3	2	2	2	2
			Outlet structure and valves	5A3	**2**	3	2	2	2	5	4	4	4	2
			Intake tower	5A4	**2**	2	1	1	2	4	3	3	4	1
		Emergency spillway	Emergency spillway	5B1	**2**	1	1	1	1	3	3	3	3	1
			Spillway	5B2	**1**	2	1	1	1	3	2	2	2	1
			Spillway chute	5B3	**2**	2	1	2	1	4	3	4	3	1
			Stilling basin	5B4	**3**	2	1	1	1	5	4	4	4	1
	Tailwater channel	-	Tailwater channel	6	**2**	1	1	1	1	3	3	3	3	1
									ΣR_{ij}	111	89	90	98	ΣR_j
														388

Table 4. Scoring of data availability and reliability

Data availability and reliability	Scoring	Verbal Description
Low (N)	3	Insufficient amount of data, low reliability, time consuming acquisition, some data not available.
Medium (S)	2	Data available, not complete, some of poor reliability and availability, technical documentation not complete, some measurements not performed.
High (V)	1	Amount and quality of data sufficient, good availability (historical records, manipulation regulations, technical documentation, measurements, etc.).

By summarizing the scores along the lines/columns in Table 3, partial risks of a higher order (see Eq. (2) and (3)), and the total risk score for the entire scheme according to formula (4) is obtained.

Additionally, the availability and reliability of data entering the analysis has to be assessed. Three-point scoring was used in the analysis according to Table 4.

The scoring for data availability and reliability is graphically expressed in the upper right corner in Fig. 2 and attached in the last column of Table 3. If high partial risk (depicted in yellow or red - scoring from 7 to 10 - in Table 2) is identified based on poor data, it is strongly recommended that additional data collection is carried out, along with further analysis of the corresponding element.

3 Conclusions

Recently, investments into water structures reconstruction and refurbishment are being planned and realized more or less on an intuitive basis often in cases of states of emergency. This was the case for Mostiste, Znojmo and some other schemes in the Czech Republic. The formalized assessment of risk factors using FMECA methodology offers a more objective basis for decisions within the portfolio of individual owners (e.g. River agencies). The semi-quantitative risk assessment of principal schemes provides a list, organised in order of urgency, of necessary refurbishments and risk reduction activities. Such an analysis also indicates the most risky elements of a scheme.

In this paper the method of FMECA assessment has been described and demonstrated at the Mostiste dam before its reconstruction in the year 2005. The total risk score was evaluated as 388 (Table 3). At the same time, analysis for the dam after reconstruction was carried out, with the resulting score of 303. Another case study was focused on the Sance dam, which has a similar dam type and appurtenant works (rockfill dam with thin clayey core, side spillway, bottom outlets in a diversion tunnel), the refurbishment of which is planned in 2009 with the aim of increasing dam safety. The score for the Sance dam was assessed to be 352, which is better (smaller) than that for the Mostiste dam before reconstruction, but worse than its score after reconstruction.

Acknowledgement

The content of this paper is part of a research Project NAZV QH81223.

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A30**

Inverse Integration of the Open Channel Flow Equations

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Abstract. The problem of inverse flood routing using the system of Saint Venant equations and the storage equation is discussed. In the first approach the hydrograph at the upstream end is determined by the solution of inverse problem for the hyperbolic equations. The inverse flood routing using the storage equation and the steady flow equation is solved as the standard initial value problem for the ordinary differential equation integrated with negative time step. It is shown that due to the numerical diffusion the simplified model is not able to reproduce the expected effect of the peak wave increasing towards the upstream end, whereas using the Saint Venant equations one can obtain the appropriate results.

Keywords: Open channels, unsteady flow, inverse flood routing

1 Inverse Integration of the Saint Venant Equations

To ensure the requested flow conditions over the considered section of the river, inverse flood routing should be carried out. Usually this problem is solved using the optimization technique. Sometimes, to this order the inverse problem for the system of Saint Venant equations (SVE) is formulated.

SVE can be written in the following conservative form:

$$B\frac{\partial h}{\partial t} + \frac{\partial Q}{\partial x} = q \tag{1}$$

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left(\frac{Q^2}{A}\right) + gA\frac{\partial h}{\partial x} + gAS = 0 \tag{2}$$

where: t is time, x is longitudinal coordinate, h is water stage, Q is flow discharge, A is cross-sectional area of flow, B is channel width at the water surface, g is gravitational acceleration, q is lateral inflow. Slope of energy line S is expressed by the Manning formula.

This system is usually integrated in a conventional way. It means that Eqs. (1, 2) are solved over a channel reach of length L for increasing time t, i.e. an integration is carried out in the following domain: $0 \leq x \leq L$ and $t \geq 0$ (Fig. 1.).

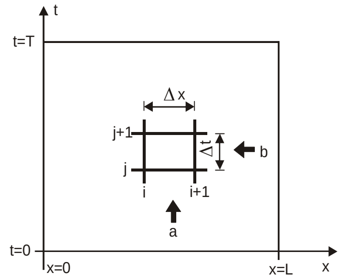

Fig. 1. Direction of integration of the Saint Venant equations while conventional (a) and inverse (b) solution

For SVE the inverse problem can be formulated as well. Eq. (1) and (2) can be integrated over the assumed time interval $\langle 0, T \rangle$, towards diminishing x (Fig. 1). This is possible on condition that the flow is subcritical. Such approach allows us to determine such a hydrograph at the upstream end that will ensure the required flow conditions at the downstream end. To this order the following auxiliary conditions should be imposed at the boundaries of the solution domain (Fig. 1):

- initial conditions: $Q(x,t) = Q_{in}(t)$ and $h(x,t) = h_{in}(t)$ for $x=L$ and $0 \leq t \leq T$;
- boundary conditions: $Q(t,x) = Q_0(x)$ or $h(t,x) = h_0(x)$ for $t=0$ and $0 \leq x \leq L$,
  $Q(t,x) = Q_T(x)$ or $h(t,x) = h_T(x)$ for $t=T$ and $0 \leq x \leq L$.

The inverse problem for SVE may be solved either with the method of characteristics [2] or with the method of finite difference using the box scheme [4, 5]. In this contribution we will apply the second approach.

To solve numerically the system of Eq. (1) and (2) a channel reach of length L is divided into M−1 intervals having length Δx, while the time ($0 \leq t \leq T$) is spaced with the time step Δt into N-1 intervals. Approximation of the differential equations is carried out using the box scheme, which works on the grid point presented in Fig. 2 [1].

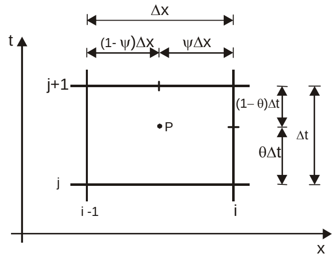

Fig. 2. Grid point applied in box scheme

$$Q = \frac{1}{n} S^{1/2} (R)^{2/3} A \tag{3}$$

The derivatives and functions approximated at point P transform the system (1, 2) to the following system of non-linear algebraic equations:

$$(1-\psi)\frac{h_i^{j+1}-h_i^j}{\Delta t} + \psi\frac{h_{i+1}^{j+1}-h_{i+1}^j}{\Delta t} + \\ + \frac{1}{B_p}\left((1-\theta)\frac{Q_{i+1}^j - Q_i^j}{\Delta x} + \theta\frac{Q_{i+1}^{j+1}-Q_i^{j+1}}{\Delta x}\right) = 0 \tag{4}$$

$$\begin{aligned}(1-\psi)\frac{Q_i^{j+1}-Q_i^j}{\Delta t} + \psi\frac{Q_{i+1}^{j+1}-Q_{i+1}^j}{\Delta t} + \frac{(1-\theta)}{\Delta x}\left(\frac{(Q_{i+1}^j)^2}{A_{i+1}^j} - \frac{(Q_i^j)^2}{A_i^j}\right) + \\ + \frac{\theta}{\Delta x}\left(\frac{(Q_{i+1}^{j+1})^2}{A_{i+1}^{j+1}} - \frac{(Q_i^{j+1})^2}{A_i^{j+1}}\right) + gA_P\left((1-\theta)\frac{h_{i+1}^j - h_i^j}{\Delta x} + \theta\frac{h_{i+1}^{j+1}-h_i^{j+1}}{\Delta x}\right) + \\ + \psi\left(\theta\left(\frac{g\cdot n\cdot|Q|Q}{R^{4/3}\cdot A}\right)_i^{j+1} + (1-\theta)\left(\frac{g\cdot n\cdot|Q|Q}{R^{4/3}\cdot A}\right)_i^j\right) + \\ + (1-\psi)\left(\theta\left(\frac{g\cdot n\cdot|Q|Q}{R^{4/3}\cdot A}\right)_{i+1}^{j+1} + (1-\theta)\left(\frac{g\cdot n\cdot|Q|Q}{R^{4/3}\cdot A}\right)_{i+1}^j\right)\end{aligned} \tag{5}$$

$$\text{with } B_p = \psi\left(\theta\, B_i^{j+1} + (1-\theta)\, B_i^j\right) + (1-\psi)\left(\theta\, B_{i+1}^{j+1} + (1-\theta) B_{i+1}^j\right) \tag{6}$$

where: i is index of cross section, j is index of time level, ψ and θ are the weightings parameters ranging from 0 to 1.

Eq. (4) and (5) can be written for each time step j=1,2,…, N-1 giving the system of algebraic equations. The unknowns are the nodal values of flow rate and water level in the preceding cross-section, i.e. Q_i^j and h_i^j. This system must be completed by the boundary conditions imposed at t=0 and t=T and next solved using the iterative method. Repeating this procedure for consecutive cross-sections i=M-1, M-2,…, 1, finally one obtains the required hydrographs at the upstream end.

2 Inverse Flood Routing Using the Storage Equation

For inverse flood routing the simplified approach based on the storage equation and the steady-state flow equation can be applied as well. In this approach instead of solving the inverse problem, an initial–value problem for the system of the ordinary differential non–linear equations, representing the cascade of N reservoirs, must be solved. This system is solved via its numerical integration over time with negative time step. It means that to determine the hydrograph at the upstream end, instead of solving relatively complex inverse problem for hyperbolic equations as previously, one can solve a rather simple initial – value problem.

Let us consider a channel reach of length L. Using N known cross-sections it can be divided into N −1 intervals of length $\Delta x = x_{i+1} - x_i$ (i=1, 2,…, N−1). In such a way one obtains a cascade of N−1 reservoirs, so that the outflow from the preceding reservoir is the inflow for the next one (Fig. 3).

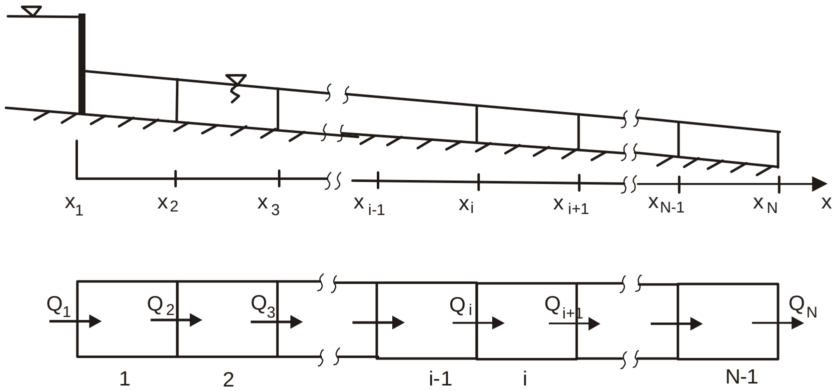

Fig. 3. Channel reach represented as a cascade of reservoirs [6]

The storage equation can be derived from the differential continuity equation (1). To this order it is integrated with regard to x over a channel's increment of length Δx. Consequently one obtains:

$$\frac{dV_i}{dt} = Q_i - Q_{i+1} \tag{7}$$

where: i is index of reservoir and cross − section, V_i is the total volume of water stored by the channel reach, Q is flow discharge through cross–section i, Q_{i+1} is the flow discharge through cross–section i+1.

To calculate the flow discharge between the neighbouring reservoirs the Manning formula is used. After eliminating V the equation (6) takes the following form:

$$\frac{d\bar{h}}{dt} = \frac{1}{F_i}\left(Q_i - Q_{i+1}\right) \tag{8}$$

where: F_i is the area of reservoir's surface depending on the water stage. The mean level of the water surface above the datum $\bar{h}$ can be expressed using the linear interpolation between the nodal values of h as follows:

$$\bar{h} = \psi h_i + (1-\psi) h_{i+1} \tag{9}$$

where: ψ is a weighting parameter ranging from 0 to 1. Finally Eq. (7) takes the following form:

$$\psi \frac{dh_i}{dt} + (1-\psi)\frac{dh_{i+1}}{dt} = \frac{1}{F_i}\left(Q_i - Q_{i+1}\right) \tag{10}$$

Similar equations can be written for each reservoir (for i = 1, 2, … , N−1). Consequently a system of ordinary differential equations is obtained. Its solution is formulated as an initial–value problem. Knowing the initial condition $h_i(t=0)=h_{i,in}$ where $h_{i,in}$ (i =1,2,. …,N−1) is given, the functions $h_i(t)$ have to be calculated for $0 \le t \le T$. To this order the equation should be numerically integrated using for example the following general two-level scheme:

$$y_{j+1} = y_j + \Delta t \left(\theta\, y'_j + (1-\theta)\, y'_{j+1}\right) \tag{11}$$

where: j is index of time level, θ is weighting parameter ranging from 0 to 1. The applied method of integration depends on the value of the weighting parameter θ [3]. Application of Eq. (10) to Eq. (9) yields the following formula:

$$\psi \frac{h_i^{j+1} - h_i^j}{\Delta t} + (1-\psi)\frac{h_{i+1}^{j+1} - h_{i+1}^j}{\Delta t} = \frac{1-\theta}{F_i^j}\left(Q_i^j - Q_{i+1}^j\right) + \frac{\theta}{F_i^{j+1}}\left(Q_i^{j+1} - Q_{i+1}^{j+1}\right), \text{ for } i = 1, 2, ..., N\text{-}1 \tag{12}$$

The flow discharge is given by the Manning formula:

$$Q_i^j = \frac{1}{n} S^{1/2} (R_i^j)^{2/3} A_i^j \tag{13}$$

While solving the inverse flood routing the required hydrograph h_N (t) is imposed (Fig. 4) at the downstream end (cross-section N). Moreover, one can assume that at the end of the considered time period T steady flow was reached. It means that the condition $h_i\,(t=T)=h_{i,in}$ for i=1, 2, …, N can be imposed. Then, using Eq. (11) with a negative time step Δt one obtains the hydrograph h_0 (t). In such a way the inverse routing is performed as a solution of the initial value problem.

In Eq. (11) only the water stage h_i^j is unknown. Since this equation is non-linear, it must be solved using an iterative method.

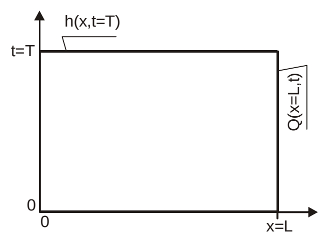

Fig. 4. Auxiliary conditions required for inverse routing using the storage equation

3 Numerical Tests

The presented approaches were applied for inverse flood routing in arbitrarily assumed channel. The trapezoidal channel of length of L=100.00 km with bed width b=20 m, slope side m=1.5, bed slope S=0.0005 is assumed. The space intervals are uniform, equal to Δx=400 m.

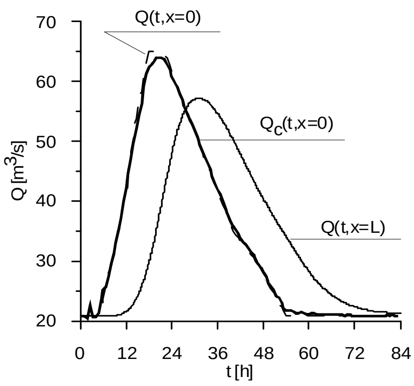

Fig. 5. Computed Q_c(t, x=0) and expected (Q(t, x=0) hydrographs using inverse solution of the Saint Venant equations

It is assumed that at first the conventional problem is solved and next the hydrograph computed at x=L is imposed as the initial condition for the inverse problems. The condition imposed at the upstream end for the conventional solution is shown in Fig. 5 as the function Q (t, x=0), whereas the hydrograph calculated at the downstream end via the solution of the SVE is denoted as Q (t, x=L). In the next step this hydrograph is imposed as the initial condition for the inverse routing. Thus one can expect that the result of inverse routing Q_c (t, x=0) should be similar to the function Q(t, x=0). Indeed, we can see a good agreement between these functions.

In the second example the storage equation is applied. For the same data as assumed previously, the inverse routing provided the results, which differ significantly from the expected one. In Fig. 6 we can observe that the storage equation does not reproduce properly the entering wave. Its peak is similar to the peak of the wave imposed at the downstream end. What more, it is even smaller than at x=L.

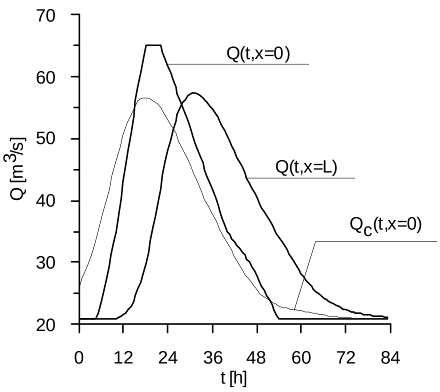

Fig. 6. Computed $Q_c(t, x=0)$ and expected ($Q(t, x=0)$ hydrographs using inverse solution of the storage equation

This seemingly surprising result is a consequence of the nature of applied model. One should be remember that the storage equation (1) with the Manning formula represent the kinematic wave model, which is capable to translate the flood wave along channel's axis only. Attenuation of the wave is caused by the numerical diffusion. In fact, the applied model in the form of Eq. (6) and (12) coincides with the kinematic wave model. These results suggest that the storage equation rather should not be applied for the inverse flood routing. This equation can be useful when the lateral inflow along channel's axis is significant. In such a case an appreciable inflow ensures the effect of increasing of the wave's peak and the numerical diffusion becomes less important. Illustration of such a case is presented in Fig. 7.

It is important to note that the stability of inverse routing for both models is ensured for $0 \leq \theta \leq 0.5$ and $1 \geq \theta \geq 0.5$. These conditions are inverted with respect to the ones valid for the conventional solution [5]. All applied models fulfill perfectly the law of mass conservation.

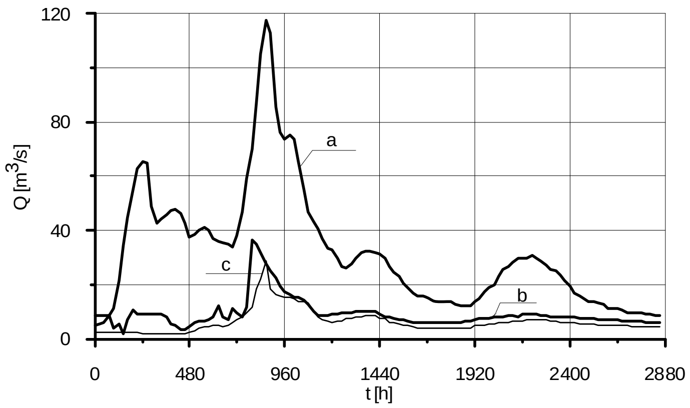

Fig. 7. Results of the inverse flood wave routing between Bondary and Siemianówka gauge stations (Upper Narew River-Poland) for period 07.03 – 04.07.1994 (a – imposed at the downstream end, b – calculated with the storage equation, c – observed) [6]

4 Conclusions

Two approaches to solve the problem of inverse flood routing were examined. The results obtained for the hypothetical flood wave allowed us to recognize general properties of the proposed methods of solution. It appeared that only the system of Saint Venant equations ensures both expected effects i.e. increasing of the wave peak and increasing of wave's steepness towards the upstream end. The storage equation is not capable to reproduce these effects.

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A32**

Experimental Study on Flow Structure in Strongly Curved Open Channel 90-degree Bends

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Abstract. This paper reports on a laboratory study that was performed on an experimental model in Ferdowsi University of Mashhad. This study examines flow structure for the control of erosion in strongly curved open channel bends. Lateral momentum and secondary flow induced in bends, causes maximum velocity transfer from center line, super elevation, which results in erosion in outer bank and depositing sediment in inner bank. Experiments were conducted in a flume, 6 m long, 0.4 m wide, 0.45 m high and 0.6 m radius of center line of bend, made up of compacted plastic materials. Study has been done at 4.5, 6, 9, 12, 15 cm depths of flow. The results showed that the transverse slope of water surface in bends is not linear. Results aim to predict depth of flow, position of maximum velocity and the place which must be protected from erosion in strongly curved bends.

Keywords: Open channel, bend, flow structure, velocity, separation effect

1 Introduction

So far, many researches have been done to explain the stage of flow in bend at open channels and rivers. This subject has more importance when we find out that the complex structure of flow and great turbulence existing in bends causes characteristics such as erosion, deposit, super elevation; score of bed, meander of river and secondary current are arisen. Researchers believe that secondary currents in bends that is created due to eccentric forces and pressure gradient are important factors in changing the above characteristics.

Lien et al. (1999) said “James Thomson (1876) was one of the pioneers in investigating velocity component in bends. He observed helical flow at bend boundaries by adding pigments to water” [1].

The other pioneers such as Yen (1942) [2], Mockmore (1943) [3], Shukry (1950) [4] and Rozovski (1961) [5] were the researches that studied flow at the bends.
Odgaard (1989) found out that the velocity and depth of flow in the axis of curve in the channel was constant and the change of the surface of water was linear reasonably [6].

Rodi (1978) [7], Shimizu (1989) [8], Yang G lai (2003) [6] represented the modeling of secondary flow.

Rodi (1987) illustrated the flow in strongly curved with 180º in turbulence conditional by using of the k-ε numerical model. Resultant surveys presented that flow model is affected by longitude pressure gradient [7].

With respect to the importance of this issue this present survey is focused on 90º strongly curve and distribution of velocity and change of depth is studied.

When flow encounters bend, the water surface increases on outer bank and decreases on inner bank because of the centrifugal force. As a result of continuity of flow and subcritical flow, the velocity decreases near the external wall, and it increases near the internal wall. Changing of water slope, presence of pressure gradient, formation of secondary flow and other characteristics of flow in bends complicate the flow pattern in curved open channel. Object of this study is examining of flow structure in strongly curved open channel 90-degree bends.

2 Illustration of Laboratorial Model

Laboratorial studies on existing setup in hydraulic laboratory at Ferdowsi University of Mashhad. The implemented details and characteristics at set up are shown in Figs. 1 and 2.

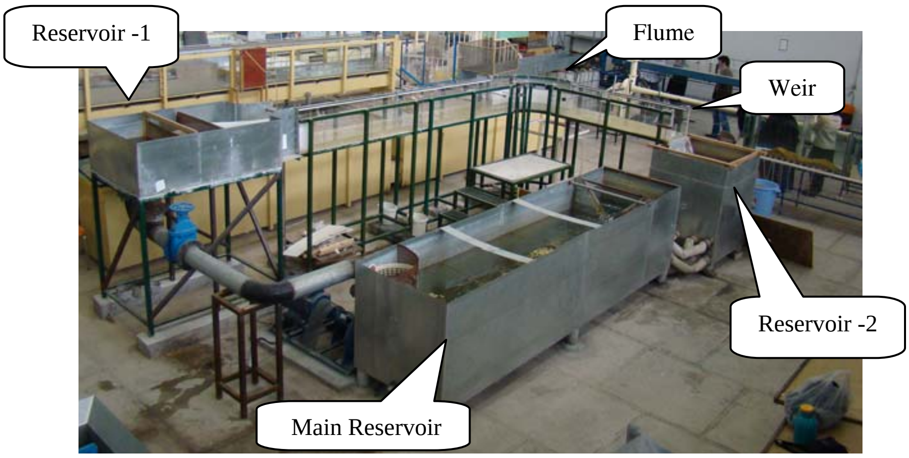

Fig. 1. Scheme of laboratorial model

The section of channel in dimensions 40.3*40.3 cm and beds and walls at channel are made of Plexiglas. At the channel entry, there is an entrance iron reservoir in dimension 150*100 cm and the height 50 cm that conveys the pumped water into the channel in order to uniform the flow in entrance it is used from 7 screen layers.

The sector of setup with a central angle 90º is a strongly curved bend $(R_c/b = 1.5)$ where R_c is the radius of curve and b is width of channel. At end of bend exists a straight channel that is long 180 cm. The aim of creation of this channel is to prevent turbulences arisen from the weir at the end of channel. After passing the top, the weir flow pours into the reservoir (Reservoir 2). The dimension of this reservoir is 1x1x1.25 meter. That it connected to the original reservoir by 11 flexible pipes with internal diameter 10 cm. original reservoir is divided in to two parts. Between two parts of reservoir a sharp edge triangle shaped weir is mounted. The discharge is measured by this weir.

In the second sector of the reservoir, the flow is conveyed to entrance reservoir by centrifuging pump.

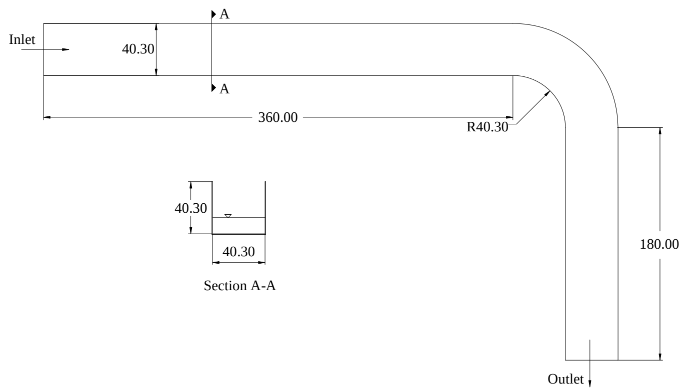

Fig. 2. Characteristics of geometrical set up

Just after the pump a gate valve with internal diameter 15 cm controls the discharge of flow.

In order to read tangential velocity, a one dimensional velocity meter is used.

To measure the depth, a verniet ruler is used that measures the width and height of channel with respectively 1 and 0.1 mm accuracy. In present survey, the experiments are performed on 5 discharges including 4.96, 6.7, 13.55, 19.36, 25.27 liter per second.

3 The Situation of Section

In order to perform these studies, quantities of velocity and depth in different sections in channel was measured. The situation of these sections contain 40 cm before the bend, 40 and 80 cm after the bend, the beginning and the end of the bend and the sections at the central angles 22.5º, 45º, 67.5º in the bend.

For every layer of these sections in 13 points, tangential velocities were measured. According to the catalogue of velocity meter the center of velocity meter in interval of boundary is equal to 1.5 cm, therefore, a distance of 2 cm from walls and 1.5 to 3 cm from top and bottom was determined. The characteristic of flow depth at downstream of bend and collected layers from surface of water when discharge was changed are represented in Table 1.

Table 1. Distance of collected layers from surface of water

Discharge (l/s)	4.96	6.70	13.55	19.36	25.27
Depth in downstream (cm)	4.5	6	9	12	15
Collected layers from surface	1.5	2	2.25	3	3
of water (cm)	3	4	4.5	6	3
			6.75	9	9
					12

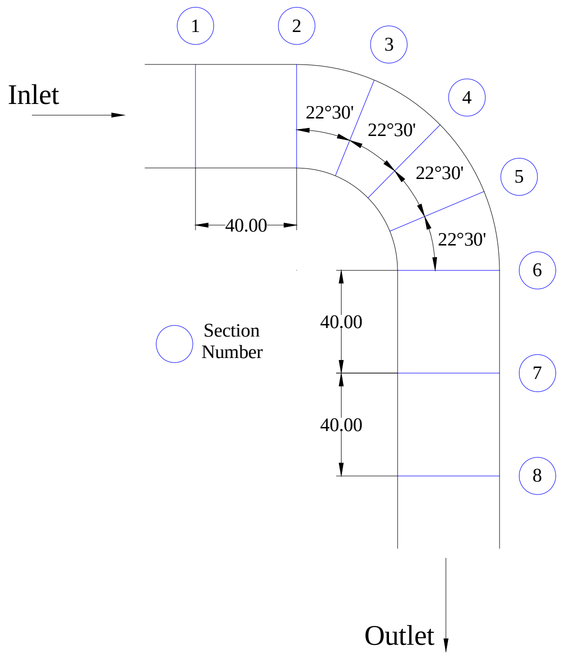

Fig. 3. Situation of section that is studied

4 Experimental Results

Tests are conducted in five discharges, but result of one of those is mentioned here. The discharge that is selected to discuss is 19.36 l/s.

Others are discussed on the analyses section. Figs. 1–3 show velocity on sections in bends (0°, 22.5°, 45°, 67.5°, and 90°) and in three layers (3, 6 and 9 cm from the surface of water).

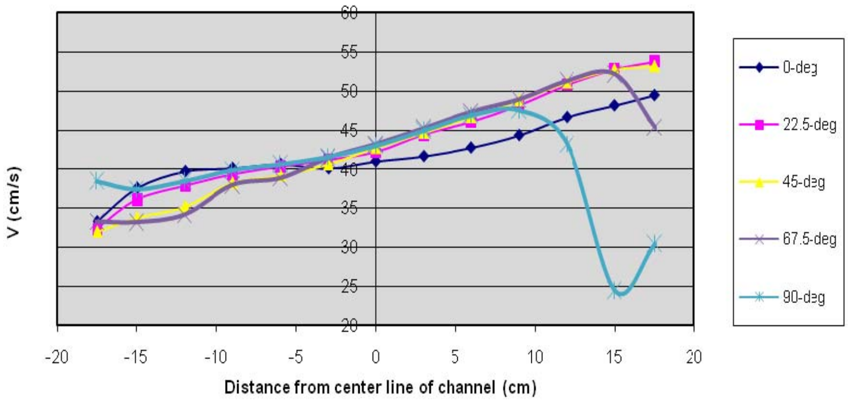

Fig. 1. Main velocity on in bend sections in upper layer (3 cm from the water surface)

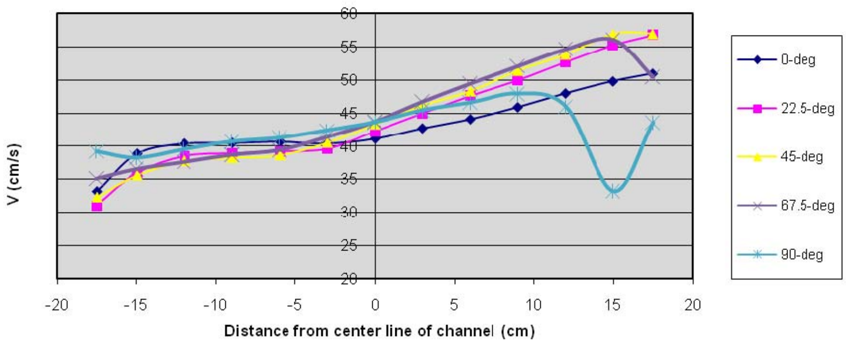

Fig. 2. Main velocity on in bend sections in middle layer (6 cm from the water surface)

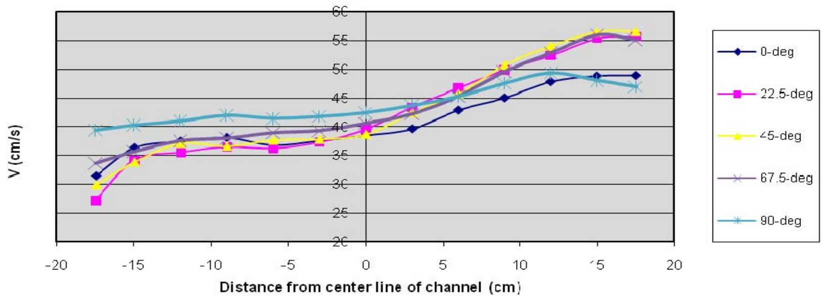

Fig. 3. Main velocity on in bend sections in lower Layer (9 cm from the water surface)

It is shown in these figures that the velocity increases in all layers from the internal wall to the external wall. What is interesting is the intense decrease of the velocity on 90-degree section in two layers (3 and 6 cm). This is like the separation effect.
The velocity of out of bend sections (40 cm before bend, 40 and 80 cm after bend) is illustrated in Figs. 4–6.

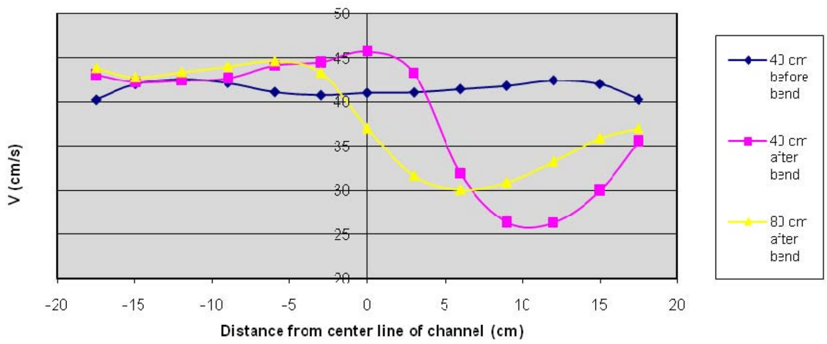

Fig. 4. Main velocity on out of bend sections in upper layer (3 cm from the water surface)

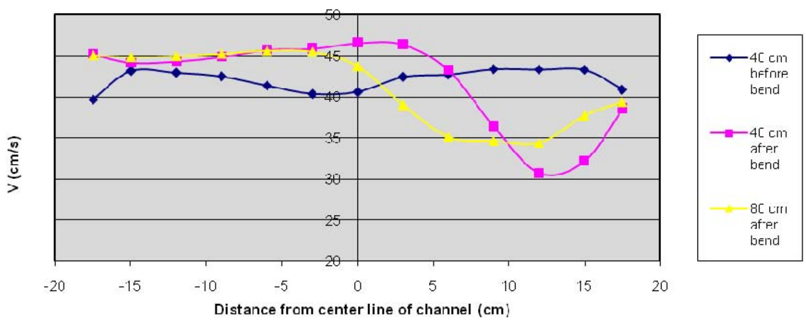

Fig. 5. Main velocity on out of bend sections in middle layer (6 cm from the water surface)

The longitudinal velocity on first section is uniform in all layers, but it is intensely decreased near the internal wall on seventh and eighth sections in two layers (3 and 6 cm). After the bend, the profile of velocity changes to become uniform. In other words, more distance from the bend, the velocity is more uniform.

Figs. 7-8 show the depth-averaged velocity for in the bend and out of the bend sections respectively.

As it is shown in Fig. 7, depth-averaged velocity increased from the external wall to the internal wall. In addition, this velocity intensely decreases on 90-degree section near the internal wall.

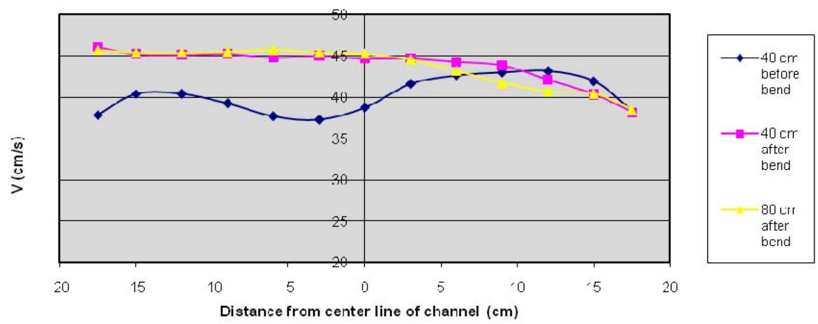

Fig. 6. Main velocity on out of bend sections in lower layer (9 cm from the water surface)

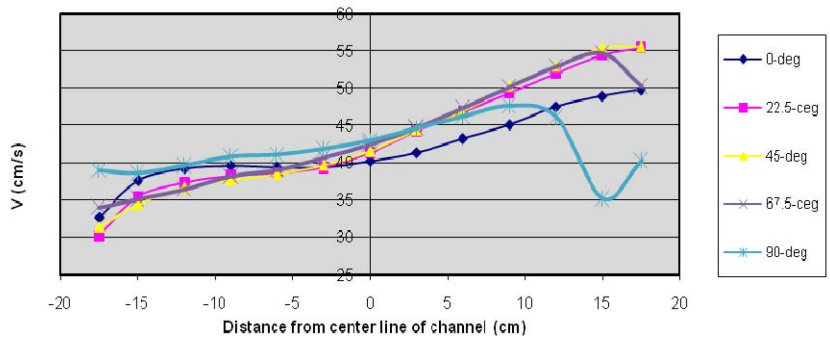

Fig. 7. Depth-averaged velocity on in bend sections

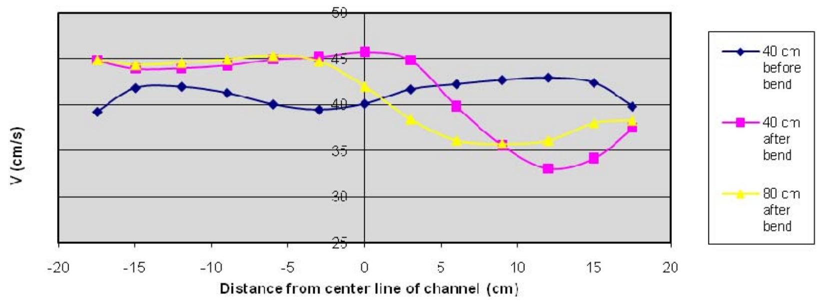

Fig. 8. Depth-averaged velocity on out of bend sections

It is clear in Fig. 8 that on the first section, depth-averaged velocity profile is uniform widthwise, but on two sections after the bend there is intense decrease of this velocity in the internal half of channel. This decrease disappears by going far from the bend.

Maximum of depth-averaged velocity and its location are obtained by the test on all sections. Its location is displayed in Fig. 9.

Fig. 9 shows that maximum velocity moves from about center line of channel in the first section to near the internal wall in the middle of bend (45 degree), and after this section it moves to the external wall.

Water depth is measured on all sections, and it is presented in Figs. 10-11 in order for in bend sections and out of bend sections.

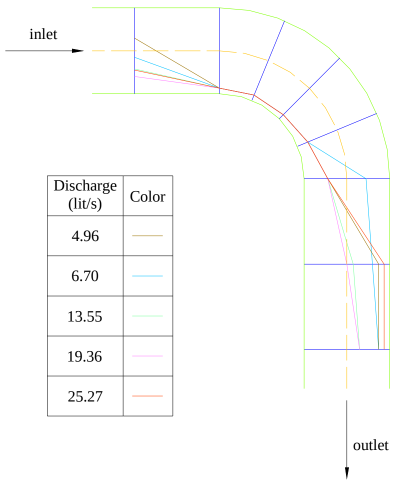

Fig. 9. Location of maximum of depth-averaged velocity along the channel on different discharges

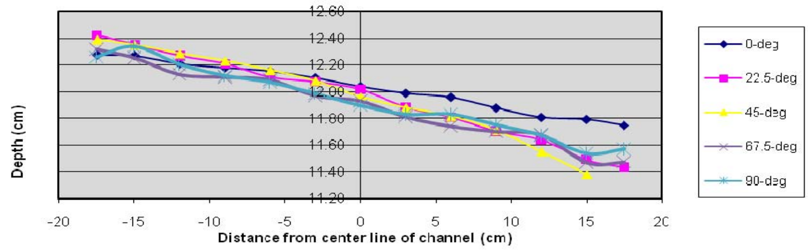

Fig. 10. Water depth on in bend sections

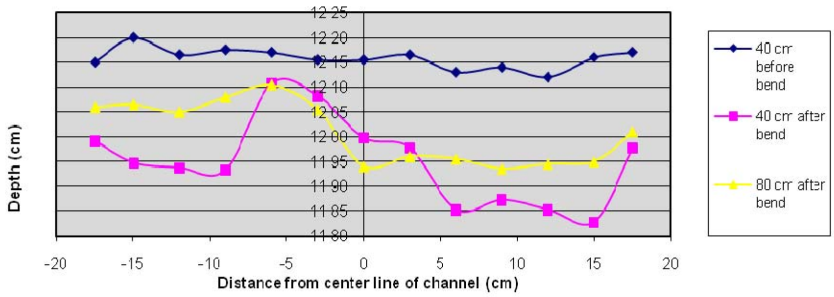

Fig. 11. Water depth on out of bend sections

As it is obvious in Fig. 10, water depth on in bend sections increases near the external wall, and decreases near the internal wall. Moreover transversal slope of surface increases from the first section to the middle of bend, and decreases after this section.

Fig. 11 shows that the slope is approximately zero on the first section. The slope decreases after the bend to become zero again. Table 2 indicates the quantity of decrease or increase of water depth near the internal and external walls.

Table 2. Decrease or increase of water depth near the internal and external walls in comparison with center line water depth on in bend sections

Section position	Decrease on Internal wall (cm)	Increase on external wall (cm)
0°	0.29	0.24
22.5°	0.58	0.41
45°	0.58	0.43
67.5°	0.46	0.39
90°	0.33	0.37

The trend of water depth along the channel is shown in Fig. 12.

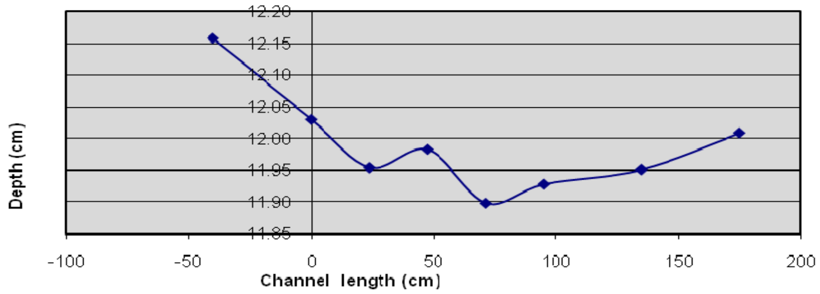

Fig. 12. Transversal-averaged water depth along the channel

It is visible in Fig. 12 that water depth before the bend is higher than it is on out flow. It decreases along the bend to 45-degree section, and after that it increases to reach 12 cm at final section.

5 Discussion

5.1 Maximum Velocity Trend

As it can be seen in Fig. 9 the maximum velocity in all discharges is about the center line of channel before the bend on the first section. Then, it moves to the internal wall. After 45-degree section, it changes its trend to the external wall. It reaches near the external wall on the final section. This is caused by the encounter of the secondary flow and longitude pressure gradient. The slope of surface is caused by the centrifugal force, and it is increases along the bend to the middle of the bend. Therefore, the slope generates transversal pressure gradient, and this gradient creates a mass flow to the internal wall. At the end of bend, transversal pressure gradient decreases because the water surface slope decreases. As a result, centrifugal force and tangent velocity cause the maximum velocity to move from the internal wall to the external wall.

5.2 Decrease and Increase of Maximum Velocity

For all discharges, maximum velocity increases from the first section to the middle of bend. This increase is about 22% for low discharges, 30% for mid discharge and 23% for high discharges. The reason the maximum velocity changes is that water depth decreases near the internal wall in this part of the bend, therefore, the maximum velocity increases. In comparison, after the middle of the bend, the water surface slope starts to decrease, and the secondary flow keeps away the maximum velocity from the internal wall. As a result, the maximum velocity decreases.

5.3 Center Line Velocity

By review of Figs. 1–6, it appears that on the first section and all in bend sections, the center line velocity is higher in the middle layer (6 cm) than it is in the upper layer (3 cm) and the lower layer (9 cm). In contrast, on the two sections after the bend, this velocity is highest in the lower layer.

5.4 The Separation Effect

In all discharges, the velocity intensely decreases on the final section of bend near the internal wall. This matter explains the tendency of flow to separate. This phenomenon occurs in middle and upper layers.

5.5 Water Depth Changes

Average water depth is higher before the bend than it is in other sections and in uniform flow. This results in losses of energy along the bend. Since there are energy losses in the bend, water depth increases before the bend to provide flow energy to pass the bend. In fact, the M1 profile is created in the channel before the bend that the flow depth increases there.

6 Conclusions

This experimental study helps to identify flow structure in strongly curved open channel 90-degree bends. By review resultant of all discharges, the most important matters that must be considered in studying of this type of bends are listed below.

First, the water slope is observed before the bend, and it lasted after the bend. In other words, the effect of bend on water slope is not only in the bend but also before and after the bend.

Then, the velocity pattern in strongly curved bends is fixed. Maximum velocity in bends is near the internal wall, and the minimum velocity is near the external wall. After the bend, the pattern is reversed. It means that after the bend the velocity near the internal wall is the minimum, and the velocity near the external wall is the maximum.

Finally, the separation effect occurs in all the discharges, but it is more intense in high discharges than it is in low discharges. This effect is observed on 90-degree section between the water surface and the middle of depth. On the bottom, because of overcoming of hydrostatic pressure, the separation effect is not observed.

Acknowledgment

Authors hereby express their thanks for the support they received from directors of the hydraulics laboratory of Faculty of Engineering of Ferdowsi University of Mashhad, where this study was conducted. In addition, we would like to thank Dr. J. Abrishami, professor of Hydraulic Engineering for the fruitful discussions during doing this research.

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A36**

Spatial and Temporal Averaging of Velocity Profiles in Natural Watercourses

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Abstract. Introduction of ADCP devices resulted in great progress in hydrometry of open watercourses. Simple use and fast measuring allow collecting of a large number of data on channel geometry, distribution of velocities and the discharge. By conventional method of measuring in the watercourse, using the hydrometric wing, the point velocity is obtained by temporal averaging of the velocity component in the direction of the device. ADCP at the measuring point gives the ¨momentary¨ value of the spatial velocity vector. In both procedures, the discharge is obtained by integrating the product of multiplying of unit area and the corresponding vertical velocity component over the profile. The velocity profile over depth, obtained by the conventional method, has the shape of an exponential function. On the other hand, the velocity profile over depth, taken by ADCP, assumes the shape of a random function, with values that often have the negative sign. Spatial averaging of velocity profiles from several neighboring verticals, results in a smoother shape of the function, which becomes closer to the shape resulting from temporal averaging in a single vertical. This paper analyses spatially and temporally averaged velocity profiles in a number of discharge profiles of the Drava River. The conclusion is that it is possible, by spatial averaging of velocity profiles in neighboring verticals, to get sufficiently precise picture of the profile of temporarily averaged velocities over depth.

Keywords: Natural watercourses, turbulent flow, measurements, velocity profiles, logarithmic low, parabolic method

1 Introduction

The subject of the paper is the analysis of the flow field in the cut-off at Nemetin, in the meander of the Drava River downstream from the town of Osijek, at river km

12+374 to 14+210. Meander cute cut-off was constructed in 1986 in order to shift the flow beside the new port Osijek, to provide the conditions for water, ice and sediment flow, and to secure the Drava navigation waterway. After excavation of the cut-off, the old Drava river channel was planned to serve as the basin port Nova Luka Osijek, river km 12+640 to km 12+900. In November 2005 the old Drava river channel was partitioned off, directing the main stream into the cut-off. The weir on the old river channel was constructed over its entire width. The weir crest is at the mean water level, i.e. 82.00 m n.m. Construction of the weir provides the conditions for further development of the channel in the cut-off.

Measuring of hydromorphological parameters in the Nemetin cut-off was done from the boat, using acoustic doppler current profiler (ADCP). The data on channel geometry and water velocity were collected on 17 designated cross-sections, from P040 to P170. The studied part of the cut-off includes the reach from river km 15+144 (cross-section P000) to km 10+955 (cross-section P230). The cut-off itself starts at river km 14+210 (section P060), and ends at km 12+374 (section p150). We used the 1200 kHz ACDP by RD Instruments with four beams. Measuring was done in the "single ping" mode, with cell size 25 cm, at the average ensemble frequency of 1.2 Hz. Raw data were collected by Win River computer program, and post-processed by the original vba computer program.

Velocity profiles were collected from transects (spatial series) and from recordings in one position (time series). One "vertical" raw velocity profile from ADCP is under predominant influence of turbulence, which makes it unacceptable for further processing. Therefore, the usual practice is averaging of a series of "raw" profiles in order to reduce the dominant fluctuation effect. Thus, in measuring in a single position averaging of velocity components in time was done, and in measuring over the cross-section averaging in space was carried out. Averaging in space was done in a similar manner as averaging in time, the only difference being the use of considerably lower number of ensembles, i.e. those measured at points of maximum depths of the cross-section.

Fig. 1. Plan view of the cut Nemetin in Drava River (Google Earth)

In this paper, the data on sediment discharge in the Drava were used, taken over from the study "Forecasts of morphological-psamological processes in the Drava river after construction of the hydropower plant Novo Virje" by Institut za elektroprivredu i energetiku d.d. Zagreb. This study gives the grainsize curve of bed material at the confluence with the Danube. It was shown that the river channel consists of sand, almost uniformly graded with the d_{50} of 0.35 mm. This value is chosen as the typical bed sediment size used in further analyses.

2 Theoretical Considerations

In watercourses the flow is turbulent and non-stationary, and the flow regime changes along the course and with changes of the discharge. The flow is mainly random and vertical, with pulsations of velocity and pressure

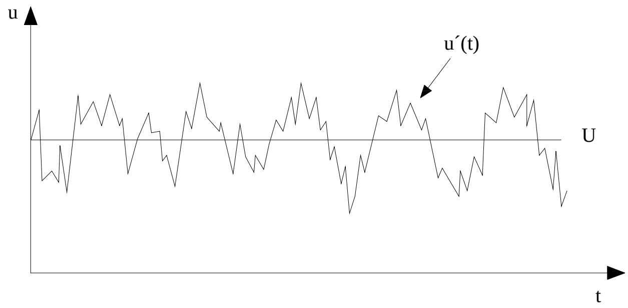


Fig. 2. Typical presentation of velocity measuring in turbulent flow

Random nature of turbulent flow makes it impossible to describe the movement of every flow particle in every moment of time. The usual practice is using of the Reynolds decomposition, where momentary velocities are defined by composition of average value of velocity U and the fluctuating component u'(t):

$$u(t)=U+u'(t) \tag{1}$$

Temporally average value of velocity is obtained by the formula:

$$U = \frac{1}{\Delta t}\int_0^{\Delta t} u(t)dt \tag{2}$$

While temporally averaged fluctuation component by definition is equal to zero:

$$\overline{u'} = \frac{1}{\Delta t}\int_0^{\Delta t} u'(t)dt \equiv 0 \tag{3}$$

Now it is not necessary to stress the dependence on time of the components u and u´, and we can write:

$$u=U+u' \tag{4}$$

Distribution of velocity and turbulence over the vertical is different in the boundary layer closer to the bed from the layer of free turbulence farther from the bed. In the flow over flat sloping bed the boundary layer is usually found to relative altitude from the bed z/H<0.2, and may be divided into the laminar sub-layer and the turbulent layer. Laboratory experiments prove the theory of constant shear over height in the laminar sub-layer, and logarithmic distribution of velocity over height in the turbulent layer (log-law). In flow over rough bed, changes of velocity profile and turbulence over height occur. Thus, the zero velocity is no more found on the bed, but is elevated from the bed by a certain value, which is most frequently related with hydraulic roughness. At rough bed, the model of logarithmic cross-section, proposed by Clauser (1956) is often used. In this paper, the bed shear is estimated according to the Clauser method and by the parabolic method.

2.1 Logarithmic Law (Clauser)

The logarithmic law, referred to by some authors as the Clauser method (1956), applies in the inner layer (z/H=0.2). White (1974) determines this regularity for flows with pressure gradient, and Song and Graf (1994) and Kironoto and Graf (1995) for flows over the gravel bed.

In the logarithmic method, velocity of shear stress depends on the direction of regression, describer by mean velocity u and ln[(z+0.15d_{50}/d_{50})]:

$$u = \alpha \ln\left(\frac{z + 0{,}15 d_{50}}{d_{50}}\right) + \beta \tag{5}$$

where β and α – constant and slope of regression direction.

Combining of this equation and the general law of the wall gives:

$$\frac{u}{u_*} = \frac{1}{\kappa}\ln\left(\frac{z+\omega}{d_{50}}\right) + B_r \tag{6}$$

where: z – distance from bed, u - velocity of shear stress, k-Karman constant (0.4), B-integration constant, ω- permissible deviation from surface z=0 and d_{50}- mean particle size.

Now we have the logarithmic law for calculation of bottom shear stress velocity, as follows:

$$u_* = \kappa\alpha \tag{7}$$

2.2 Parabolic Method

The parabolic method is derived from two formulas:

$$\frac{u_{max} - u}{u_*} = -A\ln\left(\frac{z + 0{,}1d_{50}}{H}\right) + E_1 \tag{8}$$

for inner layer ($0<z/H<0.2$) and

$$\frac{u_{max} - u}{u_*} = \lambda\left(1 - \frac{z + 0{,}1d_{50}}{H}\right)^2 \tag{9}$$

for outer layer ($0.2<z/H \leq 1$), where $A=1/k$, E_1-integration constant, λ-coefficient of parabolic method for velocity distribution:

$$\lambda = \frac{A}{2X(1-X)} \tag{10}$$

$$E_1 = \frac{A(1-X)}{2X} + A\ln(X) \tag{11}$$

where X=0.2, border between inner and outer layer. Dividing the equation with u_{max}, and rearranging we get:

$$\frac{u}{u_{max}} = -\frac{\lambda u_*}{u_{max}}\left(1 - \frac{z + 0{,}1d_{50}}{H}\right)^2 + 1 \tag{12}$$

Assuming that Ω-angle of regression direction described by u/u_{max} i $(1-(z+0.1d_{50})/H)^2$ we get:

$$\frac{u}{u_{max}} = -\Omega\left(1 - \frac{z + 0{,}1d_{50}}{H}\right)^2 + \phi \tag{13}$$

where ϕ- constant of regression direction.
From equations (17) and (18), velocity of shear stress u_ϕ may be written as:

$$u_* = \frac{\Omega u_{max}}{\lambda} \tag{14}$$

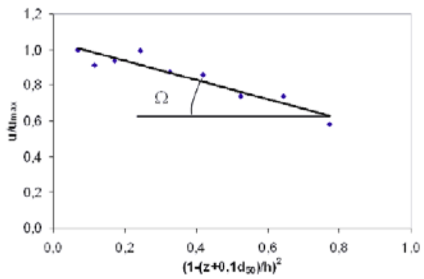


Fig. 3. Determining of regression direction slope for parabolic method

3 Results

3.1 Comparison of Measured and Theoretical Velocity Profiles

Theoretical logarithmic distributions of velocity profiles are adjusted to averaged profiles from time series. Measured and theoretical velocity profiles are shown along the studied river reach with regard to absolute depth *d* and to relative height from the river bed *z/H* (Fig. 4). Comparatively small differences between measured and theoretical velocity profiles are noticed. The differences are the least in reaches with uniform flow regime. The largest differences are noticed on the decelerating flow regions (cross-sections P100, P105, and P160) where the *S* curve of measured profiles is noticeable. Average deviation of measured and theoretical velocity values for both distributions is around 1 percent.

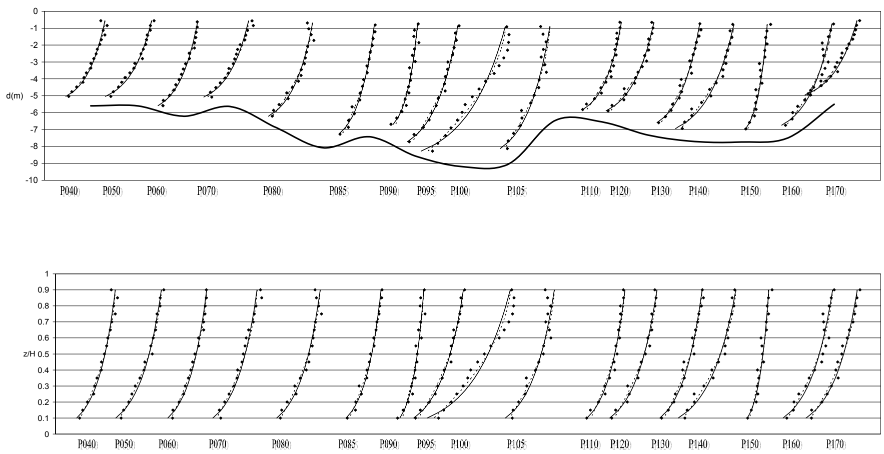
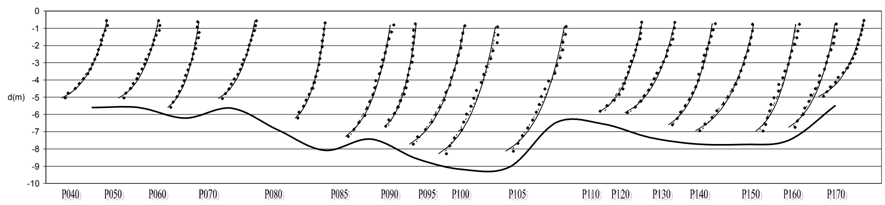


Fig. 4. Time series - Vertical velocity profiles *u* from time averaging (□), logarithmic (--) and parabolic law (- -) in relation to water depth *d* (above) and relative height from river bed *z/H* (below)

Spatially averaged velocity profiles are compared to theoretic profiles at the cross-sections (Fig. 5). Measured velocity profiles from spatial averaging also differ rather slightly from theoretical profiles on uniform flow sections. Like with time series, inflection of measured profiles (*S* curve) is noticed on decelerating flow reaches with retarded flow (cross-sections P070, P080, P100, P105, and P160) where the largest deviations of measured and theoretical profiles are noticed. Average differences between measured and theoretical velocity values for spatial series are, for both laws, around 2 percent.



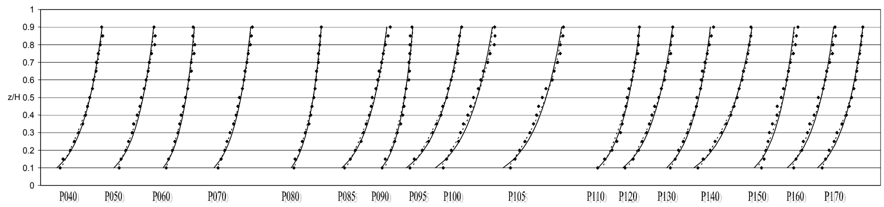


Fig. 5. Spatial series - Vertical velocity profiles *u* from spatial averaging (•), logarithmic (--) and parabolic law (- -) in relation to water depth *d* (above) and relative height from river bed *z/H* (below)

Table 1. Comparison of differences between theoretical velocity values and measured values in cross-sections

	Time series		Spatial series	
	difference Clauser (%)	difference Parabolic (%)	difference Clauser (%)	difference Parabolic (%)
P040	1.0	1.1	1.6	1.2
P050	1.6	0.9	1.9	1.1
P060	1.2	0.7	1.7	1.0
P070	1.3	0.6	2.7	1.9
P080	0.6	0.5	2.1	1.8
P085	0.9	0.9	1.0	0.9
P090	0.7	1.1	1.2	1.5
P095	0.8	0.9	1.1	1.6
P100	**2.7**	**1.7**	**6.1**	**3.8**
P105	2.2	1.1	3.2	3.0
P110	0.5	0.9	1.1	1.5
P120	0.8	0.6	1.3	1.5
P130	1.0	0.7	1.2	1.3
P140	0.9	0.7	2.5	1.8
P150	2.2	1.6	1.3	1.4
P160	1.7	1.0	2.5	2.2
P170	1.3	0.7	2.2	2.0
Average:	**1.3**	**0.9**	**2.0**	**1.7**

Table 1 shows relative deviations of measured and theoretical velocity values in cross-sections, for Clauser and Parabolic method. Slightly smaller differences between measured and theoretical profiles are noticed for time series than at spatial series. Also, somewhat smaller differences of measured and theoretical profiles are noticed for the parabolic method. Greatest differences are noticed in cross-section P100, which is located on the section with intense decelerating flow regime

3.2 Estimate of Bed Shear Velocity

Bed shear velocity is calculated separately for time and spatial series through logarithmic distribution of velocities (Clauser) and the parabolic method. Longitudinal distribution of shear velocity is shown for time series (Fig. 6) and for spatial series (Fig. 7). It is noticed that the largest values of bed shear occur at the decelerating flow (cross-section P100) for both time and spatial series.

From the comparison of bed shear velocity for time and spatial series a comparatively uniform mutual distribution of shear velocity along the reach is evident, except at cross-sections P040, P080, P100, and P170. A possible reason is a smaller sample of the number of verticals in spatial series, which cannot reduce fluctuation of measuring velocity to a satisfactory extent.

The values for shear velocity are shown for spatial and time series, as well as for both methods, Clauser and parabolic (Table 2). The differences of bed shear velocity for time and spatial series are about 19 percent on average, and the largest differences of about 48 percent appear at cross-section P100 at the decelerating section. Longitudinal distribution of shear velocity shows relatively small differences of shear velocity for both methods.

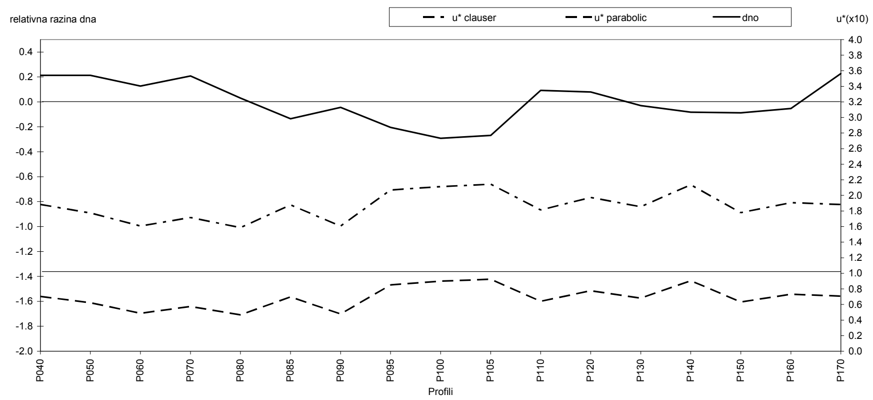


Fig. 6. Longitudinal distributions of bed shear velocity from time series

The comparison of measured velocity profiles for time and spatial series shows 7.4 percent difference of the *u* velocity component. The differences may be result of different sample sizes and from the number of “raw” verticals between time and spatial series. Several consecutive cross section transects would increase the number of verticals at a given cross-section

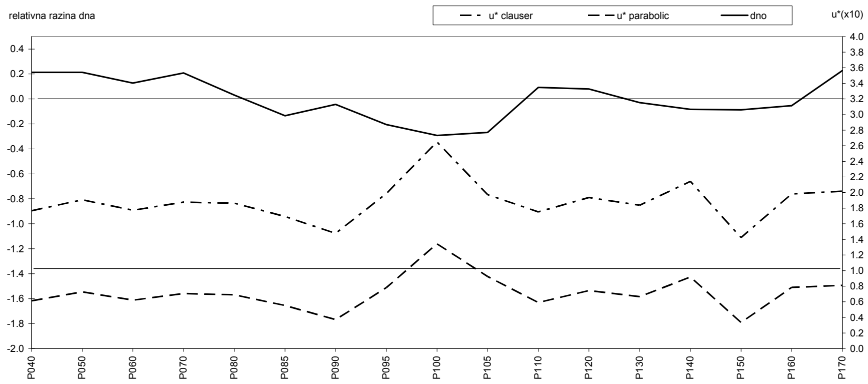


Fig. 7. Longitudinal distributions of absolute values of bed shear velocity from spatial series

Table 2. Comparison of bed shear velocity for time and spatial series

u*(m/s)	Series	Clauser	Parabolic
	spatial	0.088	0.070
P040	time	0.077	0.061
	difference in (%)	13.2	12.9
	spatial	0.077	0.062
P050	time	0.091	0.073
	difference in (%)	17.2	16.9
	spatial	0.061	0.049
P060	time	0.077	0.062
	difference in (%)	27.4	27.5
	spatial	0.072	0.057
P070	time	0.088	0.071
	difference in (%)	22.5	22.9
	spatial	0.059	0.047
P080	time	0.086	0.069
	difference in (%)	47.5	47.8
	spatial	0.088	0.070
P085	time	0.069	0.055
	difference in (%)	21.0	20.8
	spatial	0.061	0.048
P090	time	0.048	0.037
	difference in (%)	21.3	22.2
	spatial	0.107	0.085
P095	time	0.099	0.078
	difference in (%)	7.7	8.2
	spatial	0.111	0.090
P100	time	0.165	0.134
	difference in (%)	48.0	49.3

	spatial	0.114	0.092
P105	time	0.097	0.092
	difference in (%)	14.9	0.1
	spatial	0.081	0.064
P110	time	0.075	0.059
	difference in (%)	7.5	7.8
	spatial	0.097	0.078
P120	time	0.094	0.074
	difference in (%)	3.8	4.2
	spatial	0.085	0.068
P130	time	0.084	0.067
	difference in (%)	1.9	2.2
	spatial	0.114	0.091
P140	time	0.114	0.092
	difference in (%)	0.7	1.3
	spatial	0.078	0.063
P150	time	0.042	0.033
	difference in (%)	45.6	47.1
	spatial	0.091	0.073
P160	time	0.098	0.078
	difference in (%)	8.5	7.4
	spatial	0.088	0.071
P170	time	0.102	0.081
	difference in (%)	15.1	14.5
Average differece (%)		**19.0**	**19.0**

Table 3. Comparison of time and spatial averaged values of flow velocity (V) and mean flow direction (Dir), and velocity components (u), (v), (w)

Difference	**V (%)**	**Dir (%)**	***u* (%)**	***v* (%)**	**w (%)**
P040	6.0	12.6	2.2	10.8	3.6
P050	3.7	11.8	7.4	22.0	4.6
P060	5.7	6.0	3.1	6.8	10.1
P070	5.7	12.5	7.0	18.2	6.3
P080	8.9	30.9	11.3	7.3	5.6
P085	5.9	5.2	7.3	10.6	13.5
P090	9.2	7.0	4.7	6.1	4.8
P095	7.6	8.5	3.1	9.0	16.5
P100	5.8	18.1	7.3	7.5	8.2
P105	13.2	8.1	8.6	12.3	21.6
P110	3.2	13.6	10.7	25.3	5.9
P120	7.8	14.9	12.2	29.2	9.7
P130	6.4	8.5	7.3	15.2	11.0
P140	5.6	7.9	13.3	15.6	17.9
P150	9.1	6.4	15.1	15.8	11.7
P160	17.3	11.4	4.0	12.1	11.7
P170	5.5	8.7	7.5	12.9	6.7
Average	**7.4**	**11.3**	**7.8**	**13.9**	**10.0**

From the presented velocity cross sections along the course it is easy to assess the nature of flow in a given cross-section. In particular, as may be seen, at relations in cross-sections P070, P095, P100 and P105, the tangent of the cross-section curve has a considerably larger deviation from the vertical, which indicates decelerating flow and larger shear stress.

6 Conclusions

This paper shows the comparison of vertical velocity profiles and bed shear velocities estimates by ADCP measurements in non-uniform flow in a natural watercourse. “Raw” vertical velocity profiles from ADCP are averaged in time and space by original computer program. Two theoretical velocity distributions were adjusted to time and spatial averaged vertical velocity profiles, i.e. Clauser and parabolic method. The comparison of averaged profiles from time and spatial series shows the difference of the absolute velocity value of 7,4%, and the difference of *u* velocity component of 7,8%. The differences probably result from the smaller sample of verticals in spatial series. The possible solution may be to increase the sample of spatial series, i.e. consecutive repeating of cross-section transects. Comparatively small differences of averaged velocity profiles and theoretical velocity distributions are noticed where the average deviation between measured and theoretical velocities for both laws is about 2%. Largest differences between measured and theoretical profiles occur in river reaches with decelerating flow, where there is a notable inflection of measured velocity profiles (*S* curve). The differences of velocity profiles in time series from theoretical values are smaller than in spatial series. Average deviation of profiles in time series from theoretical velocity values is about 1%. Comparison of differences of bed shear velocity from time and spatial series indicates comparatively small differences and similar trends along the river reach. The largest differences of bed shear velocity from time and spatial series occur also with decelerating flow, and the average differences are around 20%. Shear velocities according to Clauser and parabolic method give comparatively small differences for all cross-sections.

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A49**

Boundary Elements for Modeling Fractured Porous Media: Single and Two Phase Flow and Transport

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Abstract. Understanding flow and transport processes in naturally fractured porous media is of interest in environmental engineering applications, geohydrology, oil reservoirs engineering, or in geological isolation of radioactive waste, when porous strata are made of rocks, which are crossed by networks of fissures and cracks. The importance of the flow and transport processes through fractured porous media has influenced development of different numerical models to predict the outcome of these phenomena. This paper compares three different models: equivalent continuum (EC) model, dual porosity (DP) model and discrete fracture/non-homogeneous (NH) model. In addition, the two-phase model for flow is analyzed. Its simulation, due to the high nonlinearity of the governing equations and their strong coupling, provides a challenging numerical problem. The Boundary Element Dual Reciprocity Method – Multi Domain scheme (BE DRM-MD) has been implemented, which has been used for first time to solve the dual-porosity model and the two-phase model.

Keywords: Flow transport, boundary elements, dual porosity model, two-phase model

1 Introduction

The quality problem of the water becomes the limiting factor in the development and use of water resources. Although it may seem that groundwater is more protected than surface water, it is still subjected to pollution, and when the latter occurs, the restoration to the original, unpolluted state, is usually more difficult and time-consuming process. Therefore, the interest in the mathematical and numerical treatment of fluid flow and transport in porous media, i.e. the necessity for sophisticated mathematical models and numerical tools capable of understanding, predicting and optimising all the physical phenomena occurring in this field, has been increasingly rising. Understanding flow and transport processes in naturally fractured porous media is of interest in environmental engineering applications, in geohydrology or in oil reservoirs engineering, when porous strata are made of rocks, which are crossed by networks of fissures and cracks. Recently, fractured rocks attracted the attention in connection with the problem of geological isolation of radioactive waste.

In spite of the large number of existing numerical packages for simulation of the fluid flow and contaminants transport in fractured porous media, most of them are based on the Finite Element (FEM) and Finite Difference (FDM) methods. The aim of this work, which is a short summary of the author's PhD thesis [7], was to develop an accurate and flexible software module which uses the Boundary Element Dual Reciprocity Method – Multi Domain scheme (BE DRM-MD), suitable for predicting flow and transport in a porous medium. The principal aim was to develop simple usable flow and transport models for porous fractured media, based on an understanding of discontinuity structures. Different models for fluid flow and contaminants transport processes for homogeneous and heterogeneous domains have been analyzed:

Single-phase flow and transport through fractured porous media

- non-homogeneous model (NH)
- equivalent continuum model (EC)
- double porosity model (DP)

Two-phase flow model through porous media.

The BE DRM-MD numerical scheme is used for the first time to solve a double-porosity and two-phase model. Verification of the complex numerical models is very difficult in absence of sufficient experimental measurements and accurate analytical solutions. Therefore, the comparison of the double porosity model with the non-homogeneous and simpler equivalent continuum model was expected to give in-depth understanding of the physical phenomena and opportunity to choose the correct model that should be used in certain real circumstances. Another objective is analysis of modelling two-phase flow in porous media, with complete insight in the complex nonlinear relationships between pressures, saturations and permeabilities. The influences of the choice of boundary and initial condition, the heterogeneity of the domain, the temporal and space discretization, and some physical parameters on the fluid process are important to be determined in order to clarify the mathematical model and areas of existing uncertainty.

2 Single and Two Phase Flow and Transport Models

Porous media often exhibit a variety of heterogeneities, such as fractures, fissures, cracks, and macro pores or inter-aggregate pores. The crystalline bedrock consists of solid rock, cut by a network of fractures. Water flows unevenly through an intricate network of paths formed by fracture intersections. However, water does not move along all of the fractures. For various reasons, no driving force exists in a number of fractures that are "dead-ends", but only in small parts, i.e. flow channels, of the fractures, which form the active flow paths with high permeability. Thus, water flows primarily along a small portion of inter-connected fractures (water-bearing fractures), while most of the fractures and other volumes with low fracture density (matrix blocks) contain essentially stagnant water. The hydraulic characteristics of the rock are mainly defined by the properties of the fracture network, i.e. the permeability, density, size and orientation distributions of the fractures. The microscopic structures or processes affect water and solute movement at the macroscopic level by creating non-uniform flow fields with widely different velocities. Therefore, when modelling groundwater flow, the characteristics of the porous media need to be considered. From a conceptual point of view, various models can be adopted to carry out the study of water flow in the far field [1]. The type of results aimed for, the data available, the scale of the modelled volume and some practical limitations like computational resources affect the selection of the modelling approach. Furthermore, the applicability of alternative methods for modelling various physical processes in a domain is different. In the groundwater flow analyses, the fractured porous media in the far field (Fig.1a) can be modelled conceptually with three alternative approaches: *the discrete fracture models (non-homogeneous NH),* (Fig.1b), the *double-porosity (DP) models* (Fig. 1c) and the *equivalent-continuum (EC) models* (Fig. 1d). The three of them, as well as the *two-phase model* (Fig. 1e), are described in this paper.

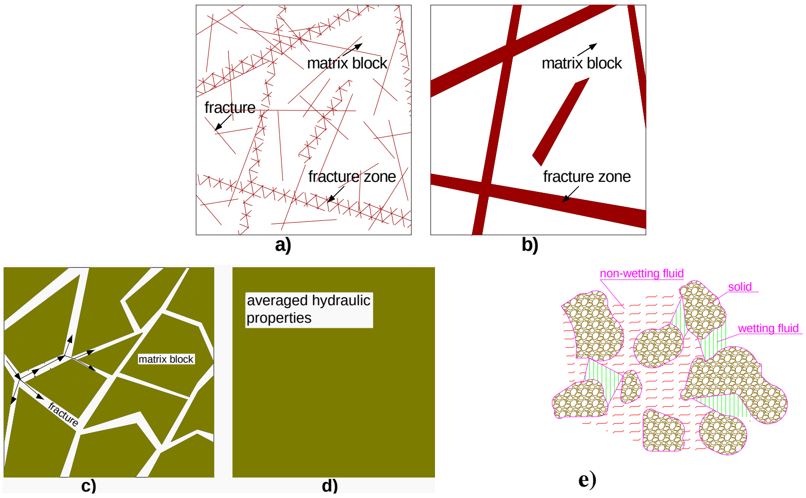


Fig. 1. a) fractured porous media; b) non-homogeneous representation; c) double porosity representation; d) equivalent continuum representation; e) two-phase model

2.1 Non-Homogeneous (Discrete Fracture) Model

The porosity and permeability for the non-homogeneous model are allowed to vary discontinuously and rapidly, as both quantities are significantly greater in the fractures than in the porous rock. Computational and data requirements for treating such a model are too large, which makes this approach not suitable for practical purposes, see [7]. The equation that describes the transient case of saturated flow in isotropic porous media can be written as:

$$C \cdot \frac{\partial h}{\partial t} + S_{ource} = K \cdot \nabla^2 h \tag{1}$$

where: C - specific storativity [L^{-1}], h- the hydraulic head [L], K - the hydraulic conductivity [LT^{-1}], t - time [T] and S_{ource} - the source term [T^{-1}].

Equation (1) is valid for both, porous matrix and fractures. The matrix–fracture interface is treated in the same way as in any other case of non-homogeneous medium. The flow velocity field is described by the Darcy law:

$$\vec{V} = -K\nabla h \tag{2}$$

The following equation for transient solute transport is used:

$$R\,\frac{\partial c}{\partial t} = \frac{\partial}{\partial x_j}\left(D_{ij}\,\frac{\partial c}{\partial x_i}\right) - v_i\,\frac{\partial c}{\partial x_i} - \Omega c \tag{3}$$

where: c – solute concentration [ML^{-3}]; R – retardation factor; Ω – coefficient related to a first-order chemical reaction [T^{-1}]; v_i –velocity in the x and y direction [LT^{-1}]; D_{ij} – dispersion coefficient [L^2/T].

The first term on the right – hand side of the equation (3) describes the influence of the dispersion on the concentration distribution; the second term is the change of the concentration due to advective transport, while the third one represents concentration changes due to decay and chemical reactions.

2.2 Equivalent-Continuum Model

In the equivalent-continuum (EC) approach, the same equations as for the NH model, (1) and (2) for flow and (3) for solute transport, are used. The difference from the NH model is that the fractures are not modelled explicitly, the fractured bedrock is treated as a continuum. No distinction is made between the water-bearing fractures and the matrix blocks, water is assumed to flow through the whole system. The hydraulic properties of the domain are averaged over the sub-volume, or representative elementary volume (REV), containing sufficiently large number of fractures; each hydraulic unit separately is treated as a homogeneous and isotropic continuum. They are estimated according to the equations for the simplest case of flow through domain intersected by a family of parallel fractures of equal aperture for the porosity and for the hydraulic conductivity, respectively:

$$n_{equi} = n_m \cdot \frac{V_m}{V_t} + n_f \cdot \frac{V_f}{V_t} \tag{4}$$

$$K_{equi} = \frac{\rho g}{\mu L}\left(\frac{m b^3}{12} + K_m\left(L - m b\right)\right) \approx K_m\,\frac{V_m}{V_t} + K_f\,\frac{V_f}{V_t} \tag{5}$$

where: n_f and n_m - porosities of the fractures and matrix blocks in the REV, respectively, V_f and V_m - volumes of the fractures and matrix blocks in the REV [L^3], respectively, V_t - total volume of the domain [L^3], L - total thickness of the domain [L], b - aperture of the fracture [L], m - number of parallel fractures, ρ – fluid's density [mL^{-3}], μ - dynamic viscosity [$mL^{-1}T^{-1}$], and K_f and K_m - hydraulic conductivities of the fractures and matrix blocks, respectively [LT^{-1}].

The equivalent dispersion coefficient is calculated according to the following expression

$$D_{equi} = D_m \frac{V_m}{V_t} + D_f \frac{V_f}{V_t} \tag{6}$$

where: D_f, D_m are dispersion coefficients in the fractures and matrix blocks [L^2/t], respectively. The calculation of the equivalent dispersion coefficient is obtained under assumption of parallel fractures and flow parallel to the fractures, and steady state condition when there is no more lateral diffusion into the matrix. Under such conditions the transport is also parallel to the fractures and the 2D problem is reduced to a 1D problem, both in the fractures and porous matrix, and the following equation is used for the respective dispersion coefficients:

$$D_{x(i)} = a_{L(i)} V_{x(i)} + D_{d(i)} \tag{7}$$

where: a_L - longitudinal dispersivity, D_d - the coefficient of molecular diffusion, and (i) stands for m, matrix block, or f, fracture.

2.3 Dual Porosity Model

As an alternative, discontinuous nature of the porosity and permeability can be avoided by replacing them locally by their average values, and the interchange between the fracture and the matrix must be modelled. In such a model, the void space of the fractures is visualized as a continuum (occupied by one or more fluids), while the void space within the blocks is regarded as another continuum that is occupied by the same fluid, or fluids, see [2], [7]. The two void-space continua may exchange fluid's (or fluids') mass between them at every macroscopic point within the considered domain. The transport of other extensive quantities, e.g. mass of the solute, may also take place within each of the two continua, with a possible exchange between them. Double porosity (DP) model is much more complicated mainly from the fact that since the fracture system is viewed as a porous medium, both matrix and fracture flow and transport are defined at each point of the matrix. The numerical double-porosity model assumes that the equation for transient water flow and the convection-dispersion equation for solute transport can be applied to both pore systems. Hence, macroscopically, two flow velocities, two pressure heads, two water contents and two solute concentrations characterize the porous medium at any point in time and space. The model assumes that the properties of the bulk porous medium can be characterized by two sets of local-scale properties: one set associated with the fracture pore system (subscript f) and the other with the matrix pore system (subscript m). Assuming applicability of Darcy's law, saturated water flow in the fracture and matrix pore regions are described by a coupled pair of equations, see [2]:

$$\nabla^2 h_f = \frac{C_f}{K_f}\frac{\partial h_f}{\partial t} + \frac{\alpha_w (h_f - h_m)}{w_f K_f} \tag{8.1}$$

$$\nabla^2 h_m = \frac{C_m}{K_m}\frac{\partial h_m}{\partial t} + \frac{\alpha_w (h_m - h_f)}{w_m K_m} \tag{8.2}$$

where: h – pressure head [L]; C – specific water capacity dθ/dh [L^{-1}]; K – hydraulic conductivity [LT^{-1}]; t – time [T]; w_f – relative volumetric proportion of the fracture pore system; w_m = 1- w_f ; Γ_w – water transfer term [T^{-1}]; α_w first-order mass transfer coefficient for flow [$L^{-1}T^{-1}$]

$$\Gamma_w = \alpha_w (h_f - h_m) \tag{9}$$

$$\alpha_w = \alpha_w{}^* \cdot K_a = \beta/_{a^2}\, \gamma_w K_a(u) \tag{10}$$

where: a - distance (L) from the centre of the fictitious matrix block to the fracture boundary (half width of the matrix block); β – dimensionless factor depending on the geometry of the aggregates, β =3 (for rectangular slabs) – 15 (for spheres); γ_w = 0.4 (more or less independent of the aggregate geometry and the applied initial pressure and conditions); K_a – effective hydraulic conductivity of the matrix at the fracture/matrix interface.

In the similar manner as for the flow, the solute transport in a saturated fractured porous medium is described using two coupled double-porosity advection-dispersion equations, here directly written as non-homogeneous Laplace equations for the sake of brief presentation:

$$\nabla^2 c_f = \frac{1}{D_f}\left(R_f \frac{\partial c_f}{\partial t} + v_{f_i}\frac{\partial c_f}{\partial x_i} + \Omega_f c_f + \frac{\Gamma_s}{w_f} \right)$$

$$\nabla^2 c_m = \frac{1}{D_m}\left(R_m \frac{\partial c_m}{\partial t} + v_{m_i}\frac{\partial c_m}{\partial x_i} + \Omega_m c_m - \frac{\Gamma_s}{w_m} \right) \tag{11}$$

2.4 Two-Phase Model

Two-phase flow consisted of two flowing fluids, which do not interchange mass and do not have reaction with the solid matrix, is the simplest multiphase flow, see Fig.1e). It can describe very complex processes, such as: unsaturated flow in porous media, reservoir problems in petroleum engineering, salt-water intrusion in coastal aquifers, etc. The elimination of the equation of the gas phase is often referred to as Richard's approximation and is the basis of conventional analyses of two-phase flow. The porous media itself is assumed to be incompressible.

The equations governing the flow of fluids in a petroleum reservoir are special cases of general balance laws. The most common case is two-fluid flow, consisting of water (subscript W) and non-wetting phase (subscript O) and Darcy's law holds for both the phases:

$$\eta\frac{\partial}{\partial t}(S_W\rho_W)-\frac{\partial}{\partial x_i}\left[\rho_W\frac{1}{\mu_W}K_{W_{ij}}\left(\frac{\partial}{\partial x_i}(p_W)-\rho_W g_i\right)\right]-\rho_W q_W=0 \tag{12}$$

$$\eta\frac{\partial}{\partial t}(S_O\rho_O)-\frac{\partial}{\partial x_i}\left[\rho_O\frac{1}{\mu_O}K_{O_{ij}}\left(\frac{\partial}{\partial x_i}(p_W)+\frac{\partial}{\partial x_i}(p_{COW})-\rho_O g_i\right)\right]-\rho_O q_O=0 \tag{13}$$

Equations (12) and (13) are not independent from each other, since they have to satisfy the condition that the fluids fill up the pore volume, i.e. the additional relations:

$$S_w+S_O=1 \tag{14}$$

$$p_{COW}=p_O-p_W \tag{15}$$

where: η -porosity; S_α - saturation of the fluid phase - density of the fluid phase α; p_α - pressure head in the fluid phase α ; p_{COW} - capillary pressure; μ_α - dynamic viscosity; $g_i=g\delta_{i3}$ - vector of gravitational acceleration; $K_{\alpha_{ij}}$ – conductivity of the phase α; $K_{\alpha_{ij}}=k_{r\alpha}K_{ij}$; $k_{r\alpha}$ - relative permeability of the fluid phase α; K_{ij} - intrinsic permeability tensor (fluid independent). The mobility of the fluid phase α is defined as $\lambda_\alpha=k_{r\alpha}/\mu_\alpha$. Therefore, mobility is a measure of the easiness with which a fluid will flow at particular saturation. The mobility ratio M is ratio between the mobility of the displacing phase and mobility of the displaced one, $M=\lambda_w/\lambda_o$. According the value of the mobility ratio, the mechanism of flow can be defined. Introducing Darcy's law for the wetting and non-wetting phase, neglecting the compressibility of both phases, introducing constant densities ρ_α, using few mathematical operations (summing and multiplying) with the above equations, one can obtain the final parabolic equation:

$$\begin{aligned}&\frac{\partial}{\partial x_i}\left[\bar{\lambda}K_{ij}\frac{dp_{COW}}{dS_W}\frac{\partial S_W}{\partial x_i}\right]+\left[v_{ti}\frac{df_W}{dS_W}+(\rho_W g_i-\rho_O g_i)\frac{d(\bar{\lambda}K_{ij})}{dS_W}\right]\frac{\partial S_W}{\partial x_i}+\\&+\eta\frac{\partial S_W}{\partial t}-q_W+f_W q_t=0\end{aligned} \tag{16}$$

The first term in the two-phase saturation equation (16) represents dispersive part, the second term is convective part, the third is mass storage term and last ones a source and sink terms. The equation is parabolic because of the presence of term that includes capillary pressure.

Neglecting only the gravitational effects and the source and sink term, for simplicity, and taking the significance of the capillary pressure gradient into account, one arrives to the so-called Mc Whorter problem, see *R. Helmig* [3]:

$$\eta \frac{\partial S_W}{\partial t} + v_{t_i} \frac{df_W}{dS_W} \frac{\partial S_W}{\partial x_i} = -\frac{\partial}{\partial x_i}\left(D_{ij} \frac{\partial S_W}{\partial x_j} \right) \tag{17}$$

where: $$D_{ij}(S_W) = k_{ro} K_{ij} \frac{f_W}{\mu_O} \frac{dp_{COW}}{dS_W} \text{ - dispersion tensor} \tag{18}$$

3 Conclusions

Three different models for solving flow and solute transport in fractured porous media are compared for this work. These models are: equivalent continuum (EC), double porosity (DP) and non-homogenous /discrete fracture (NH).

- The advantage of the EC model is that it is the simplest one and easiest to use. It can provide good agreement for a case where the equivalent characteristics of the fractured porous media are easy to be estimated, as in case of fracture network parallel to the flow. The disadvantage of the EC model is that it cannot provide insight in the processes of flow and solute transport in the porous matrix and fractures, and would provide less accurate results when the estimation of the equivalent characteristics of the fractured porous media cannot be easily performed.

- DP models can be used to obtain sufficiently accurate results for practical purposes, especially for modelling domains with large number of fractures with repetitive geometry and similar characteristics, not having to engage into preparation of complicated input data due to the complex geometry of the problem. The DP model offers more information than the EC model regarding the averaged properties of the flow and transport processes in the porous matrix and fractures. The sensitivity analysis of the DP model to variations of the transfer term showed that substantially different results can be obtained depending on the chosen parameters.

- The NH model provides the largest amount of information for the flow and transport processes of the three models that were compared and is usually seen as the most accurate, as it introduces a smaller number of assumptions than the other two models. The orientation of the fracture zones through the domain has little influence on the results obtained using the NH model, providing that the matrix blocks are homogenous and of uniform geometry and the fracture network consists of fractures with same properties. The disadvantage of the NH model is that the exact geometry of the fracture network is not known, the meshes for complicated geometries and distributions of fractures in the media are difficult to prepare, and it would impose serious computational difficulties in terms of CPU and memory requirements.

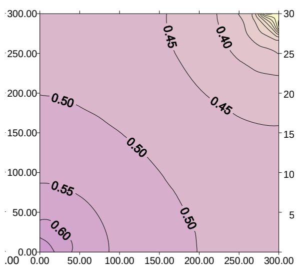
- The comparison of results for hydraulic heads and solute concentration showed good agreement for the three models in most of the cases. The results for water and solute fluxes showed that special care has to be taken when EC or DP models are used; especially for calculation of fluxes when the cross sectional area must be taken into account. For the DP and NH models the total flux consists of a flux through the fractures and a flux through the matrix blocks which participate in the cross section of interest. While the NH model can accurately estimate the fluxes since the total cross section of the fractures which participate in the cross section is accurately estimated, in the case of an isotropic DP model in a general case the fluxes would be estimated by using the volumetric factor of the fractures, which introduces errors. The problem arises since not only the "active" fractures, which contribute towards the flux, are taken into account, but also the fractures with stagnant water, providing a significant overestimate of the flux. Also, the fluxes inside the fractures are normally much higher than in the matrix blocks, making the error of the fluxes in the fractures more significant when calculating the total flux. One solution to this problem is to use more accurate estimate of the total aperture of the fractures, which participate in a given cross section, as was done in the examples. However, such approach cannot be used in the EC model, since there is only one type of porosity/permeability in this model.

For the two-phase flow, the fractional flow approach is used, or so-called Mc Whorter equation (17) was solved for all numerical examples. The fractional flow approach offers greater computational efficiency, because the two-phase system can be described with one nonlinear advection-diffusion equation only, and because the equation has a dominant hyperbolic part which can be treated with characteristic solution methods. Some conclusions are withdrawn from the analysis of different examples:

- Two-phase flow model was implemented using the boundary element DRM-MD scheme and tested on one-dimensional and two-dimensional examples. Application of DRM-MD scheme leads to solutions that are basically oscillation free, even in the presence of steep infiltrations fronts.

- Even the coarse grid simulation produces useable and accurate solution, though the results on the finer grids are considerably better. It is oscillation free and shows a good mass balance.

a)



b)

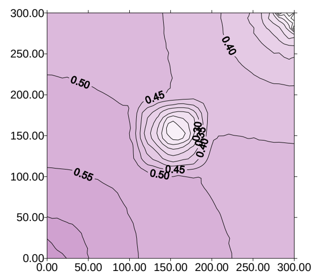


Fig. 2. Two-phase model, five-spot example: a) homogeneous domain; b) heterogeneous domain

- Areas with different permeability and porosity can be treated and identified in numerical simulation. The DRM-MD scheme performs very well for non-homogeneous domains, even for great differences in the permeabilities, Fig. 2.

- Testing simulation with different time steps on each of these meshes showed that one should be extremely careful with the choice of the time step when rather coarse mesh is used for the domain.

- Thinner domains showed more stable fronts and slightly slower breakthrough compared to wide fracture porous systems.

- Linear type of constitutive model $k - S - p$, as the simplest one, can be used for two-phase flow. The Brooks-Corey model appeared to be the most easiest and accurate at the same time, because it has two fitting parameters only and its derivative is easier to calculate. The Van Genuchten model overestimates the saturation and has more dispersive front.

It is important to point out that to assemble appropriate material relationships, and coherent set of boundary and initial conditions, in other words, to use the two-phase model appropriately, in-depth understanding of the physical processes is required, because of the strong nonlinear coupling. Unfortunately, quantitative measurements of two-phase fluid flow are lacking in the literature. This work points to the necessity for further laboratory experiments designed to obtain the data necessary to verify numerical predictions of capillary dominated immiscible flow.

The examples showed that this BE formulation provides stable results and can be used successfully for solving flow and transport processes in fractured porous media. The major reason why the BE DRM-MD can be attractive is that it is advantageous for modelling heterogeneous domains with different physical properties and it does not have the problems related to the coupling compatibility of the FEM – BEM hybrid methods.

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A51**

Experimental Investigation of Relationship between Sump Flow Pattern and Pumping Energy Consumption

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Abstract. Regarding to importance of optimization of energy consumption and the large amount of energy which are devoured for water supply in pumping stations, an experimental set up was designed to study some affective hydraulic-geometric parameters on pumping energy consumption. In this study different pumping discharges were employed and in each discharge the formed flow pattern under the several depths of suction pipe submergence was recorded. The measuring of pressure and velocity was done in a 3D grid of sump to compare the flow patterns in several cases. In each set of experiments the amount of used electricity was recorded as well. Due to the results the amount of required energy for pumping a constant volume of water is dependent on sump flow pattern and, consequently, on geometric design of pumping system. Also it could be resulted that some of the proposed pump curves are not reliable and sufficient to consider the required amount of energy. Thus, in addition to the pump type, the best form of geometric design should be selected to maximize the energy conservation.

Keywords: Sump design, pumping station, energy consumption, flow pattern

1 Introduction

A large amount of energy is consuming in pumping station for water supply [1]. In addition to hydraulic features, such as the head and discharge of pumped water, geometric design of pumping could affect the amount of essential energy. A number of deficiencies and problems with pumps are often related to sump design rather than to mechanical imperfections. Vibration, cavitation, rough running lower than expected efficiency, and reduced pump life can generally be traced to flow condition in the sump and its approach area [2]. As a rule, inappropriate situation of pumping, and consequently more consumption of energy are related to intensity of disturbance of flow in pumping sump. There are some criteria to design sump dimensions, submergence depth and diameter of suction pipe, which are normally permanent for any point of pump curve. Some experimental and numerical studies show the formed flow pattern due to situation of approach channel as well [2,3,4]. In this study we notice the flow pattern under several amount of discharge and submergence depth of suction pipe. In each set of experiments the manometric pressure in sump bottom and three direction velocities in 3D grid of flow were recorded.

2 Experimental Setup and Methods

The designed setup for this research was constructed in the central water laboratory of university of Tehran. A transparent reservoir as a pumping sump, 50 centimeter above ground, was built. The coupled pump and electromotor used in this research were installed above the pumping sump. The model number of pump is 80-250 and electromotor is rotating at 1450 rev/min. The distance from pump axis to sump bottom was selected 215 cm. The inlet part of sump was designed as a perforated pipe to calm down the inflow water.

Pressure measurement was done using manometers installed in selected points of sump bottom (Figure 1). Therefore the diversion from hydrostatic pressure in each point could be computed. The constant depth of water in the reservoir which was adjusted in three levels for each discharge, demonstrate the hydrostatic pressure. The mentioned levels were fixed in each set of experiments through a side weir. Consequently in each discharge it was considered to have three amount of submergence length of suction pipe: 52.5 cm, 42.5 cm and 32.5 cm. This parameter could affect the flow pattern and vortex formation in pumping sump [4]. A complete description of data sets is showed in table 1.

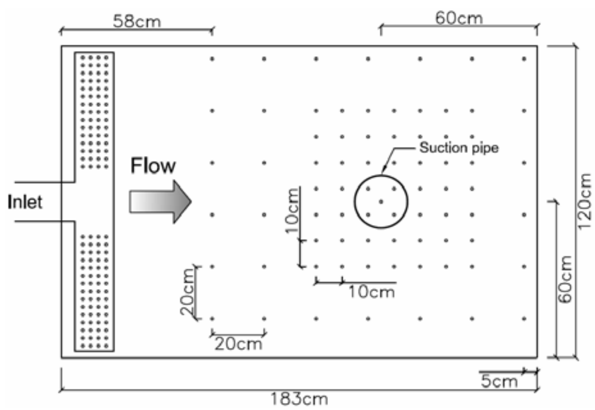

Fig. 1. A plan of pumping sump and layout of measuring points

Velocity measurements were done by means of an ADV probe which is able to record 3D velocities during arbitrary time period. ADV was installed in five levels above each showed points in Figure 1 for each set of data. Depicting of velocity vectors could show the magnitude and direction of velocity in each measured point and then the flow pattern could be compared for different sets of data.

A handy wafer valve was responsible to flow control in discharge pipe. In maximum discharge the velocity in suction pipe was about 1.05 m/s. A three-phase and analog recorder was employed to record the amount of energy (Kwh) along the experiments.

Table 1. Explanation of experimental data sets

Data Set Number	Discharge (l/s)	Water Level in Reservoir (mm)	Submergence Length of suction pipe (cm)	Pressure in suction pipe (bar)	Pressure in discharge pipe (bar)
1	15.2	625	525	-0.13	1.5
2	23.35	625	525	-0.13	1.18
3	33.02	625	525	-0.145	0.28
4	15.24	525	425	-0.13	1.5
5	23.3	525	425	-0.14	1.18
6	33.04	525	425	-0.16	0.3
7	15.3	425	325	-0.14	1.5
8	23.28	425	325	-0.15	1.18
9	33.03	425	325	-0.17	0.3

3 Results

Due to recording of energy consumption (E.C.) during the time of experiments, this parameter was computed for each hour and then for pumping a constant volume of water under the defined hydraulic and geometric condition. The results are shown in figure 2. In spite of strong vortex formation in data set 8 and 9 (Figure3), there is no significant difference between their E.C. and the others. According to figure 2 only the application of submergence length of 42.5 cm for the suction pipe in minimum discharge could reduce the E.C. relative to other submergence length.

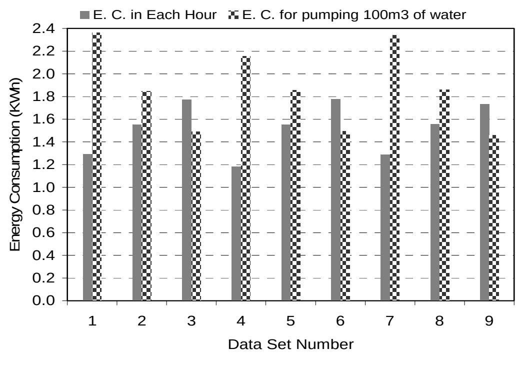

Fig. 2. Energy consumption in different data set

The Velocity pattern in x-y plot (in two different depths) for the data set 4 with minimum E.C. in 1 hour and data set 6 with maximum E.C. in 1 hour could be seen in figures 4 and 5.

Fig. 3. Vortex formation in the surface of the sump and its connection to suction pipe (Picture is taken from below the sump box)

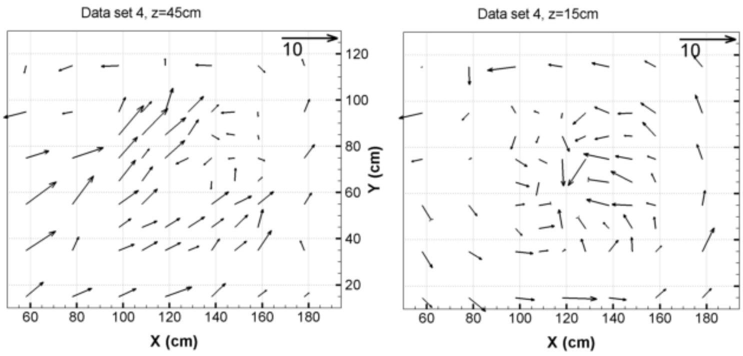

Fig. 4. Velocity pattern in x-y plot for the data set with minimum energy consumption in 1 hour

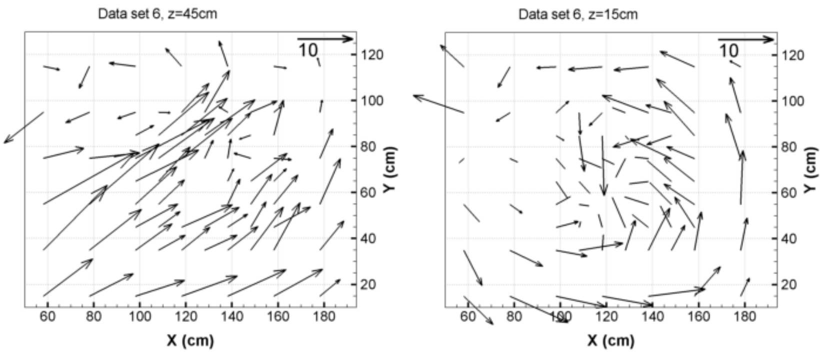

Fig. 5. Velocity pattern in x-y plot for the data set with maximum energy consumption in 1 hour (the length of reference vector is 10 cm/s)

The effect of discharge on flow pattern in a constant submergence depth of suction pipe is clear in figure 6. In this figure two dimensional velocity vectors in x-y plot are shown only in depths 15 cm and 45 cm from bottom which is respectively near the bottom and far from it.

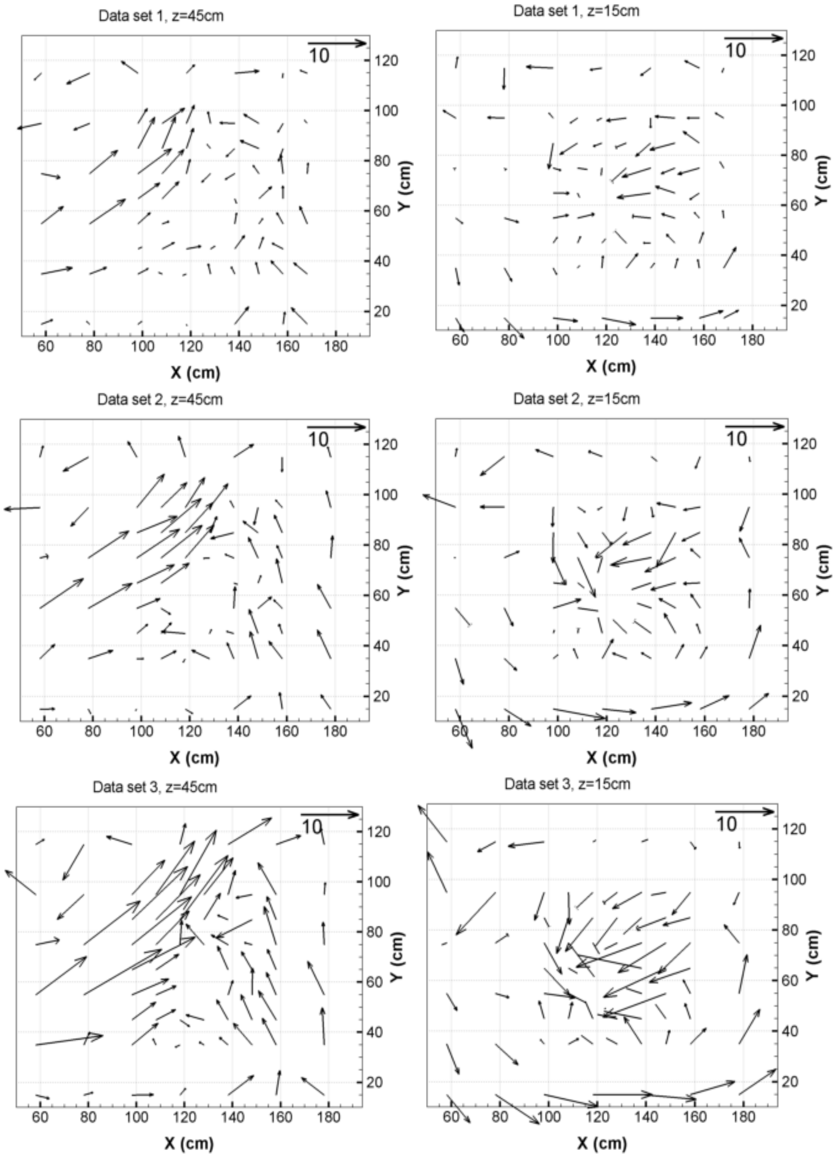

Fig. 6. Velocity pattern in x-y plot in 2 depth of water in sump for submergence depth of suction pipe equal to 52.5 cm

In despite of more sudden changes in direction and amount of velocity vectors in higher discharge, it consumes less energy for a constant volume of water. Besides to velocity vectors, the recorded pressures in sump bottom show the effects of pumping discharge and submergence of suction pipe on flow pattern. In lower discharge and more suction pipe submergence, the differences from hydrostatic pressure are fewer. Adversely, in data sets 6, 7, 8 and 9 a strong negative pressure core under the suction pipe axis is recognizable. Figure 7 shows the results of manometric pressure measurement under the suction pipe.

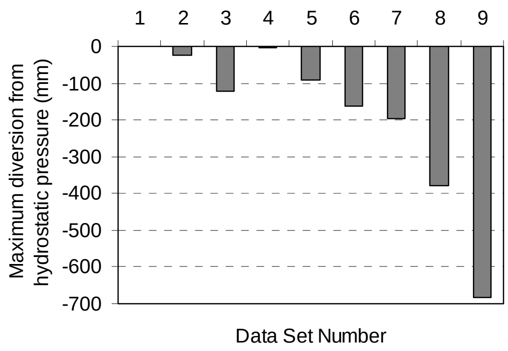

Fig. 7. Maximum diversion from hydrostatic pressure in the sump bottom which occurs exactly under the center of suction pipe

4 Conclusions

In the range of discharge which employed in this research regarding to the kind of pump and electromotor, the flow pattern could not affect the energy consumption significantly. It is possible to use higher discharge to pumping a constant volume of water with lower energy. But according to high negative pressure which is produce and may disturb the bottom of sump, especially in low submergence depth of suction pipe, it is recommended to use a submergence depth for suction pipe about 2.2D (D is the diameter of suction pipe).

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A56**

Debris Flow Impact Estimation

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Abstract. Alpine regions are exposed to several gravitational hazard processes. Such processes are debris flows, landslides or avalanches. Human settlements are protected amongst other things by structures adjusted to a certain process. The preparation of methods of calculation for the estimation of a debris flow impact force against such protection structures made of structural concrete is the content of this paper. The paper is strongly related to the Austrian Code Series ONR 2480X.

Keywords: Debris flow, debris flow impact, mitigation structures

1 Introduction

Alpine regions are characterized by some special conditions, which influence structural concrete. For example, on one hand extreme climatic condition can be found in this region, but also some mass transportation processes can be found on the other hand. Such processes often expose a hazard to humans and human settlements. Examples of processes driven by gravity are debris flows, landslides, rock falls or avalanches. Debris flows can transport up to several hundred thousand cubic meter of sediment from the mountain torrent catchment into the valley and deposit it onto an alluvial cone at the deposit area. Because such events occur very irregularly and very locally, they are difficult to observe and numerically to describe.

To develop Alpine regions and to permit human settlement, different mitigation measures against such hazardous processes are necessary. Based on an engineers view such measures can be distinguished into structural and non-structural mitigation measures. Structural mitigation measures are for example debris flow barriers. Furthermore mitigation measures can be divided into active and passive mitigation measures. Active mitigation measures intervene direct into the process and prevent or dampen the process itself whereas passive measures not directly influence the hazardous process and only limit the damage caused by the process.

One type of structural mitigation measures are debris flow barriers (Fig. 1). Debris flow barriers consist of a debris flow breaking structure, a retention basin and a pre-structure. Usually the debris flow barriers are built of structural concrete. The barrier itself consists of several piers and walls. More information about the general design can be found in [4], [6], [9], [12]. Because these structures are entirely designed as protection measure against debris flows, first the term debris flow should be introduced.

Fig. 1. Example of debris flow barrier made of structural concrete (view upstream)

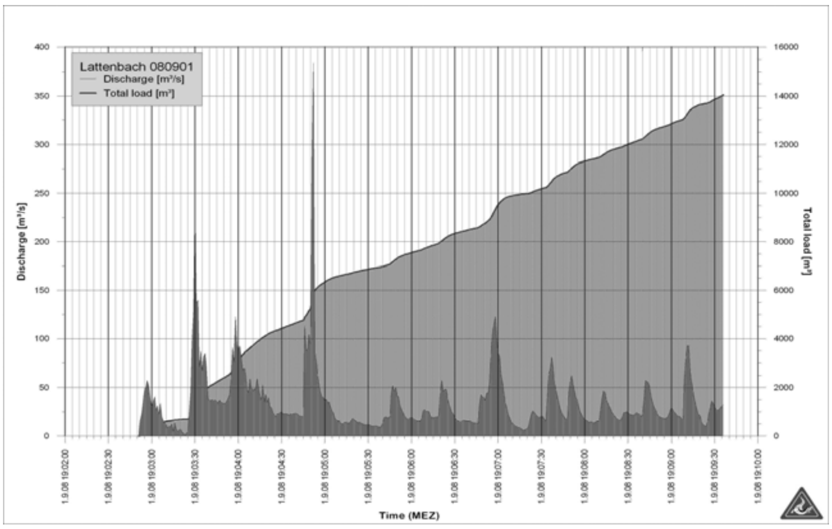

Fig. 2. Debris flow surges in the Lattenbach catchment, Tyrol, Austria (unpublished)

Debris flows are extremely mobile, highly concentrated mixtures of poorly sorted sediment in water (Pierson [22]) and may appear as different surges (Fig. 2). The material incorporated is inherently complex, varying from clay sized solids to boulders of several meters in diameter. Due to their high density (exceeding that of water by more than a factor of two) and their high mobility, debris flows represent a serious hazard for people, settlements, and infrastructure in mountainous regions. The front of a debris flow can reach velocities up to 30 m/s (e.g. Costa [5], Rickenmann [23]) and peak discharges tens of times greater than for floods occurring in the same catchment (e.g. Pierson [22]; Hungr et al. [14] (ONR 28400 [21]).

Fig. 3 shows the typical feature of a debris flow longitudinal section.

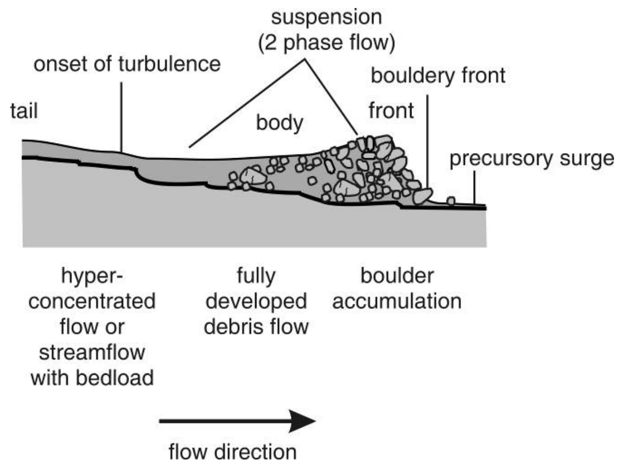

Fig. 3. Sketch of a debris flow surge (Pierson [22])

2 Debris Flow Impact Models

2.1 Observations and Experiments

To estimate the impact force of a debris flow against a concrete structure observations are required. Currently it is not possible to develop models only based on theoretical considerations.

The observations and experiments can be distinguished into observations under real-world conditions and experiments in laboratories. The observations under real-world conditions allow the measurement of impact forces of real debris flow events. Examples of such measurements can be found by Zhang [32] in China, Hübl et al. [11] in Austria or Wendeler et al. [30] in Switzerland. Fig. 4 shows a measurement station during the observations from Hübl.

However the measurement of further indicators, such as speed, density or flow height is often complicated. Also the time of the debris flow is difficult to predict. Therefore besides field measurement also experiments in laboratories are carried out.

These experiments are mainly miniaturized experiments (Fig. 5), since real size debris flows experiments are virtually impossibly to carry out in laboratories.

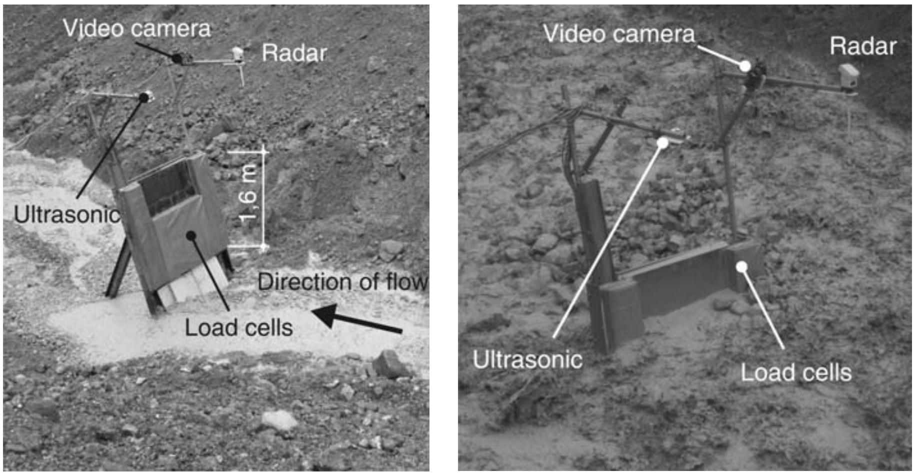

Fig. 4. Experimental set-up for debris flow impact measurement before (left) and after the debris flow (right)

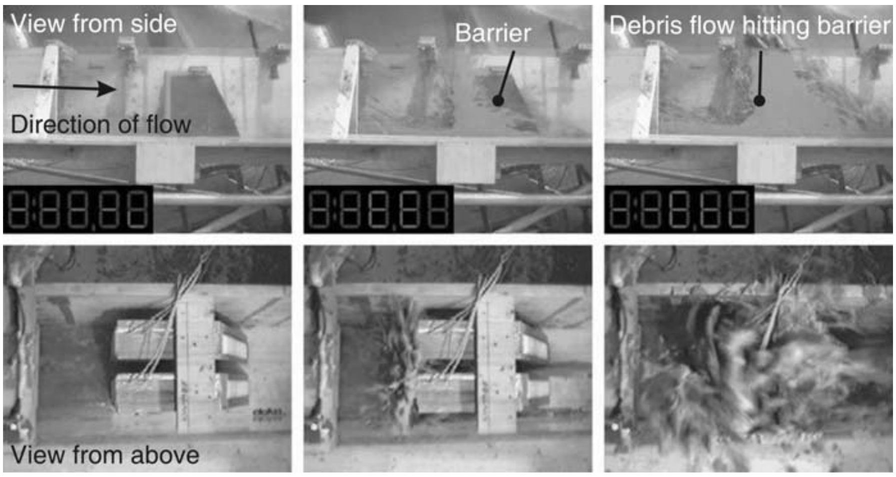

Fig. 5. Debris flow impact in a miniaturized test set-up

In the last years results of such experiments were published by Scotton [24], Ishikawa et al. [15], Hübl & Holzinger [13] und Tiberghien et al. [26]. Fig. 5 shows an example of a test by Hübl & Holzinger in order to derive maximum impact forces on structures. The disadvantages of this type of tests are possible scale effects.

2.1 Types of models

To estimate the impact force of debris flows against barriers several different models exist. The models can be classified into hydraulic and solid collision models. The hydraulic models are further separated into hydro-static and hydro-dynamic models. Examples of hydro-static models are formulas by Lichtenhahn [20] und Armanini [2]. Still, in practice the simple formula by Lichtenhahn is very popular, because only the debris flow height is required. And because often the height of the structures is taken as debris flow height, there are no unknowns in the formula and the engineer can easily design.

In general, the hydro-static formulas have the appearance:

$$p_{\max} = k \cdot \rho_{Mu} \cdot g \cdot h_{Mu} \tag{1}$$

with
- $p_{\max}$ maximum debris flow impact pressure in N/m^2*
- k empirical factor
- ρ_{Mu} density of debris flow in kg/m^3
- g gravity in m/s^2
- h_{Mu} debris flow height in m

*The maximum here is not related to statistical considerations, but to the maximum pressure value in the load distribution on the structure.

In contrast, the hydro-dynamic formulas have the appearance:

$$p_{\max} = a \cdot \rho_{Mu} \cdot v^2 \tag{2}$$

with
- $p_{\max}$ maximum debris flow impact pressure in N
- a empirical factor
- v velocity of debris flow in m/s

The empirical factor value a depends on the flow type. For example, for laminar flow and fine grained material Watanabe & Ikeya [29] estimate 2.0, for coarse material values up to 4.0 are given by Egli [7] and Geo [8]. Zhang [32] recommends values between 3.0 and 5.0. The values of Zhang are based on field measurements of over 70 debris flows.

In some publications, for example in Vandine [27], Ishikawa et al. [15] or Hungr et al. [14] a is considered as flow cross section. However, then the units have to be adapted.

A special representation of the hydro-dynamic formula is given by Hübl & Holzinger. Here the measured impact force (miniaturized tests) is normalized against the hydro-dynamic formula. Furthermore the Froude-Number has been used to achieve scale free relationships and has been related to the normalized impact force. Based on a correlation analysis a numerical expression is given as:

$$p_{\max} = 5 \cdot \rho_{Mu} \cdot v^{0.8} \cdot (g \cdot h_{Mu})^{0.6} \tag{3}$$

Mixed models considering hydro-static and hydro-dynamic elements can be found from Kherkeulitze [17] and Arattano & Franzi [1].

Besides the hydro-related models also models for solid body impacts are used for the estimation of debris flow impact forces. Here a shift towards rock fall force estimation can be found. The solid body impact models are mainly based on the Hertz model assuming elastic material behavior. However, also alternative models considering viscous-elastic and elastic-plastic behavior are known (Kuwabara & Kono [18], Lee & Hermann [19], Walton & Braun [28] and Thornton [25]). Furthermore some publications use the Kelvin-Voigt model based on spring-damper-systems.

Additionally there exist some special models which are not clearly related to some mentioned type of models. For example Yu [31] has published an empirical model. Also Aulitzky [3] has introduced a model considering the shock wave speed inside a debris flow. However both models use input data, like the shock wave speed, which are extremely difficult to obtain for real-world debris flows.

After a short overview about the variety of models, hydro-static and hydro-dynamic models are more intensively discussed. In Fig. 6 data from field measurements and from miniaturized laboratory tests is shown in a diagram, which considers on abscise the Froude number to achieve scale-invariant description and on the ordinate measured impact forces normalized either by the hydro-static (right) or hydro-dynamic (left) models.

Furthermore in the diagram two rectangle areas are visible: one on the left side reaching from the Froude number 0 to about 2 and the other one reaching from the Froude number 1.2 to 12. The second rectangle reaching from 1.2 to 2.0 is the range, in which mainly the miniaturized tests were carried out, whereas the area from 0 to 2 is the range mainly found in field measurements. Only the tests by Tiberghien et al. [26] were miniaturized tests reaching this area. Concluding one can state, that models are developed of an input data range which does not comply with field data. This is a systemic error.

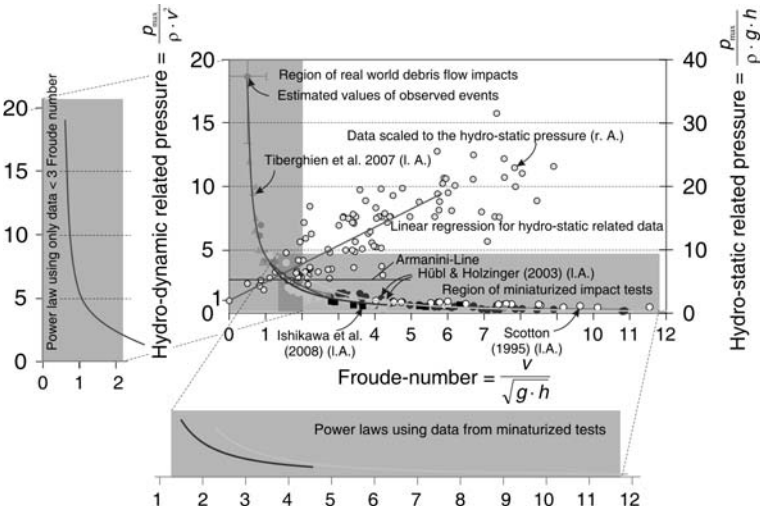

Fig. 6. Relationship between debris flow impact force and Froude-number considering field data as well as miniaturized laboratory tests

Additionally it becomes obvious on Fig. 6, that the hydro-dynamic models do not perform very well with low velocities and low Froude numbers. This is understandable since hydro-dynamic effects are not dominating in hydro-static pressure conditions. On Froude numbers higher 2 however, hydro-dynamic models work very well. In contrast, hydro-static models are very appropriate for low Froude numbers, less then 1. For higher Froude numbers and velocities impact forces are underestimated. It can be summarizing that hydro-static and hydro-dynamic models do not perform very well in Froude region found in field data. As a proof, Table 1 lists data from some field debris flow estimations and measurements.

Table 1. Debris flow properties estimated on field events based on Costa [6] and computed impact forces, empirical factors *k* and *a*, and the Froude-number

Torrent	h_{MU} [m]	ρ_{Mu} [kg/m$^{3]}$	v [m/s]	p_{max} in MN/m^2	k**	a**	Fr**
Rio Reventado	8-12	1130-1980	2.9-10	0.7	4.67	18.67	0.50
Hunshui Gully	3-5	2000-2300	10-12	0.7	8.33	2.31	1.90
Bullock Greek	1.0	1950-2130	2.5-5.0	0.13	6.50	4.06	1.26
Pine Creek	0.1-1.5	1970-2030	10-31.1	0.3	21.43	0.38	7.56
Wrightwood Canyon (1969)	1.0	1620-2130	0.6-3.8	0.07	3.68	4.09	0.95
Wrightwood Canyon (1941)	1.2	2400	1.2-4.4	0.15	5.21	6.94	0.87
Lesser Almatinka	2.0-10.4	2000	4.3-11.1	0.6	4.29	6.12	0.84
Nojiri River	2.3-2.4	1810-1950	12.7-13.0	0.44	10.07	1.37	2.71

* Mean values and based on the Hübl & Holzinger Formula
** Mean values

Parallel to the comparison of field data and the models, also the hydro-static and hydro-dynamic models should be directly compared by transferring the hydro-dynamic models into hydro-static models. This can be carried out by the application of the Bernoulli-energy line and results in empirical factors *k* for all models.

The Bernoulli-energy concept is here used in the following form:

$$k \cdot \rho_{Mu} \cdot g \cdot h = a \cdot \rho_{Mu} \cdot \frac{v^2}{2} \tag{4}$$

The factor 1/2 was already been visible in Fig. 6, when different scaling was used on the left and right axis. Table 2 lists the computed empirical *k*-factors for all formulas. These values can now be compared with the observed values given in Table 1. In general, the k-factors show a great diversity. This is not surprising, since the *k*-factor has to consider many different aspects of such an impact. In Table 1 the factor mainly reaches from 6.0 to 7.0 but two values exceed 10. This is in accordance with the results of Zhang, who estimated the factor based on field measurements between 6.0 and 10.0. Also the formulas from Armanini and Hübl & Holzinger with *k*-factors between 5.0 and 7.5 are in agreement with the observed field data.

Table 2. Estimation of empirical k-factors for different models

Author	Empirical *k*-factor	Remark
Kherkheulidze	~1.0	Mean values, no maximum values
VanDine	1.25 × A	Transferring difficult, introduction of an area A
Watanabe & Ikeya	4.0	
Lichtenhahn	2.8-4.4	Transfer from density water to density debris flow
Armanini	5.0	
Zhang	6.0-10	Field measurement (no scaling)
Hübl & Holzinger	7.5	Transfer difficult: exponents
Tiberghien	13.5	Miniaturized test set-up
Aulizky	25.0-50.0	Shock wave speed in debris flow estimated

As already in the explanation of Fig. 6 shown the miniaturized measurements are often not in the region of field observed Froude numbers. Therefore the robustness and extrapolation capability of the regression formulas has to be proofed. This will be shown for the Hübl & Holzinger formulae. Here the robustness of non-linear computed regression formulae is tested carrying out the following steps:

-Incremental consideration of further data (Scotton, Tiberghien et al. und Ishikawa et al.)

-Exclusion of outliers

-Limitation of data for some Froude number regions used for regression

-Test of alternative mathematical formulations like Harris-model, hyperbolic modes, Hoerl models, root models etc.)

As an example of the investigation, the regression coefficients of the formulae type are summarized in Table 3.

$$K'' = a \cdot Fr^b \text{ with } K'' = \frac{p_{\text{measurement}}}{\rho \cdot v^2} \text{ and } Fr = \frac{v}{\sqrt{g \cdot h}}, \quad (5)$$

Table 3. Results of non-linear regression for different populations

	Data from Hübl & Holzinger, Scotton, Tiberghien et al. and Ishikawa et al.	Data from Hübl & Holzinger, Scotton, Tiberghien et al., Ishikawa et al. and Froude-number <3	Data from Hübl & Holzinger, Scotton, Tiberghien et al. and Ishikawa et al. without outliers and Froude-number <3	Data from Hübl & Holzinger and Scotton	Data from Hübl & Holzinger	Selected data with Foude-number <3
a	5.2	5.0	5.0	5.62	4.9	5.9
b	-1.6	-1.6	-1.30	-1.66	-1.29	-1.50
r	0.96		0.881	0.9645	0.8755	0.966

The design engineer has to choose from the variety of models. For this selection the models should fulfil some general requirements:

- Models should be convergent, meaning that with increasing quality of input data the quality of the model result should also increase.

- Models should be robust, meaning that small changes in the input data should yield only to small changes in the results. Even further the model may perhaps applied in input data regions, in which it was not originally developed.

- Models should not have a systematically error meaning that the average statistical error is zero.

Furthermore model requirements:

- The input data for the models has to be either computational able or measurable. Even further input data with high weighting inside the formulae should be more precisely known than the other input data.

- The model should be easy applicable and be usable in practice.

- The model should at least partially have some theoretical background.

- The model should be chosen according with historical models, if these historical models have been proven of value.

- If two models reach the same accurateness, the model with less required input data should be chosen.

As shown already, all introduced models consider of significant errors in some regions of the Froude-number. However, it is then very important to clearly state the application limits. Some of the inherent model errors should be stated clearly to better understand the model capability.

All hydraulic models mentioned so far are based on Newton fluids (the viscosity is independent from the velocity). However since debris flow is a highly concentrated mixture of poorly sorted sediment in water, the viscose shear strength is velocity dependent and additionally other effects like plastic strain, turbulences and dispersion appear. All this effects are only included in the empirical k- or a-factor.

The impact of solid particles is only considered by Vandine [27]. However, measurements by Zhang [32] show, that the single boulder impacts against solid structures yield to the highest impact forces. To model this hard impacts further information about the stiffness and strength of the impacting bodies are required. The modelling by forces would not be possible anymore. This would yield to a dramatic increase in the modelling complexity and would finally lead to models, which are not applicable under practice conditions. Furthermore such a model would require extensive input data. For simplification reasons the so-called Hertz impact model based on elastic material properties is preferred. The related time-force-functions for debris flow are currently under development. Some work is done by Tiberghien et al. [26] and Zhang [32].

Additionally, impacts of solid particles in fluids have to consider so-called hydraulic active mass of the fluid. Such hydraulic active mass, which virtually increases the mass of the impacting body, can reach up to 20 % of the original mass of the impacting body. This effect increasing the impact force is counterbalanced by a decrease of the impact force by deposited debris flow material in front of the concrete elements. Preliminary tests carried out at the University of Natural Resources and Applied Life Sciences have shown a significant effect. However both, the hydraulic active mass and deposited debris flow material are not considered in the model.

4 Conclusions

Based on the presented investigation, the new ONR 24801 will include two design models for debris flow structural concrete barriers. Since hydro-static models have been applied very successfully over the last 30 years, the model of Armanini for the computation of the maximum pressure will be included. On the other hand, parallel to Armanini, the new model of Hübl & Holzinger will be included in the code. The model shows a robust behavior, gives rather accurate results and is still easy to use.

However the regulation of the maximum impact force is useless without giving further information about the load pattern. The load pattern will parallel to the maximum impact force be regulated in the ONR 24801. Fig. 7 shows load patterns of debris flow based on works by Wendeler et al. [30] and miniaturized tests by Hübl & Holzinger. On the right side the suggested load patterns are shown. The choice of rectangular or trapezium load pattern considers a possible debris flow in several waves filling up successive the deposit space in front of the debris flow barrier. Therefore the impact forces are climbing up the barrier. A parallel decrease of the impact forces at lower level has not been considered yet.

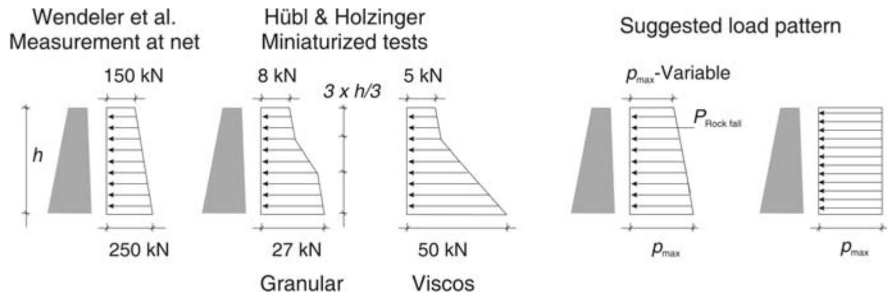

Fig. 7. Observed and suggested load patterns for debris flow impacts

Furthermore debris flows do not necessarily hit the barrier straight. Perhaps often the debris flow hits the barrier in a different angle. Therefore side impact forces have to be considered as well. Unfortunately in this field currently no tests or field measurement data is available. Overall a 1/5 up to 1/3 of the frontal impact force should be used. Besides impact forces also breaking forces of debris flows have sometimes to be considered, however they are not discussed here.

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A58**

Sensitivity of the Appraisal of Flood-alleviation Benefits to: Hydraulic Modelling, Vulnerability Assessment and the Coupling of Hazard with Vulnerability

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Abstract. The reliability of flood loss evaluation plays an important role on the flood management decision-making. The uncertainties of damage evaluation are mainly due to the absence of hazard, vulnerability and economic data. The methodology used to combine data is another source of uncertainty. In order to study the sensitivity of damage evaluation, this paper compares: (a) partial and final results of a complex 1D/2D hydraulic model in an urban area; (b) four different vulnerability assessment methods; and (c) three different GIS assumptions when coupling hazard with vulnerability. We make sensitivity tests to hydraulic, vulnerability and the coupling methodology uncertainties. The city of Holtzheim, situated within the low valley of the Bruche River, of eastern France, is taken as the study case. Direct damages to buildings and contents are calculated, and indirect damages are estimated for economic activities. The damage evaluation is extremely sensitive to all aspects studied.

Keywords: Cost-benefit, flooding, flood alleviation project, natural hazard, potential damages evaluation, sensitivity, uncertainty propagation

1 Introduction

Hydraulic engineering has long been employed to reduce flood damages. Flood management is based either on hazard and/or vulnerability reduction. The choice between different flood management strategies is generally made by comparing their forecast costs and gains. The study of the hydraulic parameters of the flood area is crucial for proposing management solutions. The assessment of the hazard spatial amplitude and the flow characteristics is essential to determine flood zones. In addition, different project scenarios can be compared in terms of hazard or vulnerability reduction. Therefore, hydraulic modelling is the first main step toward flood-alleviation project appraisal.

Decision-making process is still significantly based on cost-benefit analysis principals [1], taking the economic aspect as the most important factor in the process. Benefits are represented by potential avoidable damages. The costs concern project investments and maintenance, and sometimes, environmental impact costs. Flood damages evaluation is used to assess flood-alleviation benefits, or reveal the cost of the potential avoidable damages [2]. This evaluation is, therefore, a fundamental tool to support decision-making.

The evaluation of flood damages is based on three data: hazard data, data concerning the vulnerability of assets to flood and historical data related to damages suffered in the past [3],[4]. The objective of the evaluation is to estimate the potential damages correlated to hazards with different return periods. Some methods have been developed to assess intangible and indirect damages [5],[6]. Their application still requires great evaluation effort, and frequently these kinds of damages are estimated as a percentage of direct damages [7].

A classic method to assess direct damages consist of three steps: (1) data assessment - assess hazard, vulnerability and economic data; (2) data combination - the coupling between hazard and vulnerability serves to determine the risk; and (3) damages calculation - damages functions are used to establish the relationship between the flood parameters, the assets vulnerability and the expected damages [3],[4],[8]. Once damages are calculated for different hazard return periods, avoidable damages, or potential benefits of flood alleviation, can be compared according to different management scenarios.

Therefore, the reliability of the flood damages evaluation results plays an important role on the flood management decision-making [9]. The design and implementation of cost effective mitigation strategies, the reduction of the extent of loss and the development of compensation packages for flood victims are directly affected by the uncertainty on the damage evaluation results [10]. The results of the appraisal of flood-alleviation benefits are intrinsically dependent on this reliability. Therefore, the comparison between different projects alternatives risk do not represent the real gains of these projects. Multi-criteria or cost-benefits analysis used in these processes risk leading to false assumptions.

The uncertainties of these evaluations are caused mainly by absence of knowledge on hazard and vulnerability [11]. However, the methodology used to combine this data is also a source of uncertainty not mentioned before, to our knowledge. Concerning the hazard determination, several hydraulic models can be used to do simulate flooding. The reliability of the results of these simulations depends on different data used, on the model used, and on the technical capacity of the model operator. Many methods exist to assess vulnerability data [12],[13]. Different databases can be used in those methods. Different damages functions exist [14],[2],[15],[16], but the choice of the representative ones is a challenge in the damages evaluation process.

This paper tests the sensitivity of the economical appraisals of flood-alleviation benefits to: (section 2.1) the assessment of hazard parameters – we compare partial and final results of a complex 1D/2D hydraulic model in an urban area; (section 2.2) the method used to assess buildings and contents vulnerability – we compare four different vulnerability assessment methods; and (section 2.3) the approach used to combine hazard and vulnerability data – we compare three different GIS assumptions when coupling hazard and vulnerability. The city of Holtzheim (Fig. 1), situated within the low valley of the Bruche River, of eastern France, is taken as the study case. Direct damages to buildings and contents are evaluated and indirect damages to activities are estimated considering a percentage of direct damages.

The Bruche River, which runs through the city of Holtzheim, is the most susceptible to floods in the Urban Community of Strasbourg (CUS). The last major flood on was in 1990, a 30 year return period event, which has caused to the city of Holtzheim the equivalent to approximately 2.3 million Euros nowadays.

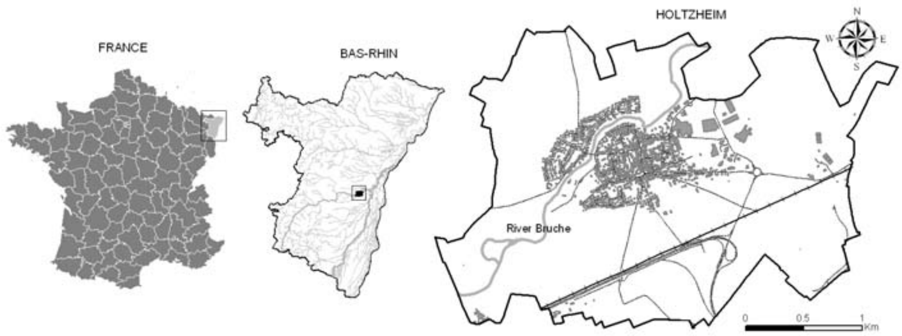

Fig. 1. City of Holtzheim, situated withi the low valley of the Bruche River (Bas-Rhin, France)

2 Methodology

The principals of the "Unit Loss Model" are used to evaluate direct flood damages to buildings and its contents: damages are calculated for each building touched by the flood [14]. The calculation depends on the hazard parameters inside the building and the buildings constructive characteristics and occupation type [7],[15],[16],[17]. Economic relationships between flood parameters and buildings characteristics allow us to calculate the expected damages for each building. Total damage caused by a flood with determined return period is calculated by summing up the damages for each building.

This evaluation is based on three classic steps: (1) to assess hazard and vulnerability data, (2) to combine hazard with vulnerability data (3) to use damage functions to calculate damages. We use existing depth-damage functions to evaluate flood loss: 17 damage functions are used to evaluate direct damages on buildings and their contents - two for residential damages $D_{residential}$ [15], and 15 for activities and public buildings damages $D_{activities}$ [7]. Indirect damages to activities are estimated as half of damages evaluated for the activities [7]. Total damage D is therefore calculated according to Eq. (1).

$$D = (\Sigma D_{residential}) + (\Sigma D_{activities}) \times 1{,}5 \qquad (1)$$

To compare the potential damages with project investments, an annual index is commonly calculated to represent the flood potential damages: Average Annual Cost (AAC). The AAC is calculated by summing the product of each potential damage associated to a known flood return period *D(q)* with the exceedance probability of the event of a given magnitude *F(q)* [15] (Fig. 2). In this study case, first damages were estimated for 5 years return period floods [18]. We considered that damage related to the theoretical infinity period return is equal to 150% of the damage caused by the 100 years return period [19].

We use different methods/approaches to realize the first two steps of the evaluation process: data assessment and data combination. We compare the results of the damages evaluation based on different hypotheses, in order to test its sensitivity to: the hydraulic modelling, the vulnerability assessment method and the coupling of hazard with vulnerability methodology. These hypotheses are showed subsequently.

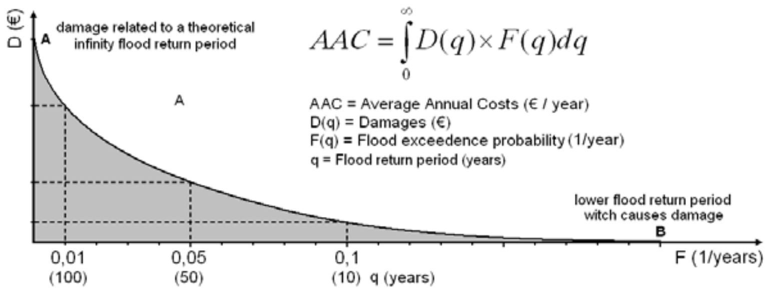

Fig. 2. Calculation of the Average Annual Cost (AAC) index

2.1 Hazard Assessment Hypothesis

Hydrologic and hydraulic models are largely employed to assess hazard data. The main interest of the use of these models is the possibility to simulate different floods with different return periods. In the flood damage evaluation context, the objective of these simulations is to determine spatial distribution of the flood characteristics. Flood depth is the main factor causing damages in urban areas [20],[21].

The hydraulic uncertainty is mainly related to the quality of the topographic data used to model the river system. However, uncertainty is also correlated to the complexity of the model used. Other uncertainty rarely mentioned is due to modelling errors and/or choices (model operator).

For assessing hazard data, we used the results of a three-year hydraulic/hydrologic study realized by the Danish Hydraulic Institute (DHI). DHI modelled all the hydraulic system of the Urban Community of Strasbourg (CUS), including the rivers Bruche and Ill and their main tributaries. Hazard modelling has been made using the Mike Flood software, by DHI. Flood in the main channel has been modelled with the 1D model Mike 11, and flood in the floodplain with the 2D model Mike 21. Three scenarios have been modelled: 10, 30 and 100 year return period floods. Water depth is represented by a grid with a resolution of 20x20 m.

In this study, we analyse the modelling results for a small section of the Bruche River, in the community of Holtzheim. We compare an intermediary result with the final result of the modelling process as explained thereafter.

Partial Hazard Determination concerns the results of the study in an intermediary phase, in spring 2008. The first results of the simulation were waiting for the verification and validation by field surveys. In this study we take these results as possible final results, making the hypothesis that these results could be the final results in case of fewer investments were done to the hydraulic modelling.

Final Hazard Determination concerns the final results of the study, finished in the beginning of 2009. The main modifications in the study area concerning the modelling process were topographic actualisation and corrections made in modelling choices related to the simulation of the bridge in the city of Holtzheim. According to local expert judgement, these results are closer to the actual phenomenon.

2.2 Vulnerability Assessment Hypothesis

The main objective of the vulnerability assessment is to assess information about the type of land-use in the flood area and its fragility to flood water. There are four different ways to study vulnerability to floods: (1) use existing databases; (2) realize field surveys; (3) ask for expert judgment; and (4) realize interviews. In order to compare different methods to assess the vulnerability of assets to floods, we compare 4 methods: Method A, B, C and D [22]. All assessment methods concern only buildings and contents characteristics, and two characteristics groups are considered to describe their vulnerability: constructive characteristics and occupation characteristics. The four methods used are based on different databases to explore these characteristics of the buildings in the flood area.

Method A is based on expert judgment, geographical information on land-use from a database represents the occupation type by 93 zone classes, e.g. industrial zone, commercial zone, residential zone, agricultural zone. This method allows us to assess the occupational type of a building. Therefore, this method does not allow the assessment of specific activity type, location of the activity inside the building, activity surface, or buildings constructions characteristics.

Method B is based on expert judgment and geographical information concerning economic activities from two databases which allow us to identify the specific type of activity per building, considering residential occupation for the others. However, the method does not allow the identification of the location of the activity inside the building, activity surface or the buildings constructions characteristics.

Method C is based on data acquired from a superficial field survey in which the information concerning occupation characteristics is researched building per building. However, the information concerning constructive characteristics is researched by homogeneous areas. All buildings characteristics are assessed using this method.

Method D is based on data acquired from a detailed field survey. In this field survey, all building's characteristics, occupation type and constructive characteristics are obtained individually, building per building. Among the 4 methods, this method is the most reliable one [22].

2.3 Hypothesis When Combining Hazard with Vulnerability Data

Once we obtain data describing hazard, and data describing vulnerability, we must combine this information in order to analyse the risk. Although this operational step is to our knowledge rarely discussed, this step is crucial for the risk quantification, and the hypothesis made here plays an important role in the forecast of the water depth inside the buildings.

When overlaying hazard data with vulnerability data, we have the same building overlaid by different hazard characteristics. The question of which value could better represent the real phenomenon can be answered in different ways. We can consider 3 hypotheses to analyse this combining process. Water depth inside the buildings can be calculated given: (1) minimum values of water depth touching the buildings; (2) mean values of water depth touching the building; or (3) maximum values of water depth touching the building. Therefore, it is hard to say which hypothesis represents the best the phenomenon. The example (Fig. 3) represents the data combination process according to these different hypotheses (this example considers that the first floor elevation of the buildings in relation to the natural ground is null).

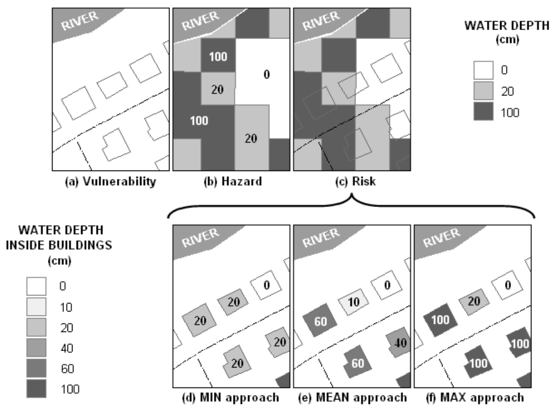

Fig. 3. Three different approaches used to combine hazard with buildings vulnerability data

MIN Approach: when overlaying flood water depth with the buildings shapes, we consider that the minimum value of water depth touching the building (except by the zero value) is the water depth inside the entire buildings area (Fig. 3, d).

MEAN Approach: when overlaying flood water depth with the buildings shapes, we consider that the mean value of the water depths touching the building is the water depth inside the entire buildings area (Fig. 3, e).

MAX Approach: when overlaying flood water depth with the buildings shapes, we consider that the maximum water depth value touching the building is the water depth inside the entire buildings area (Fig. 3, f).

3 Results and Discussions

We evaluated flood potential damage for the different hypotheses and approaches used to assess and combine data. The results of the damage evaluation are displayed according to the different hazard determinations: the partial hazard determination (Fig. 4), and the final hazard determination (Fig. 5).

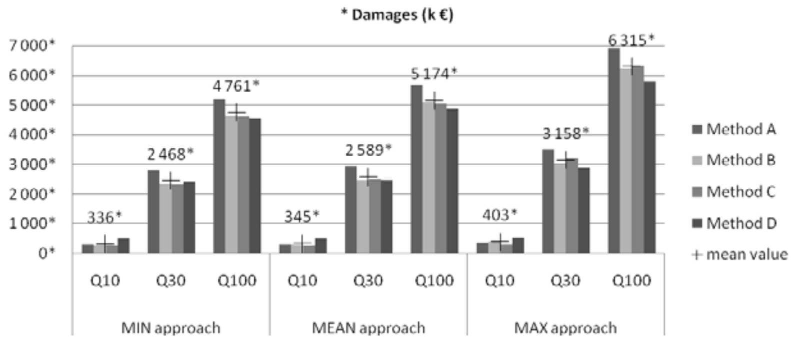

Fig. 4. Damages according to the partial hazard determination and different hypothesis

When comparing the damage evaluation according to the two different hazard determinations, we notice that damages are over-evaluated using the partial hazard determination (Fig. 4) in relation to the final hazard determination (Fig. 5), especially for the flood with return period equal to 30 years (Q30). The values calculated for the flood Q30 based on the partial hazard determination (Fig. 4) are 65% higher (on average) than damages based on the final hazard determination (Fig. 5). For the flood Q100, average damage is 19% higher, and for the flood Q10, average damage is just 5% higher.

The same variation pattern is observed between the different methods used to assess vulnerability (Methods A, B, C and D), when comparing the floods with the same return period for the different hazard determinations, and for the different approaches used to combine data (MIN, MEAN and MAX). This is true, except for the flood Q30. In the Q30 flood the damages are slightly under-evaluated when using method A in relation to method D, according to the final hazard determination (Fig. 5). In general, damages are evaluated similarly when using methods B and C to assess vulnerability.

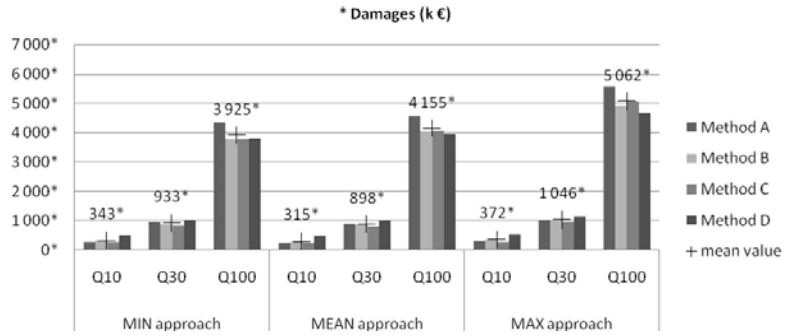

Fig. 5. Damages according to the final hazard determination and different hypothesis

We compare the results in terms of Average Annual Cost (Fig. 6), in order to test the sensitivity of this index used in cost-benefit analysis.

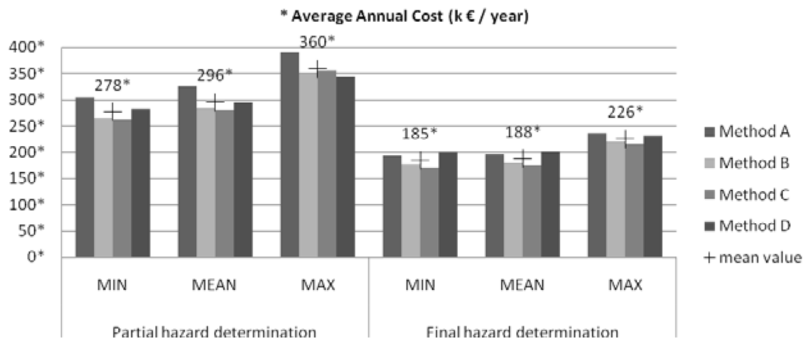

Fig. 6. Average Annual Cost of flood damages according to the different methods and approaches used to assess and combine data

When comparing the 2 different hypotheses in hazard determination, we notice that the partial determination (Fig. 6, left) over-estimates damages in relation to the final determination (plus 36 ± 2% on average) (Fig. 6, right). The same variation pattern is observed between methods A, B and C, used to assess vulnerability. However, when comparing these 3 methods with method D, the most reliable one, we notice that the individual characteristics of the buildings (only revealed by method D) play an important role in the damage determination, explaining the great variability of the results of the evaluation using method D.

The mean values for the different methods used to assess vulnerability (A, B, C and D) reveal that the evaluation results are quite similar when using the MIN approach and the MEAN approach to combine data (MIN approach under-estimated in 4% on average in relation to MEAN approach). However, damages are highly over-estimated when using the MAX approach (plus 17% on average in relation to MEAN approach).

Damages in terms of Average Annual Cost (AAC) using the less reliable data and methods (partial hazard determination and Method A to assess vulnerability) are over-estimated (plus 61% on average concerning the different approaches used to combine data) in relation to damages evaluated by mean of the most reliable methods and data (final hazard determination and vulnerability data assessed by the Method D) (Fig. 6). The higher result in terms of AAC (390 k€/year, using the partial hazard determination, method A to assess vulnerability and the approach MAX to couple data - Fig. 6, left) is 128% superior to the lower result (171 k€/year, using the final hazard determination, method C to assess vulnerability and the approach MIN to couple data - Fig. 6, right). These results prove that the choice of the methods used to assess and combine different data plays a fundamental role in the results of the evaluation of flood loss.

4 Conclusions

In this study, we tested the sensitivity of damage evaluation results to three aspects of the evaluation process: hazard modelling, vulnerability assessment method and the methodology used to couple hazard with vulnerability data. The results of damage evaluation of the city of Holtzheim are extremely sensitive to all aspects investigated. Hazard determination, as a result of hydraulic modelling, has generated more uncertainty than the others parameters tested. The method used to assess buildings' vulnerability and the approach used to couple hazard with vulnerability data are also important steps in the evaluation. When comparing damages evaluated in terms of Average Annual Cost, the highest values are up to 128% superior to the lowest values, depending on the methods and approaches used.

The main expectation of this paper is to reveal some uncertainties of flood loss evaluations. There is a need to better define standard methods to assess flood damage before using these results as decision-making criteria. This study is limited to the evaluation of buildings' tangible direct damages. Intangible and indirect damages can represent a great share of total damages [23], and must be integrated in damage evaluation process. The continuation of this study aims at testing damage evaluation sensitivity to other types of damages. We also seek to test the sensitivity of flood-alleviation benefits appraisals to the choice of the hydraulic model and to time allocated.

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A59**

Experience from Numerical Modelling of Bedload Transport at Steep Slopes

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Abstract. A sediment routing model for steep torrent channel networks called SETRAC has been developed and extensively tested. SETRAC has been applied to well document case studies on flood events in torrential catchments with substantial sediment transport in the Austrian and Swiss Alps. The influence of form roughness, armouring and transport by size fractions on modelling bedload transport is discussed. The simulation results show the importance of form roughness losses when computing bedload transport in torrents and mountain streams. Neglecting from roughness losses results in an overestimation of the observed bedload transport by a factor of about 10 on average.

Keywords: Numerical modeling, bedload transport, torrent, mountain stream, step slope

1 Introduction

Modelling bedload transport in steep headwater channels is rather challenging, as flow data - generally needed for the calibration of a hydraulic model - are mostly absent in small catchments. The input hydrographs, necessary for the simulation model, need to be generated with a rainfall runoff model and calibrated with measured or reconstructed hydrographs. Flood marks at cross-sections without morphological change have to be used for the recalculation of the peak discharge if no stream flow measurements are available. An important hydraulic model parameter, the roughness coefficient, is typically estimated by a approximate techniques [1], because no measurements for high discharges are available. The space occupied by the transported sediment is non negligible for flood events with substantial bedload transport at steep slopes [2]. For the calibration of the sediment transport model additional data about morphologic changes can be obtained by field investigations or by remote sensing techniques (e.g. differential elevation models obtained by LiDAR). In the case of a flood event a movable-boundary, unsteady-state calibration [3] is required for computational modelling. If a real model calibration is not possible due to the lack of adequate calibration data, Thomas and Chang [3] suggest computational analysis. Computational analysis is the application of a computational model to a problem in which a reliable model calibration is not possible. For the case of a flood reconstruction with bedload transport at steep slopes measured water levels are used for the back-calculation of the peak discharge and therefore they cannot be used for the calibration of the hydraulic part of the model. Hence the simulated water levels are obtained by estimates of the channel roughness and therefore are not very reliable. If a Schoklitsch-type bedload transport equations is used for the calculation of bedload transport at steep slopes [4], no flow depths have to be estimated. Hence modelling bedload transport at steep slopes and recalculations of sediment laden floods can be regarded as computational analysis.

2 The SETRAC Model

A one dimensional sediment routing model for steep torrent channel networks called SETRAC [5], [6] has been developed at University of Natural Resources and Applied Life Sciences, Vienna. SETRAC is the acronym for Sediment Transport Model in Alpine Catchments. Different sediment transport formulas [2], [7], [8], [9] and flow resistance approaches [5], [9], [10] can be selected in SETRAC for steep channel slopes. To take form roughness losses into account several approaches are available to modify the calculated transport capacity to better match observations on bedload transport. Armouring effects can also be considered. In addition it is possible to calculate fractional bedload transport and considering grain sorting effects in combination with mobile bed conditions. In SETRAC the channel network is represented by nodes, cross-section and sections. The sediment is transferred through the channel network considering sediment budget in sections. Initial erosion depth can be assigned for each channel reach. Morphologic changes due to erosion and deposition are calculated. A graphical user interface with visualizations of the longitudinal sections as well as the cross-sections has been developed. For calculation the cross-section is divided into strips to get a representative discretization of the profile. The number of strips depends on the number of points that are used to specify a cross-section, implying that the number of strips increases with the complexity of the cross-section (Fig. 1).

Flow hydrographs are routed as kinematic wave through the channel system. Sediment input as sedi-graphs is also possible. All simulation results can be stored as longitudinal profiles as well as time series for all cross-sections in user defined time intervals as text files for further analysis. The last time step with information about the highest values can also be exported as DXF file for plotting the results for engineering applications.

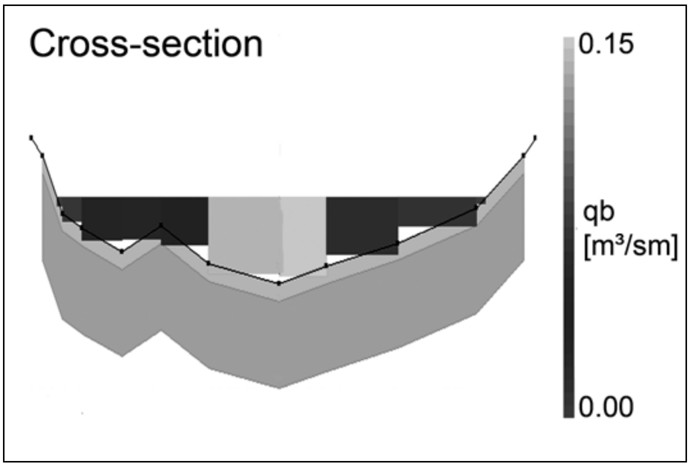

Fig.1. Structured cross-section with visualization of the specific bedload transport in the strips

2.1 Note on Availability of the SETRAC Model

The SETRAC model as well as a user guide, a technical manual and tutorials will be available soon as free download at www.alpine-naturgefahren.at.

3 Application of SETRAC to Well Documented Case Study Streams

Many regions in Austria, Switzerland and Germany were affected by the flood events from August 2005. A massive cyclone over the northern part of Italy caused heavy rainfall particularly from 21-22 August 2005. The period of relevant precipitation was about 4 days, whereas thunderstorms were not of major importance. Most events in Austria occurred in the western provinces Vorarlberg and Tyrol. In Switzerland the whole north-alpine region was affected by long-lasting precipitation. Southern parts of Bavaria in Germany were also affected by flood events.

After the extreme events of August 2005 a general event documentation was made for Austria [11]. The flood events in several torrents were documented by the Institute of Mountain Risk Engineering at the University of Natural Resources and Applied Life Sciences, Vienna [12]. Detailed investigations including a reconstruction of the flood hydrograph and the sediment budget along the whole channel were made for the Sessladbach and Schnannerbach in Tyrol and for the Suggadinbach in Vorarlberg. In Switzerland detailed investigations for the Chirel mountain stream, the Chiene mountain stream and the Schwarze Lütschine mountain stream are available. A detailed documentation and analysis of the Swiss flood events was prepared [13], [14]. Severe damage occurred along most of the affected torrents and mountain streams, because of massive bedload transport during the flood event.

4 Discussion of the Simulation Results

All mentioned events are back-calculated with the SETRAC model and presented in detail in [6]. For a systematic comparison of different case studies a simulation design was used for all field data. All simulations consider supply limited conditions. Possible erosion depths are estimated in the field and range from 0.05 m in channel reaches mainly in bedrock to 10 m for reaches in alluvial bed sediment. Morphologic changes due to erosion and deposition are considered as non negligible. Therefore the erosion module is activated within SETRAC. Due to the sensitivity concerning the spatial discretization, the reach length has been refined for each case study until there was no more significant change in the total transport over the whole length of the main channel. Then the coarsest discretization without loss of accuracy has been chosen for the different simulation runs.

For steep streams bedload transport equations of Schoklitsch [15] were recommended [4], [16]. Therefore a Schoklitsch type equation [8] is applied. Losses due to form roughness are either neglected or considered with different approaches. The influence of armour criteria [17] in combination with the one grain model on the total amount of transported material is investigated.

4.1 Agreement between predicted and observed loads

The back-calculations of the August 2005 extreme events with the SETRAC model indicate that form roughness losses are non negligible when modelling bedload transport in steep headwater streams. For channel slopes steeper than about 0.05, bedload volumes calculated with a bedload transport equation are about an order of magnitude larger on average than the observed volumes, suggesting that a correction for form resistance losses may be important [18].

Apart from limited sediment supply form roughness losses can be regarded as an important reason why transport formulas often overestimate bedload transport when they are applied to channels where the effect of bed forms on flow resistance and sediment transport can not be neglected. Palt [19] accounted for form losses and found in this case much better agreement between his bedload measurements in Himalayan rivers and the bedload transport formulas [20], [9] and [8].

To quantify the overestimation of the bedload transport equation when applied to field data the transport capacity of each cross-section was calculated. For this comparison morphologic changes of the cross-section geometry and of the channel slope between the cross-sections were neglected. To model unlimited supply conditions, the possible erosion depth was set to the high value of 100 m. For comparison with the reconstructed bedload volumes time integrated bedload volumes are calculated with SETRAC. For further analysis the ratio of the simulated versus the reconstructed (observed) bedload transport is plotted in Fig. 2 for the field cases for each channel reach. This comparison of the transport capacity and the recalculated bedload transport can not distinguish between limited sediment supply and reduced transport capacity due to form roughness losses. For steeper torrents (Sessladbach & Schnannerbach) limited sediment supply may be more important than for mountain streams, where generally more sediment is stored in the channel bed. With the exception of some reaches, the observed bedload transport is overestimated by one to three orders of magnitude. A power law trend line for all data is shown in Fig. 2.

If only a limitation of the sediment stock in reaches to model "supply limited" conditions is considered, this cannot reproduce the time evolution of the recalculated extreme events and results in a clear overestimation of the total sediment load. The hydrograph as well as the sedigraphs of the simulation without form roughness losses as well as for the simulation with consideration of form roughness losses are shown in Fig. 3 for the Sessladbach field study. The sedigraph peaks before the hydrograph, because sediment delivery is limited from the upstream reaches. The temporal evolution of the bedload transport can not be reconstructed in agreement with the observations of the inhabitants of the fan without consideration of form drag. After 17 hours of simulation time all the sediment stock is depleted. Considering form drag the temporal sequence of the event with simultaneously peaking hydro- and sedigraph and the total transported volume can be much better reproduced.

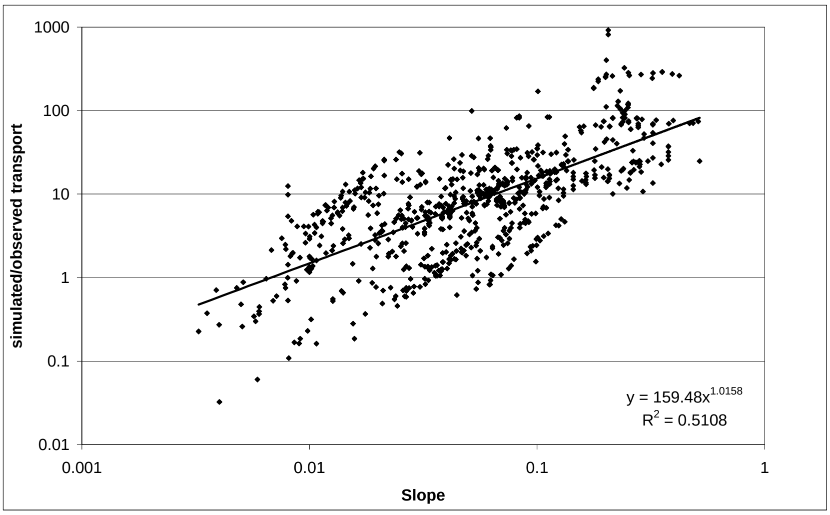

Fig. 2. Ratio of transport capacity and reconstructed bedload transport for all simulated torrents and mountain streams

4.2 Effect of Armour Layer in Combination with the one Grain Model

For sediment routing calculations in gravel-bed rivers armouring has to be considered [21]. An armour criteria as implemented in SETRAC may be regarded as upper boundary (resulting in a lower limit for bedload transport volume estimates), because the incipient motion criteria is increased for the whole simulation time.

The relation between discharge and the bedload effective water volume V_{re} is illustrated in Fig. 4. Where $V_{re,1}$ is the effective water volume considering a critical discharge and $V_{re,2}$ is the effective water volume considering a critical discharge in combination with an armour layer criteria. In natural rivers, once the armour layer is broken up the incipient motion may be reduced because of the destroyed armour layer (shown by the dashed line in Fig. 4), or effectively a mobile armour layer may form.

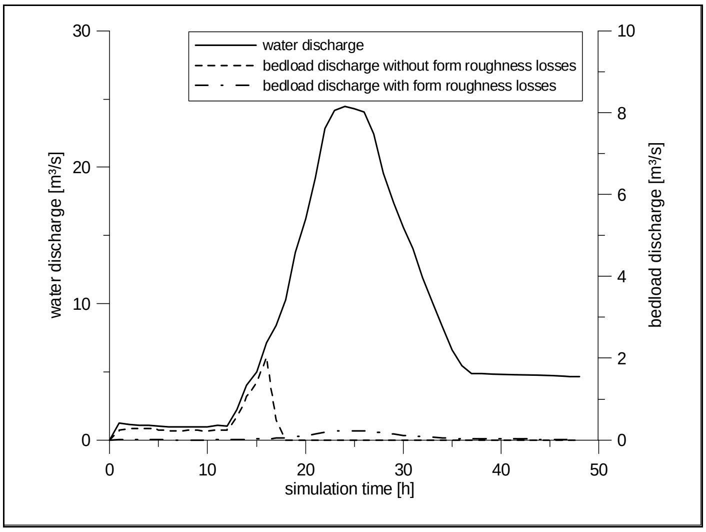

Fig.3. Hydrograph and sedi-graphs for the channel outlet of the Sessladbach

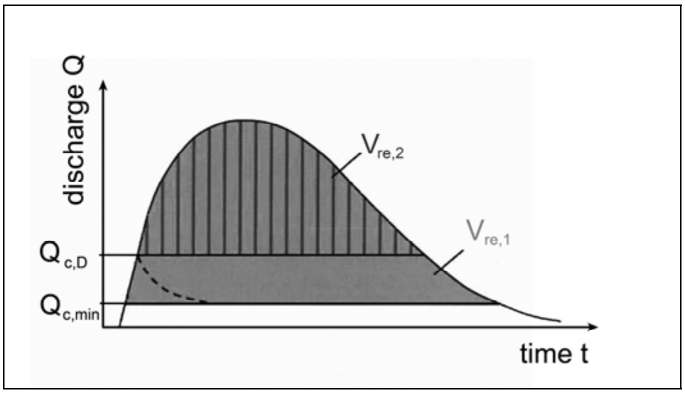

Fig.4. Relation between discharge and effective water volume after Badoux&Rickenmann [22]

The simulation results of the mountain streams modelled within this study show that considering armouring, the total bedload transport is reduced. About 10% to 20% less bedload is transported compared to simulations neglecting armouring in mountain streams.

However for the steep torrents of our study, the application of the incipient motion criteria due to armouring still results in an overestimation of observed bedload transport volumes. Therefore the consideration of simple armour criteria in combination with a one grainsize model cannot reproduce the recalculated bedload transport. Form roughness losses appear to be more important at very steep slopes. For torrents, where also colluvial sediment makes up the channel bed, the concept of armouring is questionable. Stable bed-form structures like step-pool systems are more likely to develop.

4.3 One Grain-Size Model versus Fractional Bedload Transport

Considering the main purpose of SETRAC, developing a sediment transport model for application in torrents and mountain streams dueing flood events, the one grain-size model appears to be adequate to describe the transport processes in steep head-water catchments. For mountain rivers with low to moderate discharges the effect of armouring appears to be non negligible, but further research is necessary to understand the processes of downstream fining or even downstream coarsening [23], [24] of the active layer at steeper slopes. Parker et al. [25] discussed the behavior of the grain size distribution of gravel-bed rivers during floods and concluded that the surface size distribution present during floods may differ little from that prevailing at low flow. The main problem is that the grain size distribution during high floods can not be sampled easily. Once the armour layer is broken, essentially all sizes are roughly equally mobile and therefore not much extra accuracy is obtained by calculating bedload transport for each size fraction separately and summing [26]. Computing fractional bedload transport at steep slopes requires more data for calibration. The change of the grain size distribution due to selective transport is dependent on the hiding function and the active layer thickness. Vertical mixing in gravel bed rivers has been investigated [27] and it was found that the burial depth of particles is dependent on the magnitude and duration of the flow event. In addition the number of events and the surface structure and texture influence vertical mixing. Hence the determination of the active layer thickness is not a priori known and can be used as a calibration parameter to adopt the rate of change in the grain size distribution in combination with the hiding function. Concerning lowland rivers, more data on selective transport are available. Therefore a calibration of a fractional bedload transport computation seams more reasonable than for torrents and steep mountain streams. The application of fractional bedload transport calculations to strongly aggrading river reaches shows a better agreement with the observed behavior [28]. For the application of fractional bedload transport equations to steep mountain streams and torrents further research is required, to understand selective transport mechanisms at steep slopes.

4 Conclusions

The back calculations of well documented extreme events in Austria and Switzerland stress the importance to consider form roughness losses. For the studied streams and flow events, neglecting form roughness at steep slopes resulted in overestimation of the observed bedload transport by about a factor of 10 on average. If only the sediment stock in reaches is limited to represent supply-limited conditions, the time evolution of some extreme events cannot be reproduced, because all the sediment stock is depleted during the raising limb of the hydrograph, which is contrary to observations. A simple armour layer criterion in combination with the one grain model is not sufficient in many case studies to reduce the transport capacity during flood events, particularly at steep channel slopes. Quantification of form roughness losses appears to be important to describe transport processes during flood events in steep channel systems. The quantification of losses due to form drag is challenging due to a scarcity of detailed field measurements. Depending on the channel slope, the relative contribution of form roughness to total roughness may range from 50 to 90% for natural streams. Further investigations are required to develop more reliable approaches to more reliably estimate form roughness losses for the calculation of bedload transport at steep slopes during extreme events.

Acknowledgement

The authors would like to express their gratitude to the Austrian Science Fund (FWF) for their financial support through the Translational Research Program L147 on "Sediment routing model for steep torrent channels" which enabled this work to be undertaken.

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A66**

Modelling of Wave Interaction with Submerged Breakwater Using MIKE 21 BW

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Abstract. Submerged breakwater are simple constructions which attenuate wave energy in area beyound breakwaters. Appliance of such constructions can achieve multiple benefits like coast erosion reduction, cheapest coast constructions, overtopping reduction, force reduction etc. Also, the great contribution from cultural point is preservation of old historical town sights and landscapes. Submerged breakwater design requires determining of transmission energy amount, which defines the protection level of aquatory or constructions behind breakwater.
Irregular waves transmission over submerged breakwater was analyzed in numerical model, MIKE 21 BW 1D. Results from this model were compared with laboratory measurements in physical model tests (Johnson, 2005), also with empirical models (DArgenmond, 1996, Seabrook, 1998, Buccino, 2007). In model MIKE 21 BW 1D, wave breaking was calculated with modified breaking angle parameters of surface roller concept. Transmission coefficients obtained from numerical model have shown good agreement with measured data and empirical model data. Also, transmission of wave period (mean and significant) was analized with numerical model for different wave parameters and geometrical parameters of brekawater.

Keywords: Submerged breakwater, Boussinesq model, surface roller, transmission coefficient, wave period, wave height

1 Introduction

With development of wave numerical models, modelling of wave transmission over submerged breakwaters becomes a challenge and the criterion of the quality of the numerical model. Wave transformation at arival into shallow water has been so far rather well described by various numeric models, while the process of wave deformation from shallow into deep water, so-called ¨deshoaling¨ is still the subject of scientific efforts in numerical modelling. Deshoaling effect includes complex non-linear interactions, i.e. transition of wave energy into lower and higher harmonics. Numerical model (MIKE21-BW, [1]) has been verified in the works [2], and [3]. For verification, the results of laboratory tests conducted in [4] were used. Laboratory tests were carried out on a submerged breakwater with mild slopes (ofshore slope 1:20, shore slope 1:10). Verification was done by comparison of measured and calculated wave profiles, for a narrow band of wave parameters.

This paper will demonstrate the application of MIKE21-BW-1D for calculation of the transmissmited wave parameters over the submerged breakwater. Possibility of application would be presented in two steps:

Transmission coefficients of wave heights. a) Transmission coefficients of wave heights calculated by MIKE21 model will be compared with the published transmitted coefficients (Table 1) from laboratory tests, published in the paper [5] for irregular waves (in Chapter 2.1). b) Also, comparison of transmission coefficients calculated with MIKE21 and with empirical models will be shown, (D'Argenmond, 1996, [6], Seabrook, 1998, [7], Buccino, 2007, [8]). (in Chapter 6.2), for wave parameters from Table 2.

Transmission coefficients of wave period. Transmission coefficients of wave periods calculated by MIKE21 model will be presented (in Chapter 6.3). Characteristic statistical and spectral wave periods are compared (in Chapter 6.4).

1.1 Laboratory Experiment

Laboratory experiments were done in the basin 9.7x12 m in the laboratory of Aalborg University, Denmark, [1]. The investigation measured the wave setup, transmission coefficients and flow around the breakwater. Out of all investigations, this paper will use the results of measuring of transmission coefficients of wave heights.

Fig. 1 shows the bathymetry used for laboratory experiments. The breakwater has a 2 m opening in the middle. Two berm widths were used, wide 0.6 m, and narrow 0.2 m. Points from 1 to 21 mark the measuring probes.

Breakwaters were made with the core and armour layer of nominal particle size (Dn_{50}=45mm), and side slope 1:2. Behind the breakwater, the beach was made for dissipation of waves, of quarry rock (Dn_{50}=15mm), slope 1:5.

Experiments were carried out for the submerged breakwater, emerged breakwater and zero freeboard breakwater. This paper will deal only with the data referring to the submerged breakwater. Water depth on the breakwater berm is 7 cm, depth at toe is 27 cm, and depth at the wave generator is 43 cm. Tests were carried out for regular and irregular waves. Irregular waves were generated as the JONSWAP spectrum with direction spreading of 22.7º. This paper uses only the results of transmission coefficients for irregular waves.

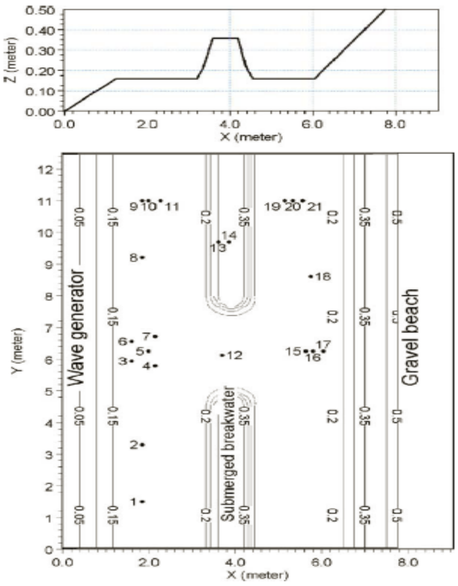


Fig. 1. Cross-section and floor plan of basin with wide breakwate crest 0.6 m

Transmission coefficients (Table 1), are determined as the relation of transmitted significant wave height, Hs_{0t}, and incident significant wave height, Hs_{0i}. Hs_{0t} is determined as the average of measured significant wave heights in gauges 19, 20 and 21, (Fig. 1). Hs_{0i} is determined as the average of measured significant wave heights in gauges 9, 10 and 11.

Table 1. Program of laboratory experiments and measured coefficients of transmission K_t. Hs_0-incident significant wave height, T_p-peak period, d-water depth on the toe, [1]

Test	Hs_0	T_p	d/L_p	wave type	berm	L_p/Hs_0	K_t-meas
33	0.12	1.97	0.04	J 3D	wide	50.03	0.52
17	0.12	1.97	0.04	J 3D	narrow	50.03	0.62
34	0.12	1.40	0.09	J 3D	wide	25.17	0.58
18	0.12	1.40	0.09	J 3D	narrow	25.17	0.69
21	0.05	1.32	0.10	J 3D	narrow	50.34	0.81
35	0.05	1.32	0.10	J 3D	wide	50.34	0.73
22	0.05	0.93	0.20	J 3D	narrow	24.99	0.76
36	0.05	0.93	0.20	J 3D	wide	24.99	0.74

1.2 Numerical Model MIKE21-BW

The numerical model is set up with identical conditions as the longitudinal cross section of laboratory model (Fig. 2). Numerical model was enlarged by 20 times, because calibration of parameters MIKE21-BW was made for realistic wave conditions. In other words, bathymetry, wave heights and wave lengths were enlarged by 20 times. Grid spacing used was Dx=1m, time step Dt=0.02s.The waves were generated as the Jonswap spectrum, and the period of calculation is 10 min, for achieving of stationary conditions.

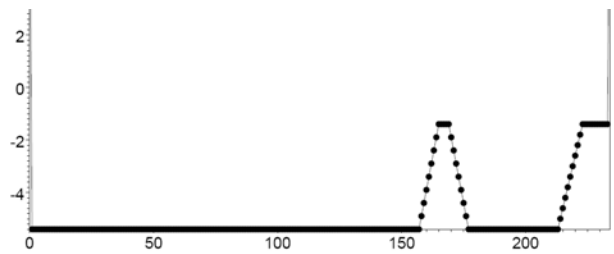


Fig. 2. Bathymetry with narrow breakwater berm used for numeric model.

Table 2. Wave parameters for comparison 2).

Test	Hs [m]	Tp [s]	berm [m]
1	3,0	4,9	WIDE
2	3,0	4,9	NARRO
3	2,4	8,8	WIDE
4	2,4	8,8	NARRO
5	2,4	6,3	WIDE
6	2,4	6,3	NARRO
7	2,4	4,8	WIDE
8	2,4	4,8	NARRO
9	2,1	4,5	WIDE
10	2,1	4,5	NARRO
11	2,1	5,8	WIDE
12	2,1	5,8	NARRO
13	2,1	8,1	WIDE
14	2,1	8,1	NARRO
15	1,5	3,8	WIDE
16	1,5	3,8	NARRO
17	1,5	4,9	WIDE
18	1,5	4,9	NARRO
19	1,5	6,9	WIDE
20	1,5	6,9	WIDE
21	1,1	3,2	NARRO
22	1,1	3,2	WIDE
23	1,1	5,9	NARRO
24	1,1	5,9	WIDE
25	1,1	4,2	NARRO
26	1,1	4,2	WIDE
27	0,8	2,8	NARRO
28	0,8	2,8	WIDE
29	0,8	3,6	NARRO
30	0,8	3,6	WIDE
31	0,8	5,1	NARRO
32	0,8	5,1	WIDE

Bed friction is defined according to the theory presented in [9]. Bed roughness parameter, k_N, for calculation of the wave friction factor is calculated as $k_N=2.5 \cdot D_{n50}$. The wave friction factor is limited to the maximum value $f_{wmax}=0.8$, and the mean velocity and the velocity at bottom were calculated according to the linear wave theory. The Manning friction coefficient is determined according to the mentioned theory, and with the given parameters, for water depth equal to mean height of the breakwater, and applied as constant along the entire length of the breakwater.

Wave breaking in the MIKE21-BW model is calculated according to „roller concept“. Standard set parameters of wave breaking were used, initial breaking angle $F_b=20°$; final breaking angle $F_0=10°$, roller form factor, 1.5, roller cellerity factor, 1.3, half time, $T_p/5$). Also for comparison, the calculation was made with recommendations for submerged breakwaters according to [2], $F_b=14°$ i $F_0=7°$, others remaining the same.

The sponge layer was placed o the left side of bathymetry, and on the right end. For comparison 1b), described in introduction chapter, calculation was made in MIKE21 according to wave parameters in Table 2. Wave parameters in Table 2 are limited by stability of the model; i.e. at large wave heights and lengths in relation to depth of water at the breakwater, the model becomes unstable. The results of the numeric model according to Table 2 are compared with empirically obtained transmission coefficients of wave height in Chapter 6.2.

1.3 Empirical Models

The empiric equations were obtained on the basis of laboratory tests of hydraulic behaviour of the breakwater, and are used to calculate average transmission coefficients of wave heights, K_t, for given wave parameters and geometric paramaters of the breakwater.

Seabrook and Hall, 1998:

$$K_t = 1 - \exp\left(-0.65\frac{F}{H_{si}} - 1.09\frac{H_{si}}{B}\right) + 0.047\left(\frac{BF}{L_p D_{50a}}\right) - 0.067\left(\frac{H_{si}F}{BD_{50a}}\right) \qquad (1)$$

where: F- freeboard [m], H_{si} - significant wave height [m], B- berm [m], L_p- peak wave lenght [m], Dn_{50} - nominal diameter of armour layer [m].

D’Angremond, 1996:

$$K_t = -0.4\frac{F}{H_{si}} + 0.64\left(\frac{B}{H_{si}}\right)^{-0.31} \times \left(1 - e^{-0.5\xi}\right), \qquad B/H_i < 8 \qquad (2)$$

$$K_t = -0.35\frac{F}{H_{si}} + 0.51\left(\frac{B}{H_{si}}\right)^{-0.65} \times \left(1 - e^{-0.41\xi}\right), \qquad B/H_i > 12 \qquad (3)$$

where: $\xi = tg\alpha/(\frac{H_{si}}{L_p})^{0.5}$ -Irribaren number. For values $8 \le B/H_{s_i} \le 12$, the values of transmission coefficient are interpolated linearily.

Buccino, 2007:

$$K_t = \frac{1}{1.18\left(\frac{H_{si}}{F}\right)^{0.12} + 0.33\left(\frac{H_{si}}{F}\right)^{1.5}\frac{B}{\sqrt{H_{si}L_p}}} \quad \text{for} \quad 2 \ge \left(\frac{F}{H_{si}}\right) \ge 0.83 \tag{4}$$

$$K_t = \left[\min(0.74;\ 0.62\cdot\xi^{0.17}) - 0.25\cdot\min\left(2.2;\frac{B}{\sqrt{H_{si}L_p}}\right)\right]^2 \text{for}\left(\frac{F}{H_{si}}\right) = 0 \tag{5}$$

For values $0.83 \rangle F/H_{si} \rangle 0$, the values of transmission coefficient of wave heights are linearily interpolated.

2 Results

To enable comparing of results, the following statistic parameter was used:

Mean square error (MSE). It represents dispersion of data around the line of “absolute agreement”.

$$MSE = \sqrt{\frac{\sum(y-\hat{y})^2}{n}}, \tag{6}$$

where: y is actual value (K_t measured in laboratory or empirical model), $\hat{y}$ is estimated value of y (K_t calculated by numeric model), and n is number of comparated transmission coefficients.

2.1 Transmission Coefficients of Wave Heights-Comparison of MIKE21 and Laboratory Measuraments

This paper compares transmission coefficients obtained by laboratory experiments for Jonswap waves with directional dispersion (22.7º), and transmission coefficients obtained by 1D numeric model without directional dispersion. As wave breaking is the dominant process influencing dissipation and transmission of wave energy, and does not depend essentially on the incident wave angle, it is assumed that 1D numeric model will describe the transmission of wave energy well enough. Also, wave diffraction through the opening in the breakwater is neglected, as the gauging probes 19, 20 1nd 21 are deep in the shadow of the breakwater (Fig. 1).

The comparison will be shown between transmission coefficients obtained by numeric model and by measuring in the laboratory (Fig. 3). Transmission coefficients on the numeric model ($K_{t\ MIKE}$) are calcualted in the identical points as those in laboratory experiment ($K_{t\ measur}$).

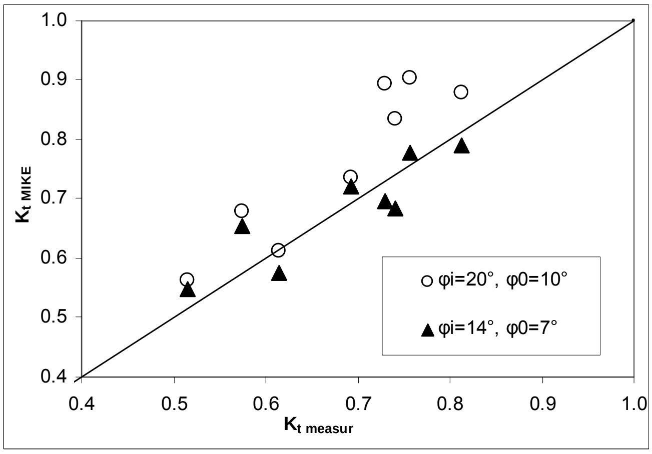


Fig. 3. Comparison of transmission coefficients obtained by numeric model $K_{t\ MIKE}$, and by laboratory tests $K_{t\ measur,}$ for different initial (F_i) and final (F_0) wave breaking angles. $MSE_{(Fb=14°;F0=7°)}$=0.05; $MSE_{(Fb=20°;F0=10°)}$=0.09

The results match better for applied breaking angles F_b=14° and F_0=7°, as recommended in the paper [2]. Calculated mean square error is $MSE_{(Fb=14°;F0=7°)}$=0.05. For standard breaking angles (F_b=20°, F_0=10°) numeric model overestimates the transmission coefficients, and mean square deviation is $MSE_{(Fb=20°;F0=10°)}$=0.09. When smaller initial and final breaking angles are used, the waves start to break earlier than in the case of standard parameters. This increases dissipation of wave energy for all tests, and the largest influence is exerted on waves with lower wave heights. (Test 21, 35, 22 i 36). Matching of transmission coefficients is satisfactory for applied breaking angles F_b=14° and F_0=7°.

2.2 Transmission Coefficients of Wave Heights-Comparison of MIKE21 and Empirical Models

Numeric model was set up as described in Chapter 4. Due to reflection from the breakwater, incident significant wave height, H_{si}, is defined at sufficient distance from the breakwater to avoid influence of reflection. The averaging zone from point 100 to 120, Fig. 4 (right), was defined where there is no influence of reflection even at longest waves. In this section, the values of relevant wave heights were averaged. H_{st} was obtained by averaging on the section from point 190 to 200.

Fig. 4 (left), presents the comparison between transmission coefficients obtained by numerical model and by empirical equations for wave parameters given in Table 2.

Transmission coefficients were calculated like: $K_t=H_{st}/H_{si}$. The best agreement of the numerical model is that for D'Angremond empirical equation, while according to the other two equations, Seabrook and Buccino, the numeric model slightly underestimates the results.

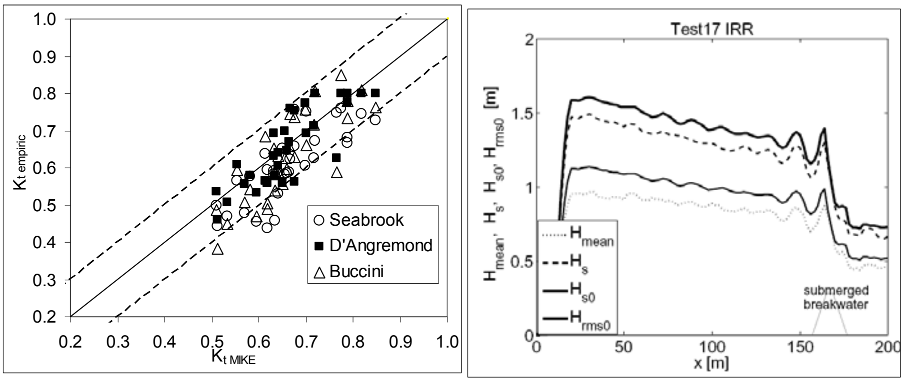


Fig. 4. Left: Comparison of transmission coefficients obtained by empirical equations and by numeric model. $MSE_{d'Angrem}$=0.06, $MSE_{Seabrook}$=0.08, $MSE_{Buccino.}$=0.08; MSE was calculated by introducing into equation (6): y-result of empirical model ($K_{t\ empiric.}$), $\hat{y}$ -result of numeric model ($K_{t\ MIKE}$). Right: Evolution of characteristical statistic, (H_s, H_{mean}), and spectral, (H_{s0}, H_{rms0}), wave heights through numerical wave chanel, for wave conditions in Test17IRR, (Table 2).

2.3 Transmission Coefficients of Wave Periods - MIKE21 Results

It is usuall for calculation of run up and overtopping over the coastal structures and reflection from perforated constructions to use periods of incoming waves. If any of mentioned coastal structures are defended by submerged structures, it is important to calculate transmitted period. Period of single wave which past over submerged breakwater remain unchanged, because of continuity conservation, but it wave height becomes smaller. When waves passing over submerged breakwater, wave breaking occures. At greater wave heights, from wave time series, dissipation of wave energy is greater then at smaller waves. In that situation smaller waves "falls" into the one third of greater waves and change significant wave period. Because of that significant period becomes smaller. Well known fenomen, triad interactions, cause transition of wave energy from primar harmonics to heigher and lower harmonics. Because of greater transition on heigher harmonics, wave field, influenced by those components with lower periods, has more shorter periods than wave field before submerged breakwater. In that situation wave periods T_{mean}, T_2 and also T_s, becomes lower after breakwater.

In Fig. 6 (left), it is shown the influence of submerged breakwater freeboard F in ratio with wave length on period transmission coefficient. Transmission coefficients are calculated with spectral (T_1, T_2) and statistical (T_s, T_{mean}) wave parameters like:

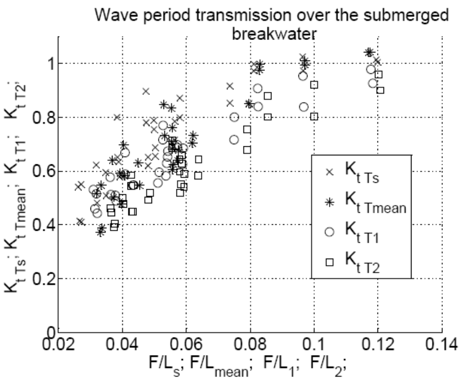
$$K_{t\,T_s} = \frac{T_{si}}{T_{st}}; \quad K_{t\,T_{mean}} = \frac{T_{meani}}{T_{meant}}; \quad K_{t\,T_1} = \frac{T_{1i}}{T_{1t}}; \quad K_{t\,T_2} = \frac{T_{2i}}{T_{2t}}; \tag{7}$$

where: T_s is statistical significant wave period, T_{mean} is statistical mean wave period , T_1 is spectral mean centroiod wave period, T_1 is spectral mean zero-crossing wave period.

Spectral periods were calculated with:

$$T_1 = \frac{m_0}{m_1}; \quad T_2 = \sqrt{\frac{m_0}{m_2}} \tag{8}$$

where: m_0, m_1, and m_2, are zero, first and second moment of spectral energy density function.



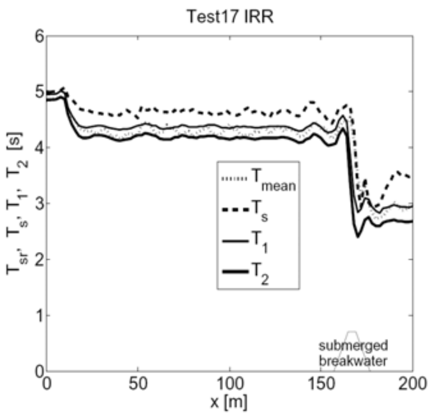
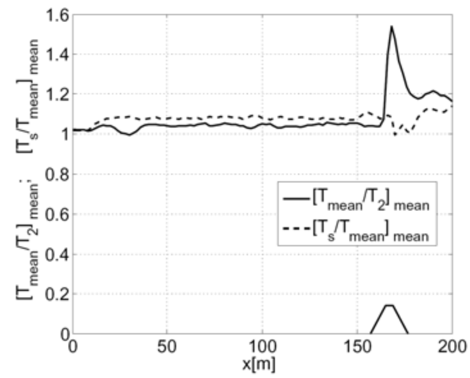


Fig. 5. Left: Comparison of period transmission coefficients ($K_{t\ Ts}$, $K_{t\ Tmean}$, $K_{t\ T1}$, $K_{t\ T2}$) obtained by MIKE21. Transmission coefficients are calculated with different spectral (T_1, T_2) and statistical (T_s, T_{mean}) wave parameters. Wave lengths, (L_1, L_2, L_s, L_{mean}) are calculated in corresponding to different periods. Right: Evolution of characteristical statistic and spectral wave periods through numerical wave channel, for wave conditions in Test17IRR (Table 2).

At Fig. 6 (right), it is presented evolution of characteristical statistic and spectral wave periods through numerical wave chanel, for wave conditions in Test17IRR (Table 2).



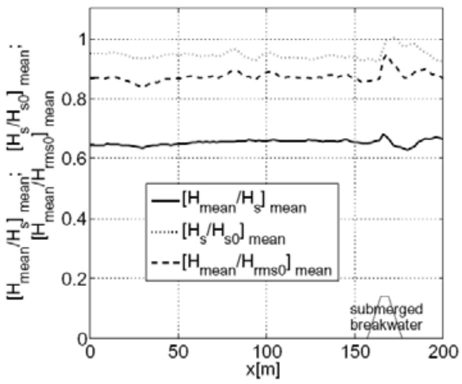


Fig. 6. Left: Characteristic ratios of statistical and spectral wave periods in numeric wave channel. Right: Characteristic ratios of statistical and spectral wave heights in numeric wave channel.

2.4 Comparison of Statistical and Spectral Wave Periods - MIKE21 Results

MIKE21BW solves Boussinesq equation in time domain, where result of calculation is surface elevation timeseries in each point of modeled space. Analisys of timeseries could be made by spectral and statistical approach.

Firstly, for every Test from Table 2, should be calculated evolution of characteristic wave parameters, like those presented in Fig. 4 (right), and Fig. 5 (right).

For example, $H_s(x)$. Secondly, for every point, and every Test (Tbl. 2.), should be calculated ratios of characteristic wave parameters. For example, $H_s(x)/\ H_{s0}(x)$. Thirdly, for every point, should be calculated mean ratio for values from different Tests (Table 2.). For example $[H_s(x)/\ H_{s0}(x)]_{mean}$. Result of those calculatins are presented in Fig.6. Ratio, $[T_{mean}(x)/T_2(x)]_{mean}$, should be 1, becouse this parameters present same period.

Variations occure near breaking and deshoaling zone on submerged breakwater. Ratio, $[T_s(x)/T_{mean}(x)]_{mean}$, should be ~1.1, [10], which is nearly satisfied. Ratio, $[H_{mean}(x)/H_s(x)]_{mean}$, sholud be ~0.63, [10], which is also satisfied. Ratio, $[H_s(x)/H_{s0}(x)]_{mean}$, sholud be ~1, where we can observe little underestimation of $H_s(x)$ in corresponding to $H_{s0}(x)$ Ratio, $[H_{mean}(x)/H_{rms0}(x)]_{mean}$, sholud be ~0.886, [10], which is also satisfied.

All ratios of wave heights have deviation in breaking and deshoaling zone at submerged breakwater.

3 Conclusions

In this paper, calibration of numerical model MIKE21-1D was conducted using laboratory results.

Calibration was conducted by fitting of breaker angles (initial-F_i, final-F_0). Numerical results have shown good agreement with laboratory results for breaking angles F_b=14° and F_0=7°.

- Validation of numerical model was conducted with empirical formulas.
- Results indicate that the greater part of numerical transmission coefficients falls inside limits defined by +/-0.1 of average values of transmission coefficients according to different authors.

Also, possibility of wave period transmssion calculation was presented and it dependance of ratio freeboard/wave length.

Comparison of statistical and spectral wave parameters showed a good agreement and consistency.

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A67**

Processing of Suspended Sediment Concentration Measurements on Drava River

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Abstract. This paper presents processing of measured concentrations of suspended sediment load in three hydrological stations on the river Drava, Croatia. Measurement data were available for discrete moments during 17-year period (1990-2007) in gauging stations Botovo, Terezino polje, and Donji Miholjac. The data were provided by the National hydrometeorological Institute (DHMZ), which carried out the sampling and initial calculations of concentration. Data processing includes only ¨profile¨ measurements where sediment sampling was done on several verticals and at several depths of the discharge profile. Data processing shows adjustment of the theoretical curve of distribution of concentration of values measured at each station. Mathematical concentrations used in further processing were obtained by excluding outliers in measured data. Further, the paper shows the dependence water surface elevation - discharge, as well as discharge - suspended sediment concentration in the stations, and interdependence of sediment concentrations between the three stations for small, medium and high water flows. Based on the resulting dependence of concentrations between the stations, the estimate is given of concentration of suspended sediment load in the downstream station Belisce. Finally, based on the estimated discharge - sediment concentration dependence on gauging station Donji Miholjac, estimate is given of total annual suspended sediment load for the period 990 – 2007.

Keywords: Suspended sediment concentration, sand-bed river, Drava River

1 Introduction

Suspended sediment load is important for the regime of a watercourse due to its channel-forming property, and its concentration and discharge is monitored in river engineering. Traditionally, measuring of suspended load is done by direct sampling in one or several measuring points in a given designated cross-section. Sediment sampling in a watercourse is a complex procedure due to difficult installing of the measuring equipment and required additional laboratory analysis of collected samples. Duration and extensiveness of works required getting a measuring sample results in measuring discontinuity, and sampling is mainly periodical. As sampling is not automatic and requires human presence, and at certain hydrologic conditions becomes quite impossible, it is often necessary to supplement load measurements with empirically determined relations.

Determining of the relation between discharge and suspended sediment concentration on the basis of measurements makes it possible to extrapolate the results on other hydrological events in the same river reach, and to apply the obtained relations on other river reaches with similar hydrological, hydrodynamic and sedimentological properties.

This paper describes the methodology of processing and analysis of measured hydrological-hydraulic parameters in gauging stations Botovo, Terezino Polje and Donji Miholjac on the Drava River, in a longer period from 1990 to 2007. The measurement data available for the paper included discharge, flow velocity, flow area, free surface top width, mean profile depths and concentrations of suspended load. The paper shows functional relations between the discharge and the concentration of suspended load (*Q-C*) and the reliability of the resulting relations. The *Q-C* relations for three stations were compared, and the distribution of *Q-C* relations along the watercourse was demonstrated. Based on the relations determined in the upstream gauging stations Botovo, Terezino Polje and Donji Miholjac, the estimate was made of the *Q-C* relation in the downstream gauging station Belišće. Additionally, the estimate was made of the total annual sediment discharge in the gauging station Donji Miholjac for the period 1990–2007. Statistical analysis of hydrological parameters was made in the program package SAS version 9.1.3.

2 Monitored River Reach

The Drava River springs in Southern Tyrolia and flows through the Austrian province Kärnten, Slovenia, and in Croatia where it forms a part of the Croatian-Hungarian border. Downstream from Donji Miholjac it turns towards inland Croatian territory, towards Osijek and flows into the Danube near Aljmaš on the border between Croatia and Serbia. The total length of Drava River is 749 km, out of which 323 km in the Croatian territory, with the navigation waterway of about 90 km – from the confluence with the Danube to Čađavica. The catchment area of the Drava River is 38.500 km^2 out of which 16,5% on the territory of the Republic of Croatia.

The area involved in the analysis of suspended sediment discharge is situated downstream from the Dubrava Hydroelectric Power Plant (HPP) to the gauging station Belišće (Fig. 1). The paper includes four gauging stations, i.e. Botovo (rkm 226+700), Terezino Polje (rkm 152+700), Donji Miholjac (rkm 77+700), and Belišće (rkm 45+300). The Mura River, with its catchment area of 10.891 km^2 is the largest tributary of Drava, and its mouth is directly downstream of Dubrava HPP (rkm 236+700), and upstream from the gauging station Botovo.

The average annual discharge of the Mura at the mouth is 170 m^3/s, and the average annual discharge of the Drava at the Dubrava HPP is 355 m^3/s. The mean annual discharge measured at the gauging station Belišće, situated downstream from Mura River mouth is 522 m^3/s, which shows that downstream the Mura River mouth there are no major tributaries. The designed discharge of the Dubrava HPP is 500 m^3/s. The water regime of the Drava is pluvial-glacial, characterized by low flows in winter, and high flows in the second half of spring and in summer. However, the natural water and sediment regime has been considerably changed by the dominant influence of a large number of hydropower plants in Austria, Slovenia and Croatia.

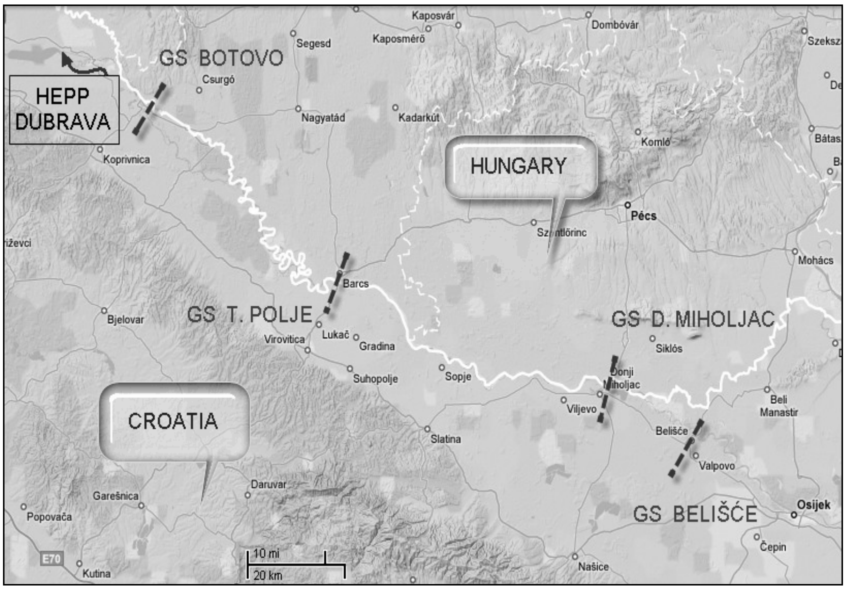

Fig. 1. Geographic location of the Drava River reaches with gauging stations

3 Available Measurements

Measuring of hydrological and hydraulic data, as well as initial processing was provided by the National Hydrometeorological Institute (DHMZ). Through the history of measuring on gauging stations Botovo, Terezino Polje and Donji Miholjac it may be seen that measuring included mean flow velocity, discharge, cross-section geometry, flow area, mean cross-section depth, flow top width at free surface, and suspended sediment discharge. The period involved in this paper is from 1990 to 2007, i.e. the period after construction of the last hydropower plant on Drava River.

Measuring of the suspended load discharge is done on the constant profile in the watercourse, assuming that its geometry will stay unchanged for a longer period (Fig. 2).

In order to determine suspended sediment concentrations, water samples are taken on a daily basis by the measuring vessel at one point on the watercourse surface. The samples are then processed in the National Hydrometeorological Institute by the standard filtration method ISO 4363:2002, giving concentrations of suspended load in g/m^3. On the other hand, profile measurements of discharge and suspended sediment are made in discreet time moments, which make 100 to 150 data sets for each gauging station in 17-year period.

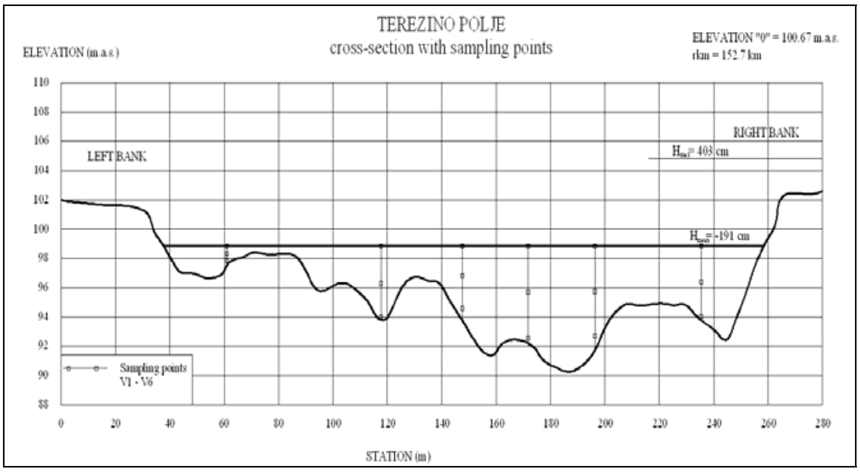

Fig. 2. Channel profile at designated cross section of the gauging station Terezino Polje

In profile measurements samples are taken by a free-flow meter in six cross-section verticals, with three samples per vertical: on the surface, in the middle and at the bottom (Fig. 3). All three samples from a vertical are joined into a single sample and processed ion the laboratory to determine the mean suspended sediment concentration for each vertical.

After processing of separate verticals, the suspended sediment concentration is averaged for the entire cross-section, and the result is referred to as the mean cross-section suspended sediment concentration. In a known designated cross-section the total quantity of suspended sediment load which will be discharged through the cross-section is defined by the double integral:

$$G = \int_0^h \int_0^B v \cdot C \cdot dh \cdot dx \; [\mathrm{kg/s}] \tag{1}$$

This paper analyzes only profile measurements, because hydrological parameters are measured on the entire cross-section of the watercourse, and not only at the surface as in daily measurements. Besides discreet measurements of discharge and suspended sediment, the available data also included water surface level, as continuous records in hourly intervals. Additionally, for the gauging station Belišće, available documentation included the rating curve of the relation between discharges and water surface levels.

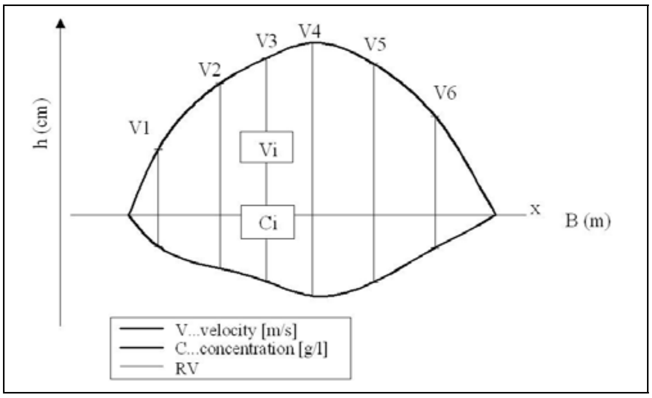

Fig. 3. Schematic of calculation method for cross-section suspended sediment concentration

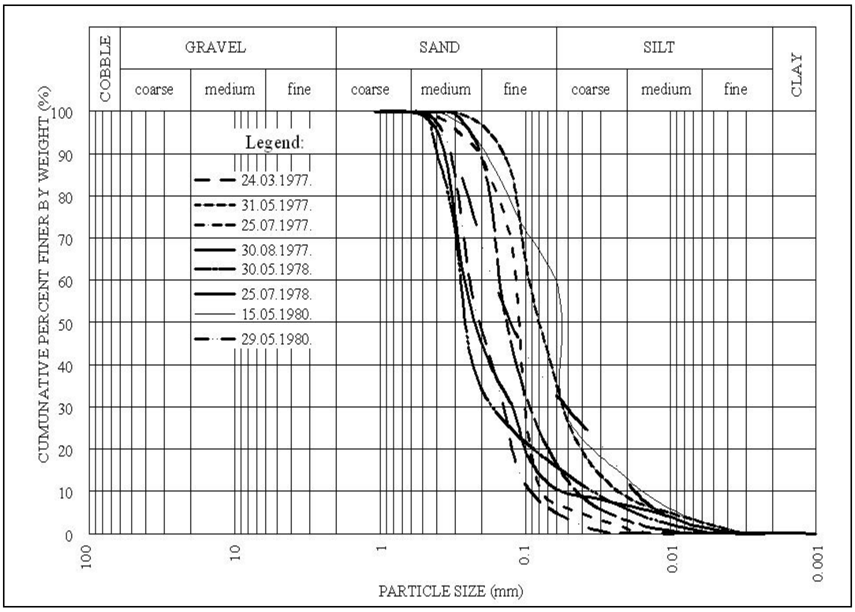

Fig. 4. Grain size distribution curve of suspended load measured on the Drava River

The composition of the suspended sediment was taken over from the study "Forecasts of morphological-psamological processes in the Drava River after construction of the hydropower plant Novo Virje" (IEE, 1997). It may be seen from the grain size distribution curve (Fig. 4) that there are about 70% of sand and about 30% of silt. The mean particle diameter of the suspended load is d_{50} = 0,10 mm.

4 Quality Control of Measured Values

During the observation period of 17 years, various techniques of measuring of hydrological values were used. Measuring of velocities and determining of discharges was done by means of the hydrological wing on a defined number of points of the hydrological cross-section, and after 2004 data were collected by means of the acoustic flow meter (ADCP). In all measurements on the watercourse, certain outliers among measured values are always expected, regardless of the measuring technique used. Therefore, before the analysis of the relation of discharge and suspended sediment concentration, quality control of measurements was carried out. In previous analyses (Oskoruš, 2008) some functional relations of discharge and suspended sediment concentration for gauging stations on the Drava and the Mura were presented. That work analyzed only the relation between the discharge and the suspended sediment concentration, while for this paper additional hydrological data were available, such as: mean flow velocity, cross-section area, mean cross-section depth and top width of free surface. This allowed more comprehensive control of the quality of measured values.

Outliers in measured values may be the result of measuring errors, but also the values deviating from the global trend and contributing to the characteristics of distribution. Identification of outliers was done with caution, through parallel comparison of the scatter plots of measured values. In the program package SAS, scatter plots were compared for discharge, flow velocity, suspended sediment concentration, mean cross-section depth, water surface level, flow area and free surface top width. All measured values were organized in the matrix, and the interrelation of hydrological values was compared on the interactive interface. All values deviating from the global trend cloud in more than two separate scatter plots was excluded.

Since introduction of ADCP, measuring of discharge, velocity profile and geometry has become considerably simpler and more precise. However, in this modernization measuring of suspended sediment concentration has become a procedure separated from measuring of other parameters. Analysis of covariance in Q-H relation compared the traditional measurements (until 2004) with measurements by ADCP to see if the data may be processed together or should be separated.

In the interrelation of discharge and velocity (Q-v) deviation of some data from the global trend cloud was noticed. The deviations noticed pertain to the group of data after introduction of velocity and discharge measuring by ADCP. Reduction of velocity is noticed at the same discharge in traditional measuring. The analysis of covariance of all measured hydrological values showed that the change of measuring technique influences only flow velocity measurements (Q-v relation), and not other hydrological values, and the conclusion was that the entire set may be considered as a whole.

5 Results

In the program package SAS, the analysis of regression in gauging stations Botovo, Terezino Polje and Donji Miholjac was made for two types of relations, i.e. between the water surface level as the independent variable, and the discharge as the dependent variable, and between the discharge as the independent variable and the water surface level as the dependent variable. To allow proper statistic processing of the sets of data, distribution function of the sets of data which will be dependent variables, i.e. discharge and concentration of suspended load, was analyzed first.

For the analyzed set of data, distribution fitting of theoretical and empiric functions were tested, by the Kolmogorov-Smirnov test. K-S parameter D is the largest absolute difference between the cumulative observed proportion and the cumulative proportion expected on the basis of the hypothesized distribution

The zero hypothesis is tested, that the sample is drawn from a theoretical distribution function, and the threshold of significance of 0.05, i.e. 5 % was chosen. The functions chosen for adjustment are normal, log-normal, Weibull's and exponential. For all selected functions were calculated statistical values of Kolmogorov parameter D and probability p that the sample is drawn from the distribution. They show that the sample is drawn from log-normal distribution, because p-value turned out to be significant only in fitting of log-normal distribution on given dataset.

Table 1. Values of statistical parameters for log-normal distribution fitting

	Station:	D. Miholjac	T. polje	Botovo
Discharge	*D:*	0.083	0.078	0.092
	p:	0.111	>0.15	>0.15
Suspended sediment concentration	*D:*	0.059	0.090	0.062
	p:	>0.15	0.112	>0.15

The hypothesis that the linear model of regression does not describe the data better than the model of the mean, i.e. the hypothesis $H_0 : \beta = 0$, at $\alpha = 0.05$, was tested by regression analysis between water level and discharge in all three stations. The alternative hypothesis is $H_a : \beta \neq 0$. It shows that the model is significant, and that the hypothesis H_0 is rejected, and the hypothesis H_a is accepted, i.e. that the constant in linear regression equation is statistically significantly different from 0 and that some relationship exist between dependent and independent variable. The curvature on the residual graph indicates that probably a polynomial of second order or an exponential curve would describe better this set of data, and these analyses were carried out as well. The comparison of regression analyses through the determination coefficient R^2 has shown that in all stations regression through the polynomial of 2nd degree, i.e. square function, describes best the dependence between the water surface level and discharge. The calculated determination coefficients are very high on all stations: Botovo $R^2 = 0.82$, Terezino Polje $R^2 = 0.92$ and Donji Miholjac $R^2 = 0.99$.

Calculated values of stage-discharge curve equations show the following values:
Botovo:

$$Q = 654095 + 44.904 \cdot H^2 - 10837.3 \cdot H \tag{2}$$

Terezino polje:

$$Q = 345438 + 37.7319 \cdot H^2 - 7260.08 \cdot H \tag{3}$$

Donji Miholjac:

$$Q = 190735 + 26.3241 \cdot H^2 - 4480.68 \cdot H \tag{4}$$

As the regressive relation between water surface level and discharge is established, it becomes apparent that it is possible in every moment, with great precision, to calculate the discharge on the basis of the measured water surface level data, and the analysis of regression between the discharge as the independent variable and suspended sediment concentration as the dependent variable can be carried out. Regression analysis in all three stations showed that in all three cases linear regression of logarithm set of data, i.e. exponential function, gives the best description of the dependence between the water surface elevation and the discharge. Calculated determination coefficients in the stations are, as follows: Botovo R^2 = 0.32, Terezino Polje R^2 = 0.34 and Donji Miholjac R^2 = 0.44. The calculated exponential equations of *Q-C* curves have assumed the following values:
Botovo:

$$C = Q^{1.0408} \cdot e^{-10.3842} \tag{5}$$

Terezino polje:

$$C = Q^{1.1111} \cdot e^{-10.7443} \tag{6}$$

Donji Miholjac:

$$C = Q^{1.1395} \cdot e^{-10.7494} \tag{7}$$

If the regression curves calculated at the logarithmic scale in all stations are compared with a graphic presentation (Fig. 5 and 6), there is a notable growing trend of suspended sediment concentration in the downstream direction for every discharge class. From each *Q-C* curve, in regular intervals within the limits of Q=200m^3/s to Q=2000m^3/s, the discharge was divided into 24 classes, and the values of suspended sediment concentration were read. Based on the geographic distance between stations along the river course the curve of suspended sediment concentration trend was established, i.e. regression analysis of the trend in the stations was made by exponential curve. The calculated curves were extrapolated 30 km downstream to the gauging station Belišće, where the exponential curve of the relation between discharge and suspended load concentration was calculated with 24 calculated data (Fig. 6). The equation of the *Q-C* curve in Belišće is:

$$C = Q^{1.1947} \cdot e^{-10.968} \tag{8}$$

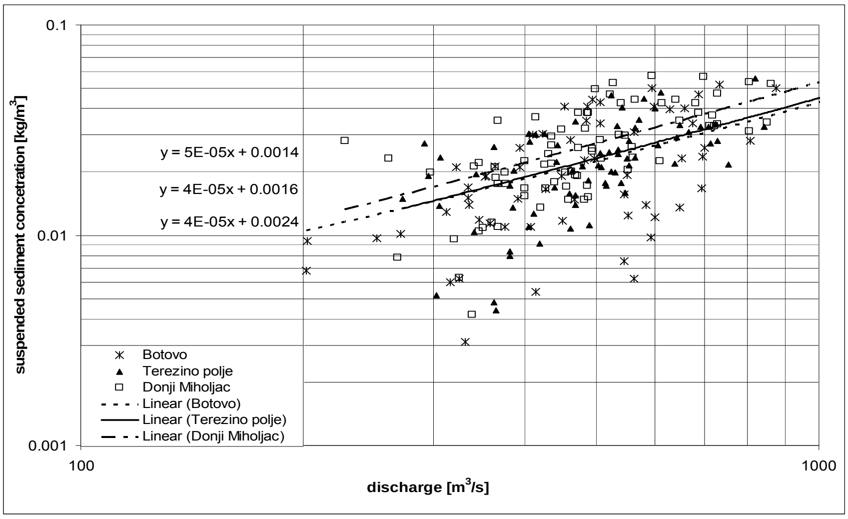

Fig. 5. Presentation of all measured values of suspended load concentration and corresponding discharge with calculated linear regression curves for each station

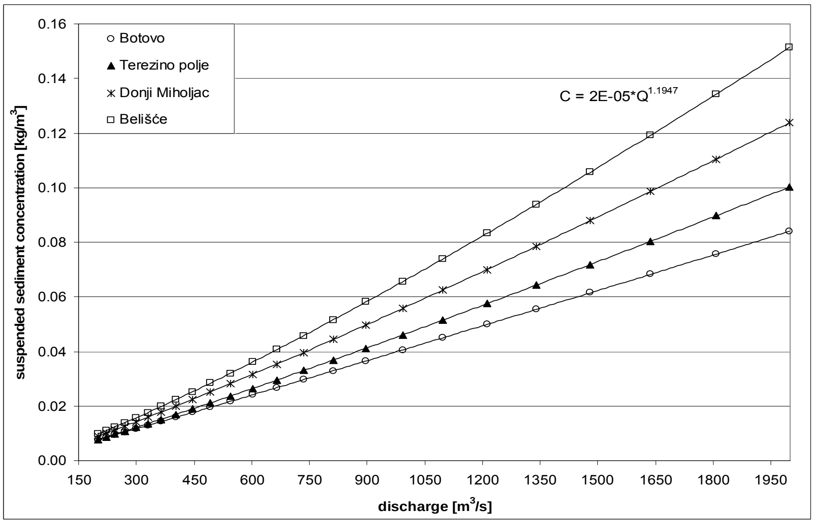

Fig. 6. Relation between *Q-C* regression curves in logarithmic scale with the assumed equation on the extrapolated station Belišće

6 Comparison with Previous Analyses

Based on the obtained dependence of the discharge and the suspended sediment concentration, the total annual sediment load *G* was calculated for the period from 1990 to 2007 at the gauging station Donji Miholjac (Fig. 7). Daily data were provided by the National Hydometeorological Institute. Obtained values of total annual sediment load were compared with the values from the paper by Bonacci, D., Oskoruš, D. The influence of three Croatian hydroelectric power plants operation on the Drava River hydrological and sediment regime. The average annual sediment load from cross-section measurements $G_{PROFILE}$ = 483 t/year is higher than the value of average annual yield from point measurement G_{POINT} = 265 t/year, by 87%.

The differences of estimated quantities of sediment are the result of different approach to measuring of sediment concentration.

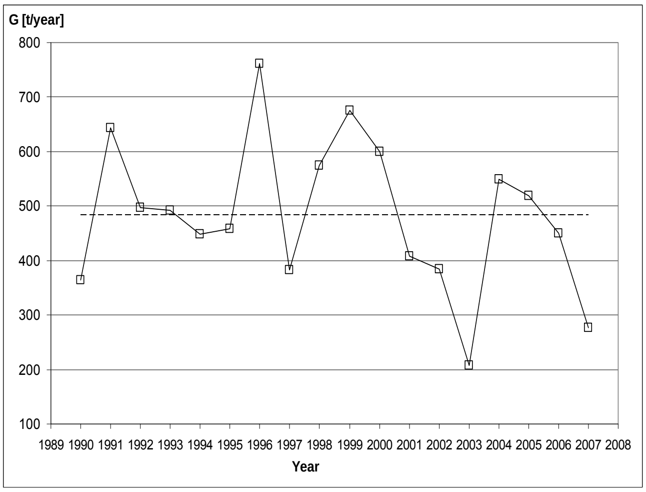

Fig. 7. Measured annual suspended sediment yield at gauging station Donji Miholjac

Previous analyses (Beraković *et al.* 1998) show the *Q-H* curve at the gauging station Botovo, which is compared to the regression relation from this paper. Compared stage-discharge curves are equal at medium and high flows, while discharges at low flows in this paper exceed the discharges from previous analyses.

7 Conclusions

The paper shows the interrelations of hydrological parameters in three gauging stations on the Drava River from Dubrava HPP to Belišće, with particular attention paid to the relation between the discharge and suspended load concentration.

After establishing of uniform relations of water surface level and discharge, and the dependence of discharge and suspended sediment concentration in the stations Botovo, Terezino Polje and Donji Miholjac, the suspended sediment rating curve was estimated on the downstream gauging station Belišće.

Regression relations of hydrological parameters make it possible to estimate suspended sediment concentration on the basis of water surface elevation in four gauging stations on the Drava River. It has been noticed that after construction of the latest hydropower plant (HPP Dubrava) no significant changes of the suspended load regime have occurred, which means that the established relations may be regarded for the entire period after 1990.

Comparison of estimates of the average annual sediment yield made by different sampling methods show that sediment yield from cross-section measurements gives 87% higher suspended sediment concentrations than in point measurements.

The limitation of obtained dependencies of suspended load concentrations is the lack of sediment sampling data in the winter period. As the flow of the Drava River in winter is small, lower reliability of obtained dependencies at low flows is assumed.

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International Symposium on Water Management and Hydraulic Engineering

Ohrid/Macedonia, 1-5 September 2009

Paper: **A76**

Interaction Matrix Method in Hydrogeological Analyses at Coal Mines

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Abstract. Groundwater has an important practical influence which must be analyzed in protection measures design in coal mining. For an instance, protection from groundwater aggressive chemical and physical action, hydrostatic and hydro-dynamical forces in stability and drainage analyses, groundwater inflow etc, have significant impact on designed structures and finally on the exploitation costs. Coal mine exploitation has also important ecological influences, which requires a specific approach during hydrogeological analyses. The estimation of input parameters in such analyses is a basic prerequisite for adequate modeling of the media. So, the objective of this paper is to present possibilities of so-called interaction matrix method, applied in hydrogeological modeling. Case study is the coal mine "Suvodol" near city of Bitola in Macedonia. The basic elements of developing of Hydrogeological Engineering Systems (HES) are also given. The paper can serve as an example for further analyses in this field for similar cases.

Keywords: Coal mine, groundwater, hydrogeology, interaction matrix method, Hydrogeological Engineering System

1 Introduction

It is well known that rational and efficient engineering design activities in coal mines is not possible without knowing in detail the groundwater conditions. It is especially emphasized in cases of complex hydrogeological conditions in the environment, as they are defined at some zones in the area of coal mine "Suvodol".

Here, we also introduce an interaction matrix method in ways in which some of the hydrogeological parameters and variables can affect one another in processes of mechanical and hydrogeological interactions.

The method is presented within the wider context of an approach to integrate all the relevant information in applied hydrogeology, in a similar way as it is given in rock engineering design and construction. Namely, the methodology of developing of so-called Rock Engineering Systems (RES) is firstly introduced by Hudson [3]. The methodology is based on the demonstrating the links between necessary parameters for engineering analyses, especially the pre-construction and post-construction interactions.

Here, throughout a case of coal mine "Suvodol" will be explained possible application in applied hydrogeology, demonstrating the possible way of developing of Hydrogeological Engineering Systems (HES). The HES concept is an approach which aims to identify the parameters relevant to a problem, and their possible interactions. The whole concept providing overall coherency in approaching engineering problems at coal mines, where the need to study the interactions has always been present. In fact, the interaction matrix method can be shortly explained as an intelligent method for solving complex problems, using appropriate conceptual, mathematical, numerical or mechanical models. This is very important, having in mind that the hydrogeological processes are usually time-dependent, which means the parameters are usually not constant- changeable in the time.

The paper deals with the importance of correctly set and carried out investigations of the groundwater conditions, as well as the need of appreciation of the principle of equal grade and whole grade of investigation for the expected zone of interaction between the natural environment and the engineering activities. A framework for this concept is earlier given in [5].

2 Analysed Area and Basic Geological and Hydro-Geological Conditions

The surface coal mine “Suvodol” is placed in the southwestern part of the Republic of Macedonia. The coal mass and the unproductive layers have been formed with a process of sedimentation in lake conditions during upper Pliocene. Mainly, there are layers at the bottom of the coal (layers of silty sands), productive series of coal and coal-like clay, and layers on the upper part of coal of volcanic material (so-called trepel). This coal mine is a main source for thermal-electricity plans in the country, with a production of about 6.500.000, 00 tons per year.

The mine area is investigated in several phases before and after opening of the mine. These investigations have been preformed to get knowledge of the entire geological, geotechnical and hydrogeological terrain characteristics. For an instance, mapping of the wider area, investigation drillings, installing of group piezometers, investigations of the chemical composition of groundwater’s, field investigation of filtration coefficient, as well as laboratory analyses of physical and mechanical properties is applied.

This paper presents only the part dealing with the groundwater conditions, while the other results are shown in appropriate reports [1], [2], [6], and [7] and on conferences [4], [5]. The obtained data from investigations were basis for preparing reliable physical models (profiles, maps, geotechnical models) used in further stability, dewatering and drainage analyses [6], [7].

The main idea of the investigation methodology is to include evenly and wholly the space in which it is expected mutual influence of the engineering activities and the natural environment. Because of large scale of excavations, it should be mentioned that, a wide area of the environment can be affected, including the earth fill dam upstream of the zone of excavation, zone of active landslide bellow the dam etc., Fig. 3 and 4.

Taking into consideration the fact that groundwater has the greatest influence on the stability of the terrain; this was a problem of primary interest in this coal mine. The most important data were obtained by installing of so-called group piezometers. More than 60 piezometers groups are installed in several investigation stages. For each group piezometer special borehole was made which is the most safety way to isolate every aquifer zone.

An illustration of the geological and hydrogeological conditions, as well as the scheme of installed triple piezometer is shown in Fig. 1 and 2.

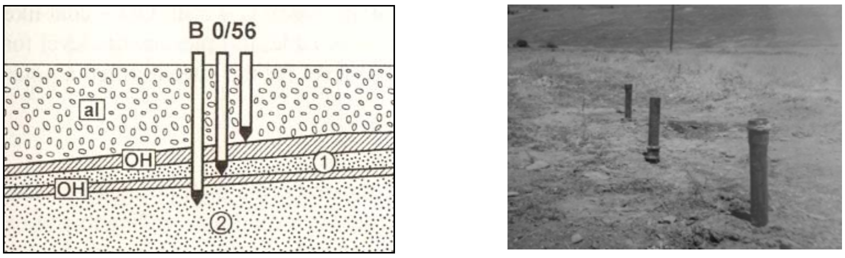

Fig. 1. Scheme of triple piezometer installed in a borehole B 0/56: al-alluvial sediments; OH-coal-like clay; 1-interstratified aquifer zone; 2-aquifer zone at the bottom of clay

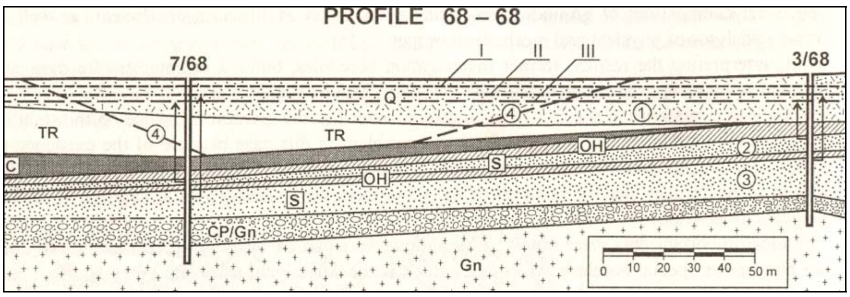

Fig. 2. Detail of geological and hydrogeological composition at one zone of coal mine ˝Suvodol˝: 1-Aquifer zone with free water level; 2-Interstratified aquifer zone under pressure; 3-Aquifer zone at the bottom of the coal layer; 4-Designed cut; Q-Quarterian silty sand layer; TR-trepel (aquiclude); C-coal; OH-coal-like clay (aquiclude); S-silty sands (aquifer); Gn-gneiss; I-free water table; II-piezometric level for the aquifer zone at the bottom of the coal layer; III-piezometric level for the interstratified aquifer zone under pressure

Some of the zones at the coal mine are with high lithological heterogeneity which is the reason why there is high heterogeneity of hydrogeological characteristics. So, by the help of installed piezometers, the presence of several physically separated aquifer zones is found. Fig. 3A shows a model of ground water movement for the aquifer zone with free water level. Fig. 3B shows a model of groundwater movement for so called interstratified aquifer zone under pressure, placed between two layers of coal-like clay. In fact, the Fig. 3 is given as an example that that separate aquifer zones can exist on a same part of the investigated area.

Fig. 4 gives a model of groundwater movement for the aquifer zone at the bottom of the coal layer which can be found in the whole mine "Suvodol". Here, the aquifer zone under artesian conditions with very high values of pressures is found.

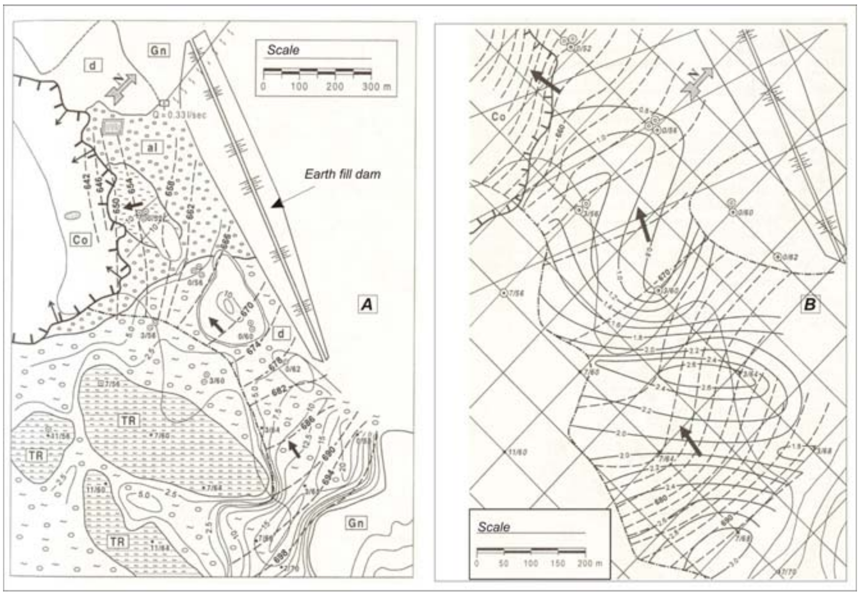

Fig. 3A. Engineering geological map of the NE part of coal mine, legend: ➡ - groundwater flow pats for the aquifer zones; — — 670 — — contour lines of groundwater level; al-alluvial sediments; dl-deluvial sediments; TR-trepel; Gn-gneiss;

Fig 3B. Model of the groundwater movement for the interstratified aquifer zone under pressure at the NE part of the coal mine: — — 670 — — –contour line of groundwater level for the aquifer zone under pressure; —— 2 —— – contour line of equal artesian pressures (in bars); Co-colluvial material (active landslide)

From the aspect of chemical interaction of groundwater, the chemical composition is very important, because it influenced the installed equipment (pumps) for dewatering (Table 1). It can be noticed that there are aggressive groundwater components with presence of gas (CO_2, Ra and others), which is important from ecological aspect and working conditions at the mine.

Table 1. Typical chemical composition for the aquifer zones

Content of ions in mg/l	pH	Ca^{2+}	Mg^{2+}	Fe^{2+}	Cl^{-}	SO_4^{2-}	HCO_3^{-}	free CO^2	rest
Aquifer zone with free water table	6,8	20,1	12,5	0,4	158	194	701,5	-	15,5
Interstratified aquifer zone under pressure	6,5	216	21,8	2,6	184	256	760,5	70	6,8
Aquifer zone at the bottom of the coal layer	5,7	140	24,3	4,8	19	43,6	549,3	111	1,3

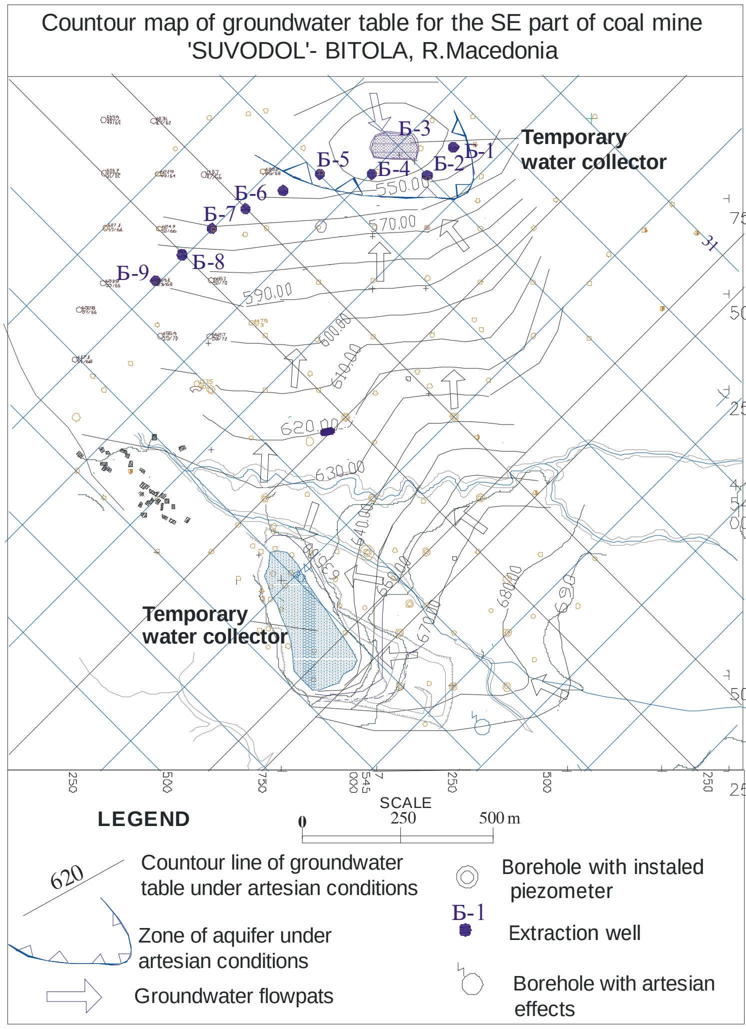

Fig. 4. Model of the groundwater movement for the bottom-coal aquifer zone under artesian pressure

All geological, hydrogeological, chemical and other characteristics of the natural environment are the basis for forming of conceptual matrices and they are treated as possible impact factors that should be analyzed in the presented methodology bellow.

3 Presentation of Interaction Matrix Method in HES Methodology

As it is mentioned earlier, the interaction matrix method is the basic device used in rock-engineering analyses. For a coal mine engineering projects, the most important step in the HES methodology is to establish the objectives of the project and the analysis. This statement is also valid for the engineering problems in general.

The relevant 'state variables' must be chosen in a first place and they are placed along the leading diagonal of the interaction matrix. In some problems, these variables have to be more conceptual in nature. Sometimes, there may be enough information to use well-defined physical properties with definite units. Then, all the interactions are established so that the problem structure is developed. If the state variables are conceptual in nature, the off-diagonal interactions can be assessed using a semi-quantitative method of coding [3].

One example of conceptual matrix with four elements in a leading diagonal is given on Fig. 5. These factors are involved mutually, in 12 basic kinds of interaction.

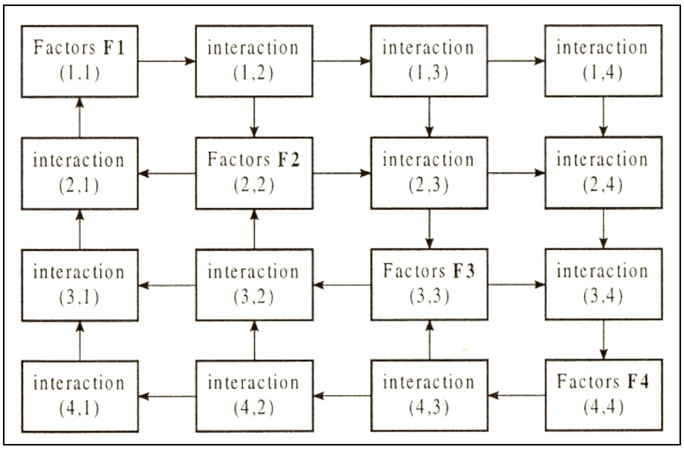

Fig. 5. Conceptual matrix of interaction between four basic factors which have influence on the stability of the coal mine: F1-Structural physical and mechanical characteristic of the sediments; F2-In situ stress; F3-Groundwater condition; F4-Characteristics of the engineering activities

F1 group of factors is related to the sum of properties such as: unit weight, porosity, moisture content, strength properties, discontinuities, plasticity of clay, thickness of layers and so on. F2 group is related to the in situ stress. F3 group is related to the velocity of ground water movement, hydraulic gradients, hydrostatic and hydrodynamic pressures, filtration characteristics and aggressiveness of the waters etc. At the end, group of factors F4 is related to the characteristics of the structures in the mine (wells, slopes etc), such as its depth, wide, dips and heights, technology of excavation and so on. All possible interactions of these factors have influenced the efficiency of working and safety of the dewatering structures and mine slopes.

It is obvious that not only the group of geological, geotechnical and hydrogeological elements influences the characteristics in the designing of the object, but also the construction of the structures contributes to the change of series of properties and conditions. Qualitative expression of these basic influences is given bellow.

Table 2 Qualitative explanation of the interactions

Interaction 1,2	Layers thickness and their weight influence the in situ stress.
Interaction 1,3	Changes in lithological composition, effective porosity influence the filtration, groundwater movement velocity
Interaction 1,4	Physical and mechanical properties are the base for designing stable dips and highs of the slopes
Interaction 2,1	In situ stress influences degree of compaction of sediments, opening of the joints and cracks in the field
Interaction 2,3	In situ stress influences the groundwater condition, pore water pressure, decreasing of the filtration in higher stressed zones
Interaction 2,4	In situ stress determines the point up to which excavation could be made without penetration of groundwater on the floor of the cut (due to hydrostatic pressure values)
Interaction 3,1	Groundwater condition influences the decreasing of the strength properties of sediments, increasing of porosity as a result of pumping
Interaction 3,2	Groundwater condition influences the decreasing of the total stress (the concept of effective stresses)
Interaction 3,4	Artesian pressures, inflow of ground waters, chemical interaction, influence on safety measures for protection.
Interaction 4,1	Excavation influences the change in natural moisture content, unit weight, decreasing of the strength properties
Interaction 4,2	Excavation leads to the changes in situ stress, stress concentration
Interaction 4,3	Excavation influences the groundwater condition through velocity movement's changes, possibility of creating critical hydraulic gradients, appearance of underground erosion

It is possible to analyze in detail the mutual influence between each physical, mechanical, structural and other characteristic, which on the other hand lead to the increasing of dimension of matrix of interaction.

4 Practical Implementation of Interaction Matrix Method at the zone of Coal Mine "Suvodol"

The interaction aspects, besides qualitatively could be encompassed quantitatively by using analytical methods, monitoring during investigation phase and during construction. For illustration, a method used in design of dewatering wells in one zone of coal mine is given bellow (Fig. 6). For other zones and dewatering wells, similar procedures are applied, but this overcomes the frame of this article.

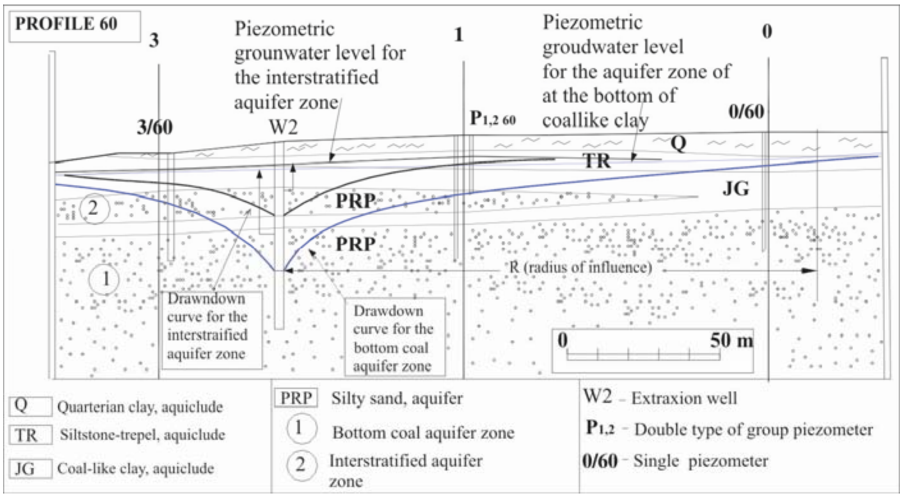

Fig. 6. Hydrogeological profile 60, with data about groundwater levels before and after installation of dewagering well W-2: 3/60 and 0/60-single piezometers along profile line; P1.2 60-duoble piezometer along profile line for control of drawdown

In every case, the first step in analyses is to prepare the conceptual matrices as it is shown in Fig. 7. One form of such matrix is presented in Fig. 8.

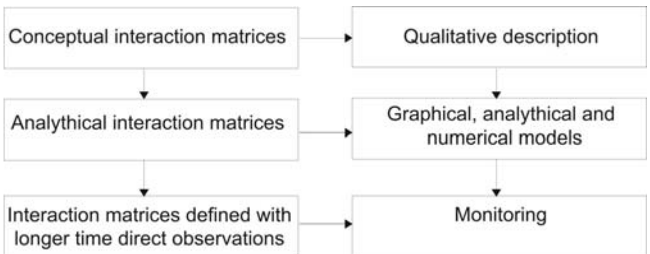

Fig. 7. Main steps in defining of HES methodology starting from conceptual, analytical to matrices defined with direct observations during exploitation phase

Second phase in application of described procedure is usual use of known analytical solutions in order to calculate possible cone depression radius (R), the expected level of drawdown (S), groundwater inflow into the well (Q) etc. Finally, most appropriate quantitative definition of interactions, earlier predicted with known analytical methods, is with direct observations during longer time. Illustration of direct measured data during exploitation phase for the profile 60, are given in Fig. 9, 10 and 11. The data from measurements of drawdown (S), pressures (P) at the bottom of the aquiclude at different distance from observation points (L), are mutually interconnected and analyzed with regression analyses.

F-1 Hydrogeological conditions (number of aquifer zones, groundwater levels, permeability, gradients etc.)	Hydrogeological conditions has an influence on the in situ stress conditions expressed in a value of normal effective pressures	Hydrogeological conditions are the basis for choosing of the position of the dewatering wells, their depth and type
In situ stress has an influence on possible groundwater inflow in excavation, terrain stability etc, which cal lead in a changes of hydro geological conditions	F-2 In situ stress (hydro-static, artesian pressure at the bottom of the aquiclude layers etc.)	The level of stress is one of the basic elements in choosing of the position of the wells (maximal effects when the construction is placed at zone of highest pressures)
A construction of well has an influence on the establishing of new hydrogeological conditions, new gradients, velocity of groundwater flow around well, settlements etc.	An adequate chosen place of drainage and dewatering structures has a direct influence on the decreasing of stress and improvement of the stability of terrain	F-3 Position of the well, depth of pump installation ant other constructional characteristics

Fig. 8. Conceptual interaction matrix of between tree basic factors in design of dewatering wells

In Fig. 9 is presented the single regression line between observed drawdown (S) and distance of the analyzed point from the well (L).

Further ways of connection is illustrated in Fig. 10. This is a case of so-called 2x2 interaction matrix in which the parameters drawdown (S) and distance (L) appear on the leading diagonal. In calculating the regression line linking these two parameters, it could be considered one independent variable as it is illustrated in the off-diagonal boxes. Data points are the same in each case: the axes have simply been exchanged, and a different regression line is obtained in each of the two cases.

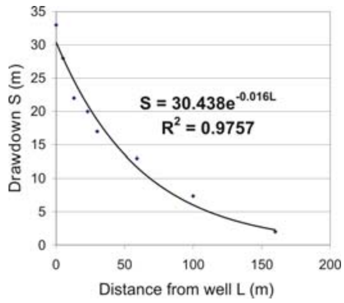

Fig. 9. Single regression line showing influences between drawdown (S) and distance from the well (L) for the zones along profile 60

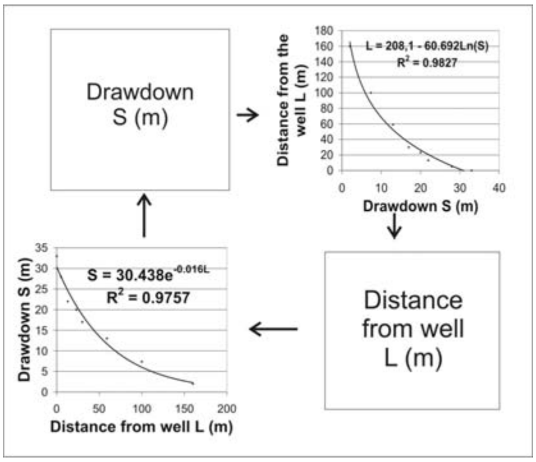

Fig. 10. Drawdown-distance from the well relations, illustrating the 2*2 interaction matrix

In Fig. 11 it is shown 3x3 interactions matrix in which the parameters drawdown (S), distance (L) and artesian pressure at the bottom of the coal layer (P) appear on the leading diagonal. Further interactions with increasing of factors in leading diagonal can be defined with multi-regression analyses. A way of linking between tree parameters is given with following expression P=0.3976-0.0134S+0.0077L, where P is the pressure at the bottom of aquiclude layer (in bars), S is the value of level of cone depression at certain point (in meters), and L is distance from the extraction well (in meters).

All this data are very important during the defining of the distance between wells, expected drawdown, inflow of groundwater into the well etc. The formula is established with multi-regression analyze for one quasihomogenous zone at the NE part for coal mine “Suvodol”:

It is obvious that even matrix of interaction of lowest range, shows several complex mutual influences between the natural environment and the engineering activities. Here, known method of simple regression and multiple regression analyses are very useful to analyze observed data, and to incorporate the results in interaction matrices of most important parameters.

Defined parameters with such approach are a good basis for complex analytical and numerical analyses, where the interactions can be defined with all necessary outputs (safety factors, stress-strain conditions, groundwater quantities etc).

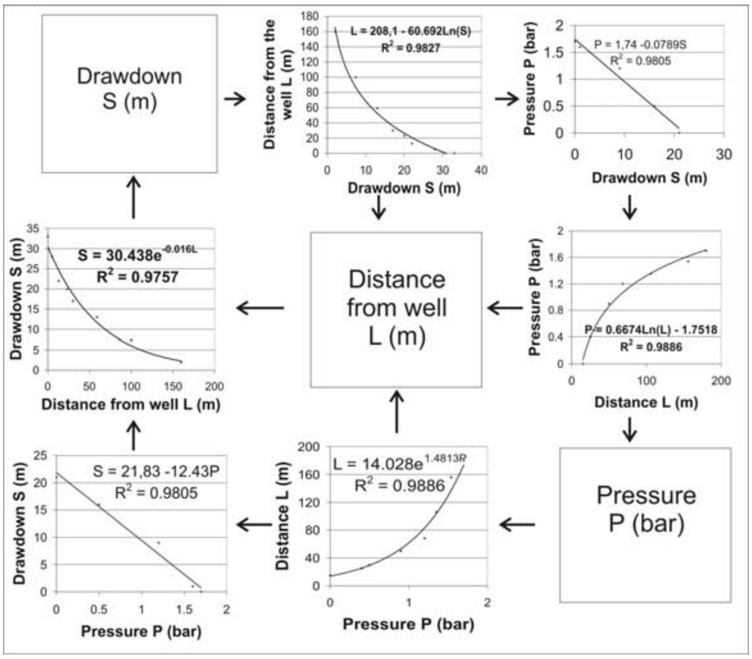

Fig. 11. Drawdown-Distance-Pressure relations, illustrating 3x3 interaction matrix

5 Conclusions

It is fundamental for successful design of each engineering activity to get acquainted in detail with the properties and conditions of natural environment. Among the factors that have the greatest influence on the excavation conditions, protection of environment, stability of terrain and so on, the groundwater conditions are specially emphasized. Without an adequate methodological approach in investigating, which will be completely adapted to the characteristics of the natural environment, it is not possible to define the physical model of the terrain. The physical model of the terrain must be the base for all mathematical models. It can be noted that this are only part of analyses used in design of dewatering and stability analyses of "Suvodol" coal mine. More details overcome the frame of this paper, but it is clear that similar methodology can be applied for all engineering problems in coal mining, as well as for a similar hydrogeological and geotechnical problems. The authors recommend using separately for each object the concept of matrix of interaction about all influential factors and this will be the subject of our further analyses.

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A80**

Optimization of the Discharge Regime of the Gabčíkovo Water Structure with Regard to the Safety of Water Transportation

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Abstract. The article handles with the methodology for water manipulation regarding the Gabčíkovo Hydropower Plant situated on the Danube River. The aim is to increase the effect of energetic operation while maintaining the safety of water transportation.
The methodology includes: a) design of discharge models of the Gabčíkovo hydropower plant with regard to maximization of the economic effect, b) assessment of the impact of discharge models of Gabčíkovo hydropower plant on water level mode and parameters of the fairway, and c) determination of traffic boundaries of the Gabčíkovo hydropower plant with regard to the safety of the waterway operation and the compliance of the fairway parameters.

Keywords: Water structure, navigation, hydropower plant, regulation, hydrodynamic modelling

1 Introduction

The operation of water structures affects the hydraulic regime and thus the navigational conditions. The vessel movement on a waterway is largely influenced by the hydraulic parameters of the water flow. The optimal and safe vessel movement in the flowing water as well as the handling and maneuvering of vessels are directly related to the direction, orientation and size of the velocity vectors of the flow, the water surface gradient, the water level, the parameters of the fairway and the time course of these parameters.

On multi-purpose water structures, which consist of different functional objects (lock, weir, hydropower plant), the operations of these facilities affect the flow and through the force action of flow they influence the route of the vessel movement.

There is also the affecting the fairway parameters (especially the depth) in the case of an individual water structure, which is not a part of an interconnected system (a cascade) of water structures.

To assess the effects of flow on a vessel movement, it is important to quantify the flow parameters and then to evaluate its effects on the vessel itself and the trajectory of its movement. It is necessary to evaluate the fairway parameters according to the flow parameter changes.

Based on the quantification of parameters of the flow effect on the vessel movement, it is possible to correct the operating parameters of existing water structures. If the flow, as a result of a manipulation at an operated regulation hydropower plant (HPP), endangers the safety of the vessel movement, it is necessary to correct the operational parameters of the HPP regulatory operations. The result of calculations and measurements can also be the recommendation for a more dynamic use of the HPP regulatory functions.

For the assessment of the interaction of the navigational operation and the operation of water structure objects is from the theoretical point of view needed to apply:

- theory of the steady nonuniform flow and the unsteady flow in open channels;
- theory of the wave impact on the vessel translational movement and its effects on moored vessels;
- impact theory of force action of the longitudinal and transverse flow components on the vessel movement.

The Gabčíkovo Water Structure (WS) is a type of a multi-purpose structure. Its regulatory power functions may be used only in a limited way, because the compensatory reservoir of the Nagymaros Water Structure, which was meant to create a WS system with the Gabčíkovo WS.

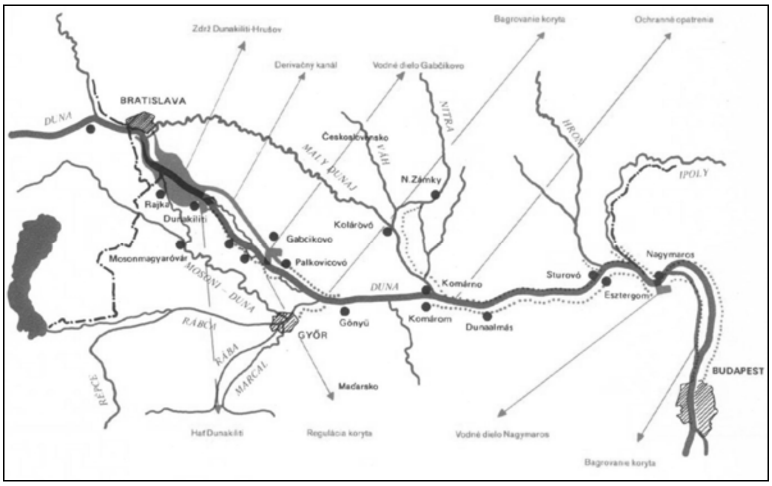

Fig.1. Scheme of the Gabčíkovo Water Structure

For enabling at least partially the use of the regulatory power functions of the Gabčíkovo WS without endangering the navigation safety on the Danube River, the research determined the limits of the safe regulatory power operation.
The work on the "Water Management Model of the Gabčíkovo WS Operation " primarily included the research of the effects of the regulation of the Gabčíkovo Hydropower Plant (HPP) on:

- water depth in the fairway in the sector between Gabčíkovo HPP and Komárno;
- wave parameters in the diversion channel;
- flow velocities in the diversion channel;
- water level regime in the reservoir;
- water level fluctuations below Gabčíkovo HPP.

According to the size of the paper, only the results of the research will be presented, which concern the effects of the regulatory operation of the Gabčíkovo HPP on the depth of water in the fairway. The research has shown that the decrease in water levels is a limiting factor for the determination of the regulation operation parameters.

2 Navigational Depths by the Regulation Operation

On a single water structure, which is not a part of a water structure cascade, or is the final downstream structure of a cascade, is dangerous in a particular range of discharges any decrease of discharge by the hydropower plant, which reduces the depths in shallow stages below the safety margin under the vessel (e.g., for the Danube River is the margin 2 dm). It causes the contact (impact) of the vessel with the bottom of the waterway, if it is found just in the critical location. The range of discharges is given by the range $[0 \div Q_{HR}]$ where Q_{HR} is the limiting the discharge, for which the navigational depths at 25dm are already assured.

For discharges above Q_{HR}, the regulation operation of the HPP is possible in strictly defined conditions. The regulation of the Gabčíkovo HPP may be realized only so that the decrease of the water depth in a limiting shallow does not cause the vessel hit the bottom, thus the navigation safety is not endangered. The shallows are not reported if the depth is more than 25 dm. Since the vessels are loaded according to the depth in the limiting shallow with a minimum margin, the regulation may be realized only for such discharges, for which the maximal expected decrease of discharge causes the maximal decrease of the water depth in the limiting shallow, so the depth does not fall below 25 dm.

The boundary discharge, for which the regulation cannot be realized, can be determined on the basis of an analysis of the measurement results and the calculations of the water levels below the Gabčíkovo HPP. For the determination of this discharge is necessary to know the actual parameters of the shallow stages. The determining discharge is the one, for which the navigation depth of 25 dm in the fairway as well as in the critical shallow is assured.

At the time of the calculations was the boundary discharge the discharge of Q= 1300 m^3s^{-1}, for this discharge the navigational depth of 25dm in the limiting shallow in rkm 1.796 was assured.

If the situation in the shallow is changed, the boundary discharge assuring the navigational depth of 25 dm will be changed as well. Following it would be necessary to update the model of flow and then calculate the minimal discharges, for which the regulation of the HPP can be realized, as well as all other calculations of allowed regulations.

3 Modeling–Scenarios of the Gabčíkovo HPP Operations

To examine and quantify the possibilities of the Gabčíkovo HPP regulation operation, a series of calculations of the water levels and discharges in the Gabčíkovo WS diversion channel and in the stage from Sap to Komárno has been made. The calculations were performed in the flow model of the stage from the Gabčíkovo HPP to Komárno created in the HEC-RAS.

Since it is not possible to determine in advance which type of discharge time changes at the Gabčíkovo HPP causes the exceeding of the navigation safety limits, plenty of calculations for different initial discharges and water levels and for manipulations of different sizes and durations have been made.

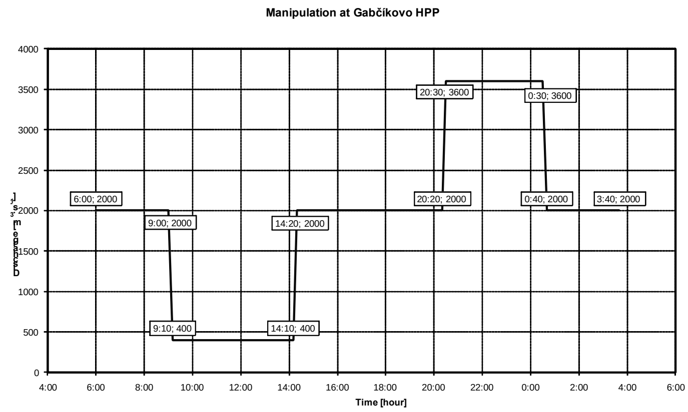

Fig. 2. Example of a manipulation scheme for the regulation of the Gabčíkovo HPP

The calculations in the stage from the Gabčíkovo HPP to Komárno:

1. Time course of water levels for discharge decrease manipulations on the Gabčíkovo HPP (they are limiting in terms of depths in the shallow stages) for over 800 combinations of parameters (starting discharge, discharge decrease, duration of discharge decrease, discharge increase to the initial starting discharge) with an evaluation for the limiting shallow in rkm 1796;
2. Water level decreases under the Gabčíkovo HPP for different combinations of the parameters;
3. Courses of water levels and discharges for typical regulation operation scenarios with decrease and subsequent increase of the discharges such as shown in Fig. 2;
4. Necessary volumes in the reservoir for the single manipulations.

4 Regulation with Discharge Decrease on the Gabčíkovo HPP

Over 800 discharge regimes were simulated in the simulation model of the Danube River developed in HEC-RAS by the Water Research Institute (VÚVH). Water level courses were calculated for different combinations of manipulations with different durations of the manipulations (30, 60, 90, 120, 150, 180, 240 min), with different amount of the regulation discharge decrease (dQ = 100÷1.700 m^3s^{-1} with a step of 100 m^3s^{-1}) and with different discharge under the confluence of the outlet channel and the old river bed before the time of the manipulation (1350 ÷ 2250 m^3s^{-1}). Before assessment of the results, the limiting shallow had been based on an analysis of the reports on discharges and water stages at the water gauges in Devín, Medveďov and Komárno and reports of the batter service for the period of 01.2007– 01. 2008.

The limiting shallow in the stage between SAP to Komárno is the shallow in rkm 1796. The Medveďov rating curve shows that, for a steady state of the flow, a navigation depth of 25 dm for a discharge of 1260 m^3s^{-1} is assured. According to the calculated rating curve in rkm 1796.02 this discharge corresponds to the water level of 107.50 m a.s.l. It is the limiting water level, under which is the depth below the required 25 dm of depth.

Similar parameters has the shallow in rkm 1735, but this is at such a distance below the place of the discharge manipulation at the Gabčíkovo HPP that the decrease of the water level at the shallow is due to a downstream flattening of the wave smaller.

For the same manipulation at the Gabčíkovo HPP, the greatest water level deformations are directly under the Gabčíkovo HPP and they decrease gradually along the stream. Although the water level decrease between rkm 1796 and Sap shows higher values than in rkm 1796, due to the depths in the stage between SAP and rkm 1796 it is reasonable to assume that the situation of navigational depths during the regulation at this stage is not worse. For an example at rkm 1808 is the decrease of tens of centimeters higher for extreme manipulations (the discharge decrease of 1500 m^3s^{-1} during 4 hours), however the depth in this stage is more than 1 m higher (according to the riverbed geometry used in the model).

5 Conclusions

The current regulations of the operation manual concerning the operation control according the water level and discharge regulation are based on the assumption that the possibilities of regulation operation are significantly limited. This is adapted to the limits of regulation operation (water level regulation in Čunovo reservoir and at Medveďov). Limiting are the conditions of permitted fluctuations in Medveďov (maximum 10 cm, or 30 cm for discharges over 1700 m^3s^{-1} and for maximal discharge decrease of 300 m^3s^{-1} during 1 hour maximum), because the limits of water levels in the reservoir allow significantly greater regulation.

The problem of the control according to these criteria is the fact, that the operation manual does not define the relationship between the regulation parameters and the allowed water level decrease in the stages below Sap (with the exception of the regulation -300 m^3s^{-1} for 1 hour, which is also inaccurate). The mentioned shows that it is not possible to plan the manipulation in advance, because the water level decrease this regulation causes is not known.

Such a kind of the control places considerable demands on the experience and the correct estimation of the on the day before proposed operation. The uncertainty grows with forecasting errors at the key water gauges. An often consequence is the operation under conditions which are not optimal in terms of utilization of the effects of the Gabčíkovo HPP.

The results of the simulations at the hydrodynamic flow model indicate the possibility of significant changes in the way the Gabčíkovo HPP is operated and make it more effective. An example to the evaluation is shown in Fig. 6.

Evaluation of the simulation and calculation results

1. The range of the regulation operation possibilities with respect to the criterion of maintaining the navigability depths is considerable. The regulation is possible for an example at a steady flow of 1550 m^3s^{-1} under Sap (1730 m^3s^{-1} in Devín) with a discharge decrease of 500 m^3s^{-1} during 1 hour. The water level in rkm 1796.02 drops to 107.71 m a.s.l. (depth of 27 dm in the fairway). For comparison with the currently valid limits – a possible regulation at the discharge of 1700 m^3s^{-1} during 1 hour is the discharge decrease of 1100 m^3s^{-1} (presently allowed discharge decrease at the discharge of 1700 m^3s^{-1} during 1 hour is only 300 m^3s^{-1}).
2. The speed of shutting down the Gabčíkovo HPP has a negligible effect on the extreme decrease of water levels in rkm 1796 (Fig. 4).
3. The simulations of the complete regulation cycle show that the water level fluctuations cause secondary complications for the navigation - vessels moored directly to the bank or the lock approach wall or vessels anchored near the Gabčíkovo HPP must have continuous on-board service for adjusting the length of the anchor ropes. It does not apply to the vessels moored to pontoons.

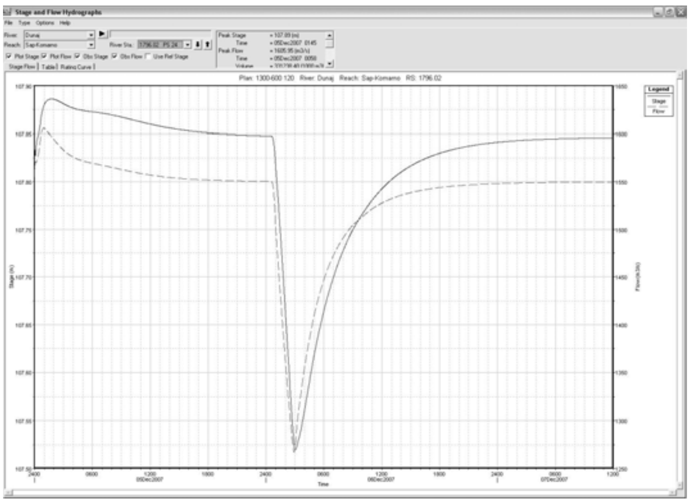

Fig. 3. Example of the unsteady flow calculation output for manipulation at the Gabčíkovo HPP

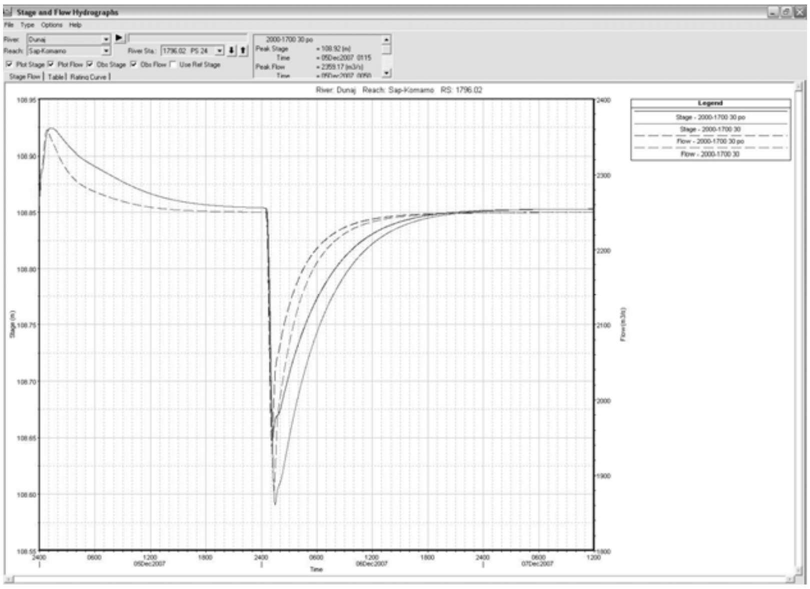

Fig. 4. Graph of time development of the water level in rkm 1796.02 for the regulations with various times of the Gabčíkovo HPP shut-down (10 s, 10 min)

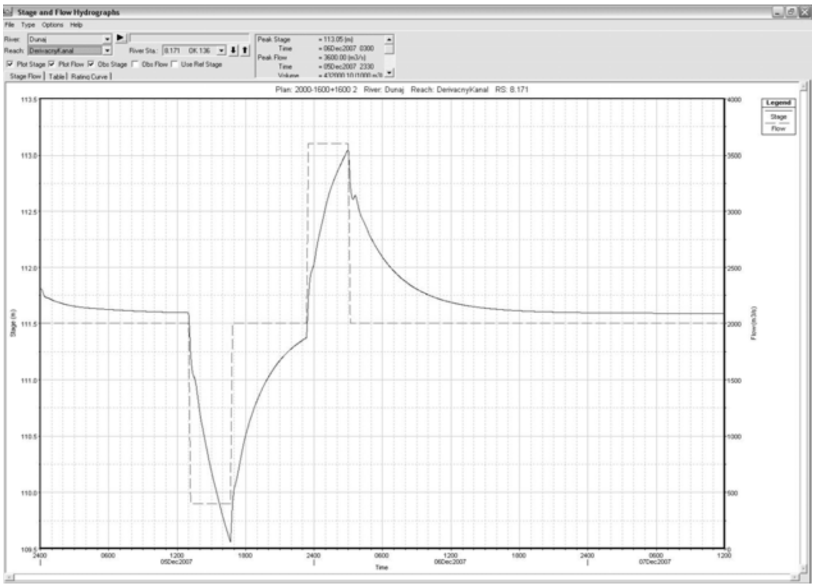

Fig. 5. Graph of the time development of the water level below the Gabčíkovo HPP according the regulation shown in Figure 2

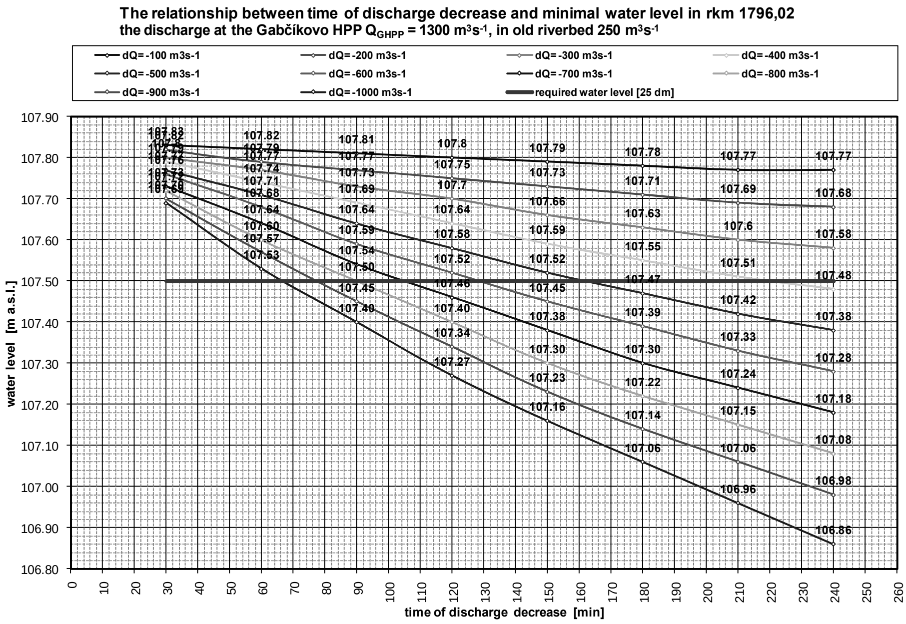

Fig.6. The relationship between the time of discharge decrease and the minimum water level in rkm 1796.02 - an example of the calculation results; the discharge at the Gabčíkovo HPP $Q_{GHPP} = 1300\ m^3s^{-1}$, in the old riverbed $250\ m^3s^{-1}$

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A81**

Modeling of Fish Passes in Conditions of Slovak Streams

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Abstract. Mandatory standards for design of fish passes at water structures do not exist in Slovakia. Therefore it has been necessary to prepare a standard for mentioned problems. There was a physical research on models in hydraulic laboratory of Water Research Institute. Whereas it is difficult to model all existing variants of fish passes and it is especially difficult to measure velocity field or directions of velocity vectors correctly, it was necessary to cooperate with mathematical model. River2D - a two dimensional model was selected for mathematical modeling. It is depth averaged hydrodynamic and fish habitat model developed specifically for use in natural streams. Geometry of fish passes has been designed for conditions of Slovak streams, it means for fish zones of trout, grayling, barbell and bream. All variants of designed fish passes were simulated by this mathematical model and results were compared with results from physical model.

Keywords: Fish passes, physical research, mathematical modelling

1 Introduction

This article is a partial result of cooperation between the Department of Hydraulic Engineering and the Water Research Institute in Bratislava on the task No. 6838 “Model research of the boulder meander fish passes as a part of the small hydro power plants”. Our department was responsible for the mathematical modeling of selected fish pass types. The Water Research Institute led by Ing. Filip Rebenda, the executive researcher of this task, made a physical research in the hydraulic laboratory simultaneously to the mathematical modeling.

This cooperation is trying to find a possibility of two dimensional software applications for the given problems, as well as to determine the conformity of obtained results, but especially to plot the estimated velocity field, which is almost impossible to measure on the physical model. After the verification of these assumptions, in the future it would be possible to apply modifications in the fish pass geometry directly into the mathematical model without the modifications of the physical model. The physical model would be used only for the new variants or potential problematic areas.

For the mathematical modeling the River2D freeware (a two dimensional software), which was developed at the University of Alberta in 2002, was selected. This program is especially suitable for the natural streams. It enables hydraulic simulations and includes the fish habitat as well. By the means of this model was simulated water level and velocity regime for the chosen fish pass types. On the basis of the water level regime the reached depths were calculated and on the basis of velocity regime from the velocity vectors the flow direction was plot, (clearly visible rest areas, so called refugium, which are important for migratory ichthyofauna).

2 Input Data for Mathematical Models

Data for the composition of the mathematical models of selected fish pass types were:

- river bed geometric arrangement, in cross and also longitudinal direction (Fig. 1.), whereas on the physical model were tested fish passes with straight and curved barriers with following variants of geometry:
    - a) basic shape (bottom basin is flat, between basins is a “chute” in bottom),
    - b) sloped bottom (fish pass river bed is sloped along whole its length),
    - c) basic shape + baffles of width 0,2 m (same like a) and to the barriers are added “baffles” of width 0,2 m),
    - d) sloped bottom + baffles of width 0,2 m (same like b) and to the barriers are added “baffles” of width 0,2 m),
    - e) basic shape + baffles of width 0,4 m (same like a) and to the barriers are added “baffles” of width 0,4 m),
- measured values of discharge and depths for specific variant, these were used as the model boundary conditions (Table 1, Fig. 2.),
- River2D User Manual.

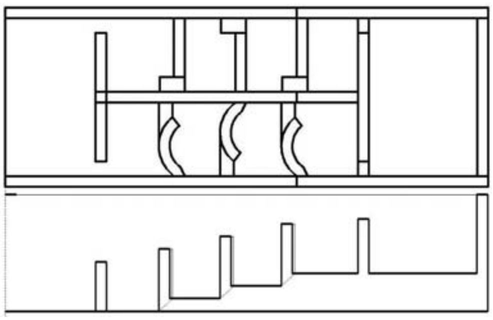

Fig. 1. Example of tested fish pass geometry [1]

Table 1. Summary of measured variants [1]

Geometry	Trout Q ($m^3.s^{-1}$)	Trout y (m)	Grayling Q ($m^3.s^{-1}$)	Grayling y (m)	Barbel Q ($m^3.s^{-1}$)	Barbel y (m)	Bream Q ($m^3.s^{-1}$)	Bream y (m)
Basic configuration	rectangular							
	0.220	0.3	0.313	0.4	0.265	0.4	0.287	0.5
	0.327	0.4	0.398	0.5	0.361	0.5	0.359	0.6
	0.491	0.5	0.497	0.6	0.431	0.6	0.483	0.8
	0.632	0.6	0.705	0.8	0.635	0.8	0.678	1.0
	curved							
	0.190	0.3	0.316	0.4	0.274	0.4	0.260	0.5
	0.309	0.4	0.398	0.5	0.352	0.5	0.300	0.6
	0.434	0.5	0.506	0.6	0.442	0.6	0.428	0.8
	0.567	0.6	0.703	0.8	0.676	0.8	0.618	1.0

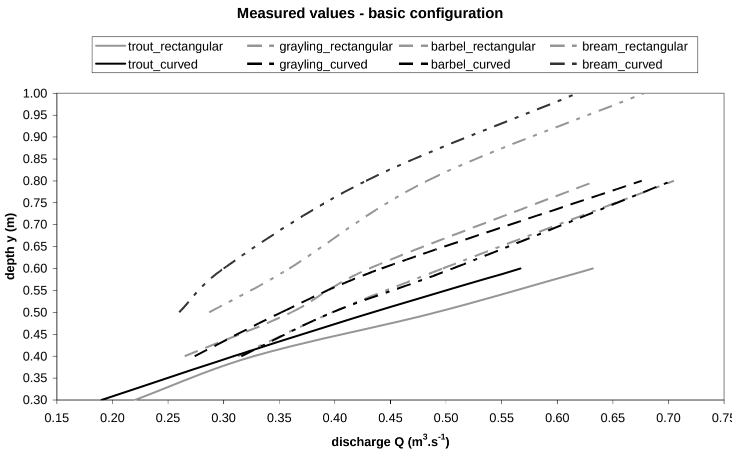

Fig. 2. Plotted values from Table 1

3 Procedure of Mathematical Modeling

For the problems of the water level and velocity regime solution for various designed geometries of the fish pass river beds was selected River2D software. The River2D is a two-dimensional depth averaged model of the river hydrodynamics and the fish habitat, developed specifically for the use in natural streams and rivers. It is a Finite Element model, based on a conservative Petrov-Galerkin upwinding formulation. The hydrodynamic component of the River2D model is based on the two-dimensional, depth averaged St. Venant equations expressed in conservative form. This model is used not only for the hydraulic simulations but also for modeling of the existing fish habitat [2, 3, 4, 5].

This program includes these submodules [2, 3, 4, 5]:

- R2D_Bed – input topography of simulated river bed,
- R2D_Mesh – topography modification, computational triangular mesh generation,
- R2D_Ice – intended for defining and editing ice topography files (ice cover of water surface) for the use in the River2D program (it was not used for this task),
- River2D – in this module is possible to set the parameters for ichthyofauna, e.g. weighted usable area (WUA), and here proceeds the computational process.

On the basis of given results from physical modeling the fish passes, which covered all fish regions (trout, grayling, barbel, bream), were simulated in the mathematical model. Other results will be supplemented continuously with physical modeling.

The process of creating the mathematical model for all designed fish passes was following:

1. Creating the geometry for the mathematical model on the basis of the physical model shape;
2. Defining the modeled area;
3. Entering the boundary conditions;
4. Generating the computational mesh with accepted mesh quality index (QI);
5. Creating the River2D input file, which is a basic file for the River2D simulation module;
6. Simulating the given variant;
7. Interpreting the results (depths - y, velocities - v) of the mathematical model in the measured points from the physical model.

3.1 Geometry

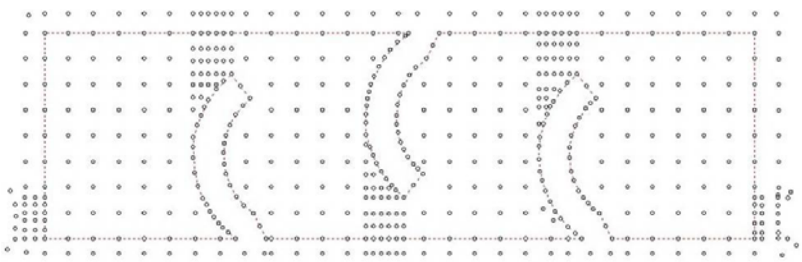

Fig. 3. Fish pass geometry (file *. *bed*) with exterior boundary loop

On the basis of the physical model geometry a point file, which characterized this model in real size, was generated. Consequently the file was modified to a form, which is possible to import into the submodule R2D_Bed, it means the points were numbered, the roughness coefficient (concrete fish pass river bed) was set and to the file a suffix *.*bed* was added. After the opening of such a file in R2D_Bed exterior boundary loop was set, it means real edge of the fish pass with barriers.

3.2 Grid and Boundary Conditions

Modified file from the R2D_Bed was opened in the next submodule R2D_Mesh, where the boundary conditions were set (from physical model):

- inflow area – discharge at the upstream boundary,
- outflow area – water level at the downstream boundary set by elevation above see level.

Afterwards mesh was created on the basis of set points on the boundary of modeled area (so called boundary nodes) and set points inside of the fish pass river bed with such a density to reached the required mesh quality index (suitable QI index is in the range 0.1 – 0.5 [5]). The generated computational mesh is saved as a file with extension **.msh*.

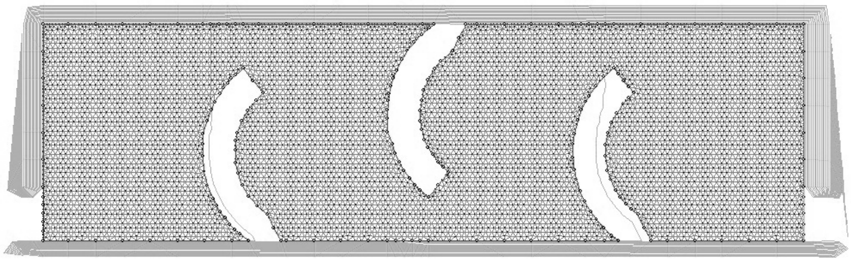

Fig. 4. Generated computational mesh on the fish pass geometry with mesh quality index QI = 0.453, consists of 6593 points (file **.msh*)

In R2D_Mesh submodule is a geometry file with generated mesh saving also as a River2D Input File, which has the extension **.cdg* and this file creates an input for hydraulic processes simulations of the modeled area in the River2D submodule. To make the iteration process possible for the software to start, it is necessary to set the initial condition – the estimation of the upstream water level (water level in the inflow of the area) by its saving. The more accurate the approximation is (it is not suitable to start iteration to the dry river bed) the smoother and easier will be the process of simulation in the River2D submodule.

3.3 Simulations, Results and their Interpretation

After file **.cdg* opening in the River2D submodule, the bounded modeled area (red line) and the boundary conditions (green – inflow, blue – outflow) are displayed. Here it is still possible to edit the boundary conditions, as well as to reset the initial conditions. The user chooses the steady or the unsteady flow (for simulated fish passes was chosen the steady state regime) and the calculation can start. It is possible to display the obtained results by contours, color shading or by vectors. In the case of uncertainty of the results is suitable to display generated mesh, which can be additionally refined in the problem area of the model to achieve appropriate results.

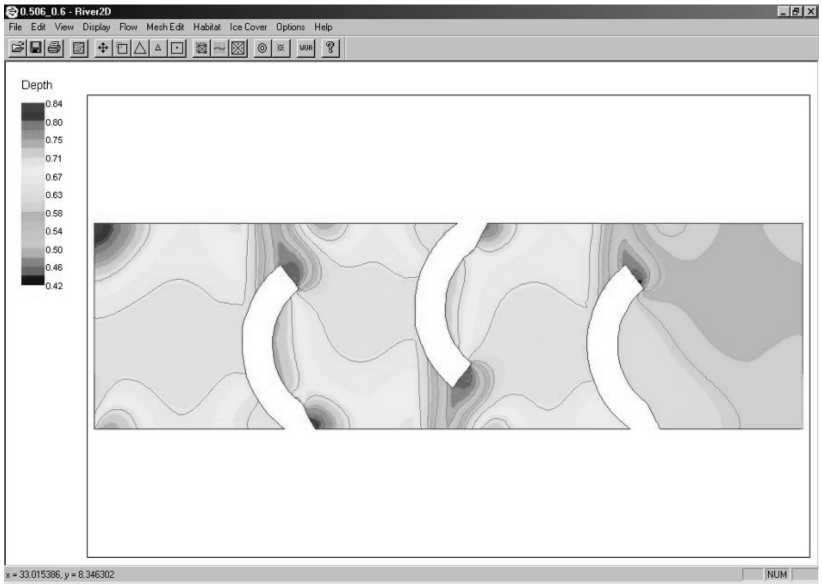

Fig. 5. Results displaying (depths) via shading color and contours

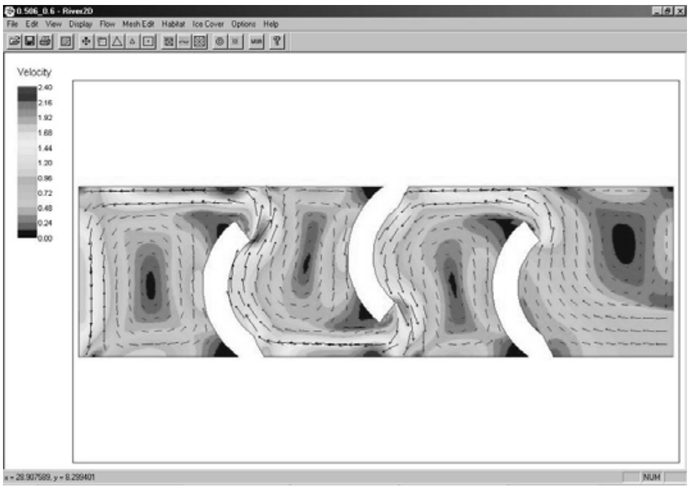

Fig. 6. Results displaying (velocity field) via shading color and vectors

The most important outputs from the mathematical model are the velocity fields, because in the fish pass (of any variant) complicated flow conditions occur. It is very problematic to measure these velocities on the physical model (direction of velocity vector is assumed (estimated)). Exceeding velocity values reached by the mathematical modeling can occur due to inappropriate generated mesh in a specific area. The more precise results can be reached by local adjustments to the generated mesh in the problem area, but this rearrangement prolongs the computation time. The results (depths, but especially velocities) are not constant along the whole course of the fish pass. It is only possible to evaluate them if the required parameters (maximum velocity, minimum depth) are reached in the areas, which are problematic for the upstream migration of the ichthyofauna. Evaluated can also be the ratio of the fish pass area, in which fish have to develop maximum swimming velocity and the ratio of the area, which fish can use as a rest place (refugium) by migration through the fish pass.

5 Conclusions

Results interpretation from the mathematical model is in the points, in which were realized measurements of depths, velocities and their directions on the physical models of the fish passes at the Water Research Institute. On the basis of given coordinates of the measured points, result files were exported from the River2D submodule in these points for solved variants (Table 2).

Table 2. Example of result summary table for basic geometry, bream fish region

Average depth in basin [m]	0.47		
Discharge Q [$m^{-3}s^{-1}$]	0.287		
Number of measured point	v [$m.s^{-1}$] measured	v [$m.s^{-1}$] computed	Δ [$m.s^{-1}$] (v_{meas}-v_{comp})
1	0.7735	0.6463	0.1272
2	0.994	0.8492	0.1448
3	0.443	0.433	0.01
4	0.4695	0.4851	-0.0156
5	0.3705	0.2988	0.0717
6	0.946	0.8079	0.1381
7	0.6315	0.5697	0.0618
8	0.692	0.8546	-0.1626
9	1.1425	0.9269	0.2156
10	1.346	1.0725	0.2735
	y [m] measured	y [m] computed	. Δ [m] (y_{meas}-y_{comp})
H1	0.524	0.6431	-0.1191
H2	0.504	0.5729	-0.0689
H3	0.452	0.6038	-0.1518
H4	0.44	0.5796	-0.1396

River2D the selected two-dimensional mathematical model is a suitable additional device for the physical modeling of fish passes in conditions of Slovak streams and result files of velocity fields and vectors are excellent decision instruments, because they give complex review of the existing flow regimes in the fish passes. On the basis of these and also the future results functional relations, which will be helpful for the practical fish pass design in Slovakia, will be determined.

Acknowledgements

This work was supported by Slovak Research and Development Agency under the contract No. APVV-20-003705.

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A82**

Whitewater Course Design in Slovakia

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Abstract. The popularity of whitewater sports in Slovakia increased due to achievements of Slovak sportsmen at many championships including World Championship and Olympic Games. There are only two professional whitewater courses (Liptovský Mikuláš and Čunovo) in Slovakia where can our sportsmen train, moreover these two courses old fashioned – the courses are made with stationary obstacles, so the courses can be modified only in a very limited way. Presently is planned to build several new whitewater courses in different places in Slovakia (Trenčianske Biskupice – next to weir, Košice – directly in the Hornád River and Červený Kláštor – at Dunajec River in old riverbed). For successful whitewater course design a cooperation between watermen, who have own experiences and know how to place obstacles into river bed to achieve required parameters for individual wild water sports (kayak, canoe, raft, freestyle…), water management experts and hydraulic engineers.

Keywords: Whitewater course, basic parameters, hydraulic design

1 Introduction

Building of whitewater courses is closely associated with the Olympic Games, because canoe and kayak slalom are Olympic disciplines. A whitewater course is an artificial channel, in which obstacles are placed (stationary or movable) to achieve suitable flow conditions for wild water sports (slalom, rafting, freestyle, etc.) and to create areas for the gate attachments. The design of such a construction is very difficult as well as its realization and operation. The original stationary obstacles were replaced by the movable obstacles, which enable variability of the course and its difficulty as well as attractiveness for a wide range of water sports. The whitewater courses or their sections can be divided into competitive and training sections, according to their parameters.

The popularity of wild water sports (paddle sports) in Slovakia aroused from the success of our sportsmen at the Olympic games, World championship, etc. Therefore some municipal regions decided to build whitewater course areas in their regions. The following courses are planned:

- Trenčianske Biskupice – next to the weir on the Váh Cascade,
- Košice – directly in the Hornád River (part 1 and 2), part 3 will be a component of planed fish pass reconstruction near the weir Ťahanovce on the Hornád River
- Červený Kláštor – at the Dunajec River in old riverbed (branch).

These areas will create places for active relax and recreation and enable the possibilities for quality training of young generations.

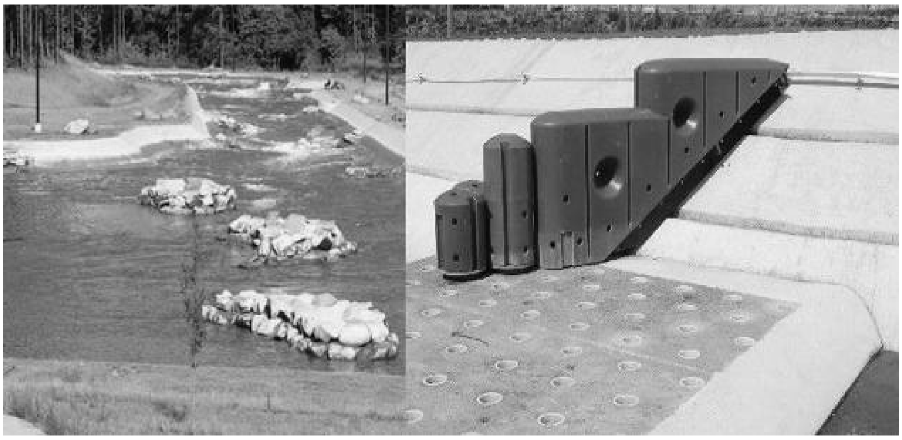

Fig. 1. Example of stationary and movable obstacles [1, 2]

2 Existing Whitewater Courses

History of the modern whitewater courses started in Augsburg (Germany) in 1972, where the first artificial whitewater course on the so called Eiskanal was created for the slalom canoeing in the Olympic Games in Munich. Its parameters are suitable for the international competitions and championships until today. The next Olympic whitewater course was built in 1992 for the Olympic Games in Barcelona, when it was included back into the olympic disciplines after a 20 year break.

2.1 Whitewater Courses in the World

In the next table are basic operational parameters of the world's most famous whitewater courses.

Table 1. Summary of the world's whitewater courses [3, 4, 5, 6]

City/State/Year	Length l (m)	Gradient H (m)	Discharge Q ($m^3.s^{-1}$)	Width b (m)	Depth y (m)	Slope i_o (%)
Augsburg,Germany, 1972	305	3.2	10	6 – 15		
Dickerson, USA, 1991	274.32		5.7 – 17	12.19		
Barcelona, Spain, 1992	340 (300)	6.5	10 (12)	5 – 17		
Atlanta,USA, 1996	600	12	25 – 70	30		
Sydney, Australia, 2000	320	5.5	14	8 – 14	0.8–1.2	
Athens, Greece, 2004	270	6	17.5		1.9	1 - 2
Zoetermeer, Netherlands, 2006	300	5	4.25 – 20.25	13 – 20	1.6	
Peking, China, 2008	280	5.88	17.5	10	1.2	2.1

2.2 Whitewater Courses in Slovakia

Construction of the whitewater courses in Slovakia is closely associated with Mr. Ondrej Cibák, who also designed the whitewater course for Olympic Games in Barcelona.

The first artificial whitewater course in Slovakia (The Ondrej Cibák´s water slalom complex) has been put into the operation in 1978 in Liptovský Mikuláš. This course is situated on the left riverside of the Váh River from which it is supplied with required discharges. The course is created by a conduit channel, which ends in the starting basin with total length of 680 m. From the basin two separate courses, which unite approximately in the middle into one course and then continue back to the Váh River. The left course with easier difficulty is called the „Váh" and the right one, which is shorter and more difficult, is called the „Orava". The courses were artificially excavated but the round stones facing of the banks give to the channel river bed a natural look. The whole complex of the whitewater course is supplied by the discharge from the Váh River, thus the water levels in the course depends on the water levels in the Váh River. The best flow conditions are in the months May and June, but the operational period of the complex is since the April until the October [7].

Fig. 2. Water slalom complex in Liptovský Mikuláš (empty and in operation)

The other whitewater course in Slovakia is situated at the Čunovo water structure and it was built as a part of the Gabčíkovo water structure. This whitewater complex is 20 km away from the centre of Bratislava. The complex is used for competitions and its closed circle consists of these parts [8]:

- The 20 m long and 40 m wide starting basin, with the depth of approximately 1,5 m. The left or the right courses are supplied with water from the basin according to requirements. In the basin ends also the return channel.

- Two competition courses – 2 channels, with combination possibilities, create five course variants with the difficulty for beginners as well as the best sportsmen. The movable obstacles are placed in the course river bed too. They enable to change configuration and difficulty of the course.

- The boat lift enables the sport boat transport from the end of the course back to the starting basin. So the sportsmen can reach the start without getting off the ship after finishing of the course.

- The return channel is 255 m long, and about 14 m wide. The channel is easily accessible from the dock. The return channel is suitable for the preparation of the sportsmen before the competition and also for the training of the beginners and children.

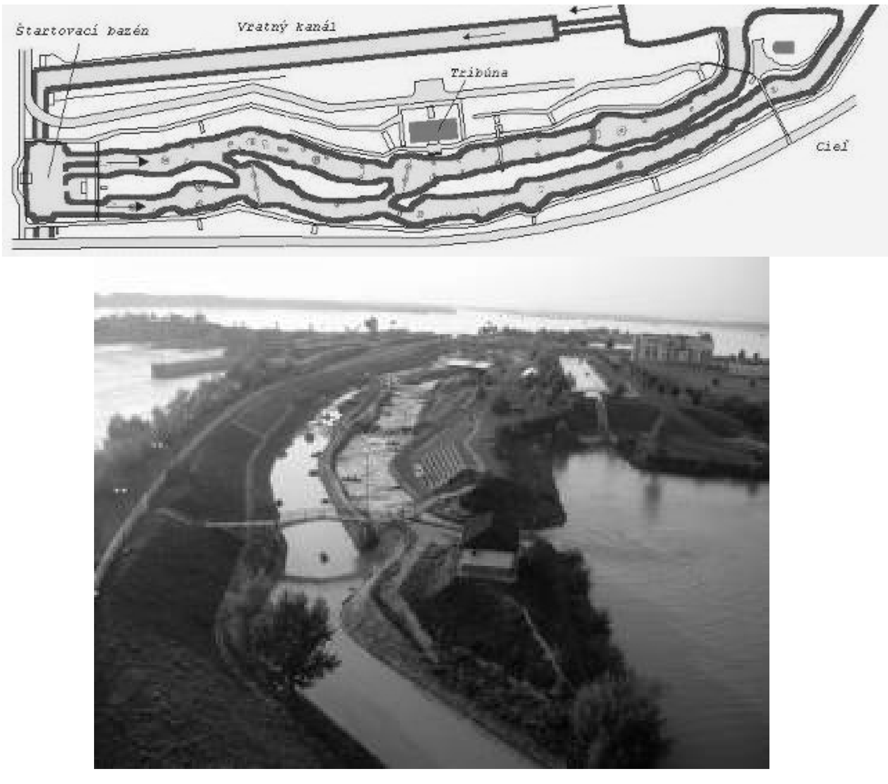

Fig. 3. Water sports complex in Čunovo

Table 2. Whitewater courses in Slovakia [7, 8, 9]

River/City	Description	Length l (m)	Gradient H (m)	Discharge Q ($m^3.s^{-1}$)	Width b (m)	Depth y (m)	Velocity v ($m.s^{-1}$)
Váh Lipt. Mikuláš	course no. 1	450	7.5	12 – 15		0.5 - 1.5	
	course no. 2	350					
Dunaj Čunovo	right course	460	6.6	7 – 12			
	left course	356		7 – 22			
Váh Žilina	placed in biocoridor	450	4.76 – 8.4	6 – 8	5 – 8.3		0.8 – 1.5

The next whitewater course is a recently established whitewater course at the biocorridor of the Žilina water structure, which is used only for the training purposes. It is situated in the lower part of the biocorridor, which leads into the Váh River under the water structure. It is operational since the April until the October (7 months), 5 days per week, 4 ours per day.

Fig. 4. Biocoridor at the Žilina water structure with whitewater course for slalom training

3 Design Parameters

The whitewater course design parameters take into account the requirements on the whitewater course – the character of the course (race or training), the character of the water sports performed on the course (slalom, rafting, freestyle), the sportsmen abilities (professional, recreational) as well as the safety regulations. To achieve the complex whitewater course requirements, correct placement of obstacles, local widenings and drop ditches is necessary. Therefore, for the whitewater course design it is necessary to cooperate with experienced watermen, with the knowledge of the wild water sports rules.

Recommendations for design parameters of whitewater courses [10]:

- Course length – minimum 300 m, because of international competitions and championships (Olympic Games, World Championship…);
- Cross section – simple shaped (trapezoid, U – shape);
- Profile width – optimum width should be 10 - 12 m, whereby it must not be less than 8 m, what is double of slalom ship length;
- Water depth – minimum 0.4 m, this dimension secures safe swimming, average depth should be approximately from 0.75 to 0.9 m, what serves for safe Eskimo roll, for freestyle kayak the depth should be approximately 1.5 m;
- Flow velocity – approximately 2 $m.s^{-1}$, for beginners around 1.4 – 1.7 $m.s^{-1}$, whereby some areas with higher velocity can occur;
- Roughness – channel surface should be the smoothest considering the possibility of ships damaging, suitable material is concrete with the smooth surface arrangement to the concrete will be embedded or stationary obstacles or platforms with openings for movable obstacles.

Table 3. International Canoe Federation (ICF) Olympic Standard Projects requirements [11]

Parameter	Magnitude
length l (m)	250
width b (m)	7
depth y (m)	1.2
discharge Q ($m^3.s^{-1}$)	14
gradient H (m)	5

4 Planned Whitewater Courses in Slovakia

After the success of Slovak sportsmen the interest in the whitewater course building is rising in Slovakia. Presently, there are three new whitewater courses planned in Slovakia.

The first course is planned in Trenčianske Biskupice, on the left bank of the Váh River next to the weir. The left bank's inundation area provides sufficient space for a whitewater course. The course is planned to consist of two sections, the upper section is the race section with the length of 325 m, the head of 5.5 m, the width of 7.5 m and the average depth of 1.0 m. The lower stage is planned as a training course with the length of 300 m, the head of 3.65 m, the width of 8 m and the average depth of 1.0 m, this stage includes also two play spots for the freestyle training. Both sections should be supplied with water from the reservoir above the weir. The maximum discharge is 10 $m^3.s^{-1}$. The course is planned to fulfill also the function of a fish pass for the weir.

The second is the whitewater course complex in Košice at the Hornád River. It consists of three sections. The first stage is a course directly in the riverbed of the Hornád River with the length of 200 m, the head of 1.48 m, the width of 7 m and the average depth of 0.7 m and planned discharge of 8 $m^3.s^{-1}$. The second stage is a freestyle play spot above the first stage in the Hornád river bed. The third stage is the course in the new fish pass of the reconstructed Ťahanovce Weir. The courses should be used for trainings and junior competitions.

The third course is planned in Červený Kláštor in the old river bed of the Dunajec River. The course consists of lateral intake, starting channel with the length of 360 m and the course itself with the length of 200 m, the head of 1.2 m, the width of 7 m and the average depth of 0.7 m and planned discharge of 5–8 $m^3.s^{-1}$. This whitewater course is planned for recreational activities and junior competitions.

Fig. 5. Visualization of planned white water complex at the Hornád River

5 Conclusions

Every whitewater course design is unique, because the particular design has to be adjusted to the particular conditions of the course's locality. It is important to take into account the race course parameters defined by the ICF, because the use of such a course only for training purposes would not be a cheap investment. Therefore it is advised to design the new courses so that they fulfill the required parameters defined by the ICF. The design and placement of the course elements such as the obstacles, basins, etc. should be consulted with experienced wild water sportsmen, who can give valuable advice and opinions for the design. The design should be verified on mathematical and physical models, so the hydraulics of the course will fulfill the requirements of the sportsmen and other users of the course.

The experience with whitewater design in Slovakia in following years could lead to forming of a methodology for construction of these sport structures.

Acknowlwgement

This work was supported by Slovak Research and Development Agency under the contract No. APVV-20-003705.

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A83**

The Model Research of the Water Intake Structure of the Dobrohošť Small Hydropower Plant

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Abstract. The Dobrohošť water structure had been build as a part of the Gabčíkovo water structure with the purpose of supplying old riverbed of the Danube River with water from diversion channel of the Gabčíkovo hydropower plant. This structure provides a small hydropower potential that can be utilized in small hydropower plant. This fact had been considered at the construction of this structure – it includes intake structure and a part of a water conduit of the planned small hydropower plant. According to actual project of this hydropower plant is presently its construction prepared. To verify projected scheme of water intake (intake structure, pressure conduit and surge chamber) a physical model of it has been build in the hydraulic laboratory of the Department of Hydraulic Engineering. The research on this model should have verified design parameters of the intake structure.

Keywords: Physical research, hydraulic modelling, small hydropower plant, intake structure, pressure conduit, Froude number

1 The Dobrohošť Small Hydropower Plant

The intake structure of the branch system of old Danube river is situated in the distance of 1,8 km of the connecting dike of the reservoir.

Its purpose is to donate the water to the left-bank inundation area of the old Danube riverbed in the rkm 1840 -1820.This structure is created by three 12 meters wide weir fields, with 3.6 meters high flap gates, the intake is protected by a floating barrage. The capacity of the whole weir structure is 234 $m^3.s^{-1}$. The discharges into the branch system vary from 6 up to 150 $m^3.s^{-1}$, while the average discharge value is varying from 15 to 40 $m^3.s^{-1}$. According to the suitable conditions at this site, the energetic utilization of the hydro power potential has been planned since the construction of the whole structure. Therefore an intake object of a small hydropower plant (SHPP) and a part of the water conduit of the hydropower plant's intake with 4 m wide and 3 m high rectangular cross-section. The assumed capacity of the planned small hydro power plant was 40 $m^3.s^{-1}$ [1].

Fig. 1. The Dobrohošť intake structure

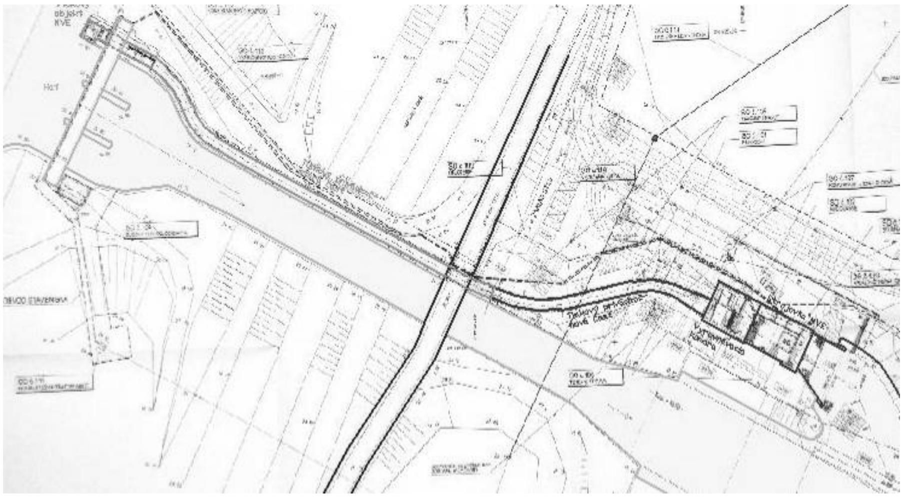

Fig. 2. A scheme of the planned Dobrohošť small hydropower plant

In 1998 a project of the Dobrohošť small hydropower plant has been developed [2]. Based on this project the Vodohospodárska výstavba company obtained a building permit for the construction of the SHPP. This project assumes the utilization of the existing intake structure of the power plant's conduit and the existing conduit itself (both structures are assumed to be reconstructed). The conduit is planned as a pressure conduit, while its orientation enables that the engine room of the power plant is situated directly in the left bank of the channel below the Dobrohošť intake structure that supplies the Danube branch system with water. The conduit leads to the surge chamber, in which the turbine intake is situated. The planned small hydropower plant consists of one horizontal Kaplan turbine with diameter of the turbine runner of 1900 mm and maximum discharge capacity of 25 $m^3.s^{-1}$ and the gross head of 8.69 m. The hydropower plant with these parameters has got the installed capacity of 1 800 kW and annual power generation of 11 873 MWh.

2 The Objectives of the Physical Research

The objectives of the model research on the physical model of the Dobrohošť small hydropower plant's intake can be formulated in the following points:

- verification of the suitability of the designed shape of the conduit intake;
- measuring and documenting the pressure conditions in the conduit during regular operation and during sudden failure of the small hydropower plant;
- measuring and documenting the friction and local losses;
- verification of the suitability of the designed small hydropower plant inlet including the surge chamber.

3 The Description of the Physical Model

The physical model of the water intake from the diversion channel and the intake of the planned Dobrohošť SHPP has been designed based on the Froude criterion of model similarity. The Reynolds number of the flow in the conduit is bigger than its limit value, thus the flow is in the auto-modelling area and the roughness has been modelled according the roughness coefficient model scale and so the keeping of the similarity for losses determination has been achieved. The geometric scale (length scale) was set for M_l=15 according to the space and discharge conditions available in the hydraulic laboratory. This scale is sufficient for creating flow conditions at the model that correspond with the real conditions and provides sufficiently precise measurement of the hydraulic parameters with measuring devices, which have been used. The physical model consists of following parts:

- part of the diversion channel with length of about 200 m and bed width of 10.5 m with discharge regulation and measurement at the Thomson triangular notch;
- three field weir in the right bank of the channel made of metal plates with a possibility of a discharge regulation and measurement at the Thomson triangular notch through one field next to the conduit intake structure;
- floating barrage in front of the weir and conduit intake made of plastic;
- intake structure of the small hydropower plant's conduit adjusted according to the projected design;

- conduit with total length of 236.5 m consisting of an original reconstructed part with length of 156.4 m and a new part with length of 80.1 m, including two reducing pieces and one arch piece with an angle of 90° made of Plexiglass;
- surge chamber with effective ground area of 129.6 m^2 at the end of the conduit, including emergency spillway with a total crest length of 32.8 m and spillway outlet flumes made of metal plates;
- turbine intake, with a discharge measurement and regulation in the conduit and a device for simulation of a sudden failure of the small hydropower plant.

According to the spatial conditions in the hydraulic laboratory, the directional arches of the conduit have been replaced by a straight conduit with one arch piece with an angle of 90°. This simplification of the real conditions has been considered for the determination of the hydraulic losses in the conduit of the SHPP.

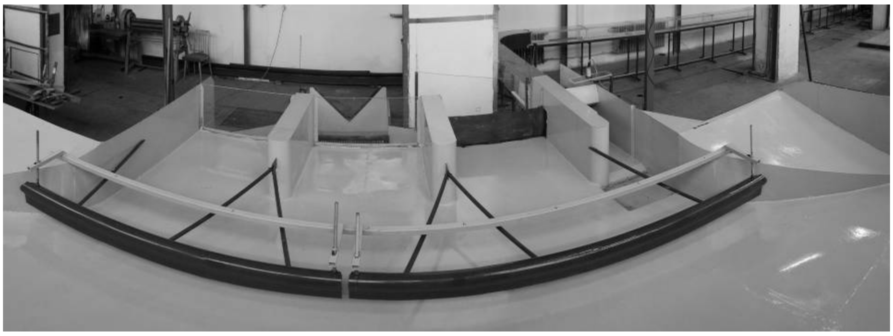

Fig. 3. A part of the model –the three field weir, the floating barrage and the conduit intake

4 The Measurements on the Physical Model

The performed measurements on the physical model can be divided in two groups. The first are steady state measurements and the second are the unsteady state measurements. The measurements have been ordered into measurement series according to the required operational conditions of the SHPP(Table 1). The measured hydraulic parameters were discharges, spot velocities, water levels and pressures in the conduit.

The steady state has been defined by the water level in the diversion channel (in the range from maximum down to minimum operational water level), the discharge in the diversion channel of 33 $l.s^{-1}$, discharge through the weir field next to the conduit intake of 0 $l.s^{-1}$ versus 28.69 $l.s^{-1}$ (25 $m^3.s^{-1}$ in reality) and the discharge through the hydropower plant of 28.69 $l.s^{-1}$ (25 $m^3.s^{-1}$ in reality). The velocity fields at the intake, water levels in the intake and the surge chamber and pressures in the conduit were measured for the steady state. The deformations of the velocity fields due to the floating barrage, the hydraulic losses in the conduit, the pressure conditions in the conduit and the suitability of the conduit intake have been evaluated based on the measurement results.

The unsteady state should simulate a sudden failure of the Dobrohošť SHPP during common operation states. Therefore, the unsteady state is based on the steady state conditions. The sudden failure of the hydropower plant has been simulated by cutting/off the discharge from the value of 28.69 $l.s^{-1}$ (25 $m^3.s^{-1}$ in reality) down to zero in the time of 3.35 seconds (13 seconds in reality). The unsteady state measurements were aimed to record the water level development in the surge chamber, verify the capacity and functionality of the emergency spillway and determine the pressure increase in the conduit due to the hydraulic hammer.

Table 1. The summary of the realized measurement series

Series tag	Operation state	Flow through	Water level in the diversion channel
1/SHPP/130.70	Steady	SHPP	130.70 m a.s.l.
1/ SHPP /131.10	Steady	SHPP	131.10 m a.s.l.
1/ SHPP /130.10	Steady	SHPP	130.10 m a.s.l.
1/ SHPP +weir/130.70	Steady	SHPP and one weir field	130.70 m a.s.l.
1/ SHPP + weir/131,10	Steady	SHPP and one weir field	131.10 m a.s.l.
1/ SHPP + weir/130.10	Steady	SHPP and one weir field	130.10 m a.s.l.
2/ SHPP /130.70	Unsteady	SHPP	130.70 m a.s.l.
2/ SHPP /131.10	Unsteady	SHPP	131.10 m a.s.l.
2/ SHPP + weir/130.70	Unsteady	SHPP and one weir field	130.70 m a.s.l.
2/ SHPP + weir/131.10	Unsteady	SHPP and one weir field	131.10 m a.s.l.

4 Conclusions

Following conclusions of the research on the three-dimensional physical model of the intake part of the Dobrohošť SHPP have been deducted from the performed measurements:

- The conduit intake of the Dobrohošť SHPP is not suitable for the planned operation because, for the minimal operational water level in the diversion channel (130.10 m a.s.l. in reality), the flow in the conduit intake turns from sub-critical to super-critical, which leads into a significant loss of pressure in the conduit and a substantial increase of hydraulic losses and thus the loss of the required head. These conditions are inadmissible for the small hydropower plant's operation.

For the water levels in the diversion channel in the range from 131.10 to 130.70 m a.s.l. is the conduit intake designed sufficiently, and the hydraulic losses and the pressure conditions in the conduit are suitable for the operation of the SHPP.

- The floating barrage in the front of the weir and the conduit intake does not have a substantial negative effect on the flow conditions in the intake and the conduit.

- In the intake area behind the floating barrage no major vortices as well as no significant air drawing into the conduit due to the effect of the floating barrage were detected.

- The surge chamber, which was tested by the sudden failure of the SHPP, has shown itself as safely designed, with a sufficient capacity of the emergency spillway.

The surge chamber sufficiently protects the pressure conduit from the pressure increase caused by the hydraulic hammer.

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A84**

Hybrid Method for Optimal Design of Water Distribution System

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Abstract. Optimal design of water distribution networks belong to large combinatorial optimization problems. Various methods have been developed and applied for the design of water distribution systems. Still there is some uncertainty about finding a generally reliable method. In the proposed method two algorithmic techniques are used – linear programming (LP) and genetic algorithm (GA). A combination of these two methods with the aim of eliminating the limitations of LP (which is not suitable for networks with loops) is used. It was investigated that proposed method provides results more reliable in terms of closeness to a global minimum. The method may be applied for use in the design and rehabilitation of both drinking water and pressurized irrigation systems.

Keywords: Water distribution system, optimization, heuristic method, linear programming

1 Introduction

The optimal design problem is to find the water distribution systems component characteristics (e.g., pipe diameters, pump heads, reservoir volumes) that minimize the total system cost. Various algorithms ranging from artificial intelligence to the optimization domain were applied. Alperovits and Shamir [1] presented a linear programming gradient for optimizing a water distribution network. Kessler and Shamir [12] used the linear programming gradient method as an extension of this method. It consists of two stages: an LP problem is solved for a given flow distribution, and then a search is conducted in the space of the flow variables. Later, Fujiwara and Khang [9] used a two-phase decomposition method extending the method of Alperovits and Shamir to non-linear modeling. Also, Eiger, *et al.* [6] used the same formulation as

Kessler and Shamir. Nevertheless, these methods fail when solving problems of large looped systems. The application of stochastic or so-called heuristic optimization methods to WDN optimization can be traced back to the late 1990s and they still remains as a vital tool for WDN optimization. Simpson, et al. [20] used a simple genetic algorithm in which each individual solution from the population of solutions is represented by a string of bits with identical lengths. The simple GA was then improved by Dandy, *et al*. [5] using the concept of the variable power scaling of the fitness function, an adjacency mutation operator, and gray codes. Savic and Walters [18] also used a simple GA in conjunction with an EPANET network solver. Many different heuristic techniques have been applied to the optimization of a water distribution system. Methods as simulated annealing [13], [4]; an ant colony optimization algorithm [14]; a shuffled frog leaping algorithm [8] and a harmony search [10].

Reca, *et al* [17] evaluated the performance of several meta-heuristic techniques. He compared these techniques by applying them to medium-sized benchmark networks. The results which he obtained for the Hanoi network (after ten different runs with five heuristic search techniques) varied from 6,173,421 to 6,352,526. These results differ by 1.5 – 4.5% from the known global optimum for this task, which is a relatively large deviation for such a small problem. It should be noted that real life problems never get the privileges afforded the Hanoi or other benchmark network.

How closely any solution from the computation of the optimization in a particular case could achieve a global optimum depends on the complexity of the problem. Generally, a more complex problem is a problem for which a larger or more complicated search space must be explored. The reduction of the search space is the main approach of this paper on how to design a method which has a greater potential to find results which are really close to a global optimum. A new hybrid GA-LP approach using a genetic algorithm and linear programming is proposed in this study for determining the least-cost design of a water distribution system. It is built on the advantages of both the deterministic and heuristic methods. The GA method is used in the outer loop of the proposed algorithm, which is intended for decomposing a complex looped network in a group of equivalent branched networks. LP is then used in an inner loop to solve each branched network and provide a minimum-cost design. After evaluating a high number of possible branched networks, an optimal solution is found for the original looped network.

The advantage of using this hybrid method consists in the fact that a GA in this case has a much smaller searching space than in a case when usually formulation of heuristic search (for example GA methodology) is used alone, which has a great impact when trying to achieve better results. How this is achieved will be explained later after introducing the details of the methodology.

2 Methodologies

2.1 Linear Programming - Deterministic Part of Proposed Method

The linear programming method has long been accepted as an approach for the optimal selection of diameters for pipes in branched networks, e.g., in the design of irrigation systems. The mathematical formulation of this problem is as follows:

$$X_{11} + X_{12} + ...+ X_{1n} = B_1$$

$$X_{21} + X_{22} + ...+ X_{2n} = B_2$$

etc.

$$X_{m1} + X_{m2} + ...+ X_{mn} = B_m \tag{1}$$

$$A_{11}\, X_{11} + A_{12}\, X_{12} + ...+ A_{xy}X_{xy} \leq C_1 \tag{2}$$

etc.

$$D_1\, X_{11} + D_2\, X_{11} + ...+ D_j\, X_{mn} = \min \tag{3}$$

The solution has to comply with inequalities:

$$X_{11} > 0;\ X_{12} > 0 \text{ etc. up to } X_{mn} > 0 \tag{4}$$

where: X_{ij} unknown length of the selected diameter j on section i. The allowable diameters must satisfy the velocity conditions on the section,
B_i total length of the section,
C_i allowable total loss for the constraint – described below,
D_i unit price of a pipeline with the diameter number i.

When linear programming is applied in order to solve the optimal design of pipeline networks, the unknown will be the lengths of the individual pipeline diameters on the section. In conditions (Eq. 1) the requirement that the sum of the unknown lengths of the individual diameters in each section has to be equal to its total length is enforced. The second type of equations - constraints (Eq. 2) - represents the condition that the total pressure losses in a hydraulic path between a pump station or tank and every critical node (the end of the pipe network; the extreme elevation inside the network) should be equal to or less than the known value. This constraint is based on the minimum network pressure requirements needed for the operation of the system. There should be the same number of these constraints as there are critical nodes in the network. Given the minimization requirement for the investment costs, the objective function (Eq. 3) is the sum of the products of the individual pipeline unit prices and their required lengths. As will be described later, this system of the equation should be compounded automatically in the fitness function of the GA.

When multi-demand conditions in the LP model are incorporated, there will be a system of constraint (Eq. 2) for every demand pattern. When pumps are also included in a model, the main input parameter for a pump is its pump curve. The right sides of the constraints (Eq. 2) vary according to the pump's operating conditions. The head of the pump may be treated as an unknown variable.

2.2 Genetic Algorithms - Heuristic Part of the Proposed Method

In order to overcome the above-mentioned deficiencies of the linear programming techniques, heuristic optimization techniques have been introduced for solving the optimization of water distribution systems. Firstly, a genetic algorithms methodology, which is also used in this study, was applied. This is a search procedure inspired by the mechanics of natural genetics and natural selection. Its basic concepts are briefly summarized below; a good introduction to the subject is given by Goldberg [11].

The first step is to represent a solution to the problem by a string of genes that can take on some value from a specified finite range. This string of genes, which represents the solution, is known as a chromosome. Then an initial population of chromosomes is constructed at random. Genetic algorithms are implemented as a computer simulation in which a population of chromosomes evolves toward better solutions by means of genetic operators such as inheritance, mutation, selection, or crossover. At each generation, the fitness of each chromosome in the population is measured. The fitter chromosomes are more often selected probabilistically to produce offspring for the next generation. This process is repeated until some form of convergence in the fitness is achieved.

2.3 Hybrid GA–LP Approach to the Optimal Design of Water Distribution Systems

The proposed method is based on a combination of linear programming methodology and genetic algorithms. The main reason is that linear programming always finds the global optimum if it exists. But because LP is suitable only for solving branched networks, the GA method is used for decomposing a complex looped network into a group of branched networks. These branched networks are characterized by hydraulic behavior identical to a looped network on the condition of having identical diameters on the corresponding sections of the network. Identical hydraulic behavior means that there are identical flows in the corresponding pipes and identical pressures in the corresponding nodes in the original looped network and branched networks investigated. The decomposition of a looped network means that before the optimization, every loop is split in some demand node (or node in which a branch is connected) which is part of this loop. Linear programming is then applied for optimizing every branch network produced by GA from the original looped network, and GA is simultaneously applied for the evolution of the best splitting option.

The main aspect of the motivation to propose the new optimization algorithm presented is that the definition of a chromosome is in an obvious GA approach as long (in the sense of the number of genes) as many pipe sections are in the water distribution network. The definition of a chromosome proposed herein produces significantly shorter strings (which mean a smaller search space) for most networks because the number of genes will be the same as the number of loops in the network and not equal to the number of pipes. This means easier searching with better results.

An example of possibilities for splitting a loop is shown in Fig. 1. Let us suppose that all the parameters of the network in Fig. 1a) are already known (diameters, lengths, demands, flows, etc.), so we are not in the design stage for this network, but we can suppose that it already exists. The loop can be transformed to a branch layout without affecting the hydraulic behavior of the network (the flows and pressures remain the same) if the loop is split in the demand node in which the flows are entering from the two pipes connected to this node. There is one such node for every loop in the network. This node could be a hydrant or branch connection. In Fig. 1a) the original loop with a node in which there is a demand 5 lps is shown. So this is the node in which it is possible to break the loop without affecting the hydraulic behavior of the network in the current conditions. A loop could be split in this node in such a way that a "twin" node is introduced to the network; the identical elevation on it is assumed as in the original node (and de facto identical position). The original demand has to be split between these two nodes: the upper node will have in the first case demand set to 4 lps and the lower node at 1 lps (Fig. 1b).

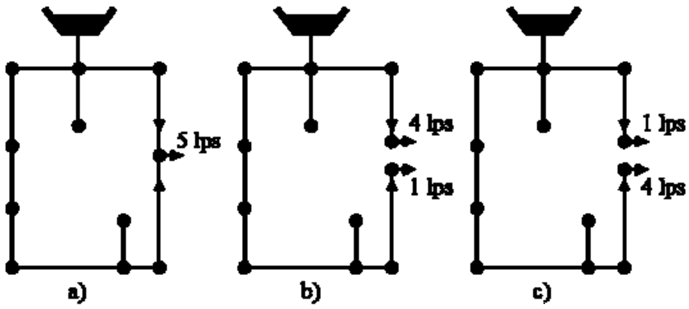

Fig. 1. Alternatives for splitting the loop in one node: a) original loop network, b) original network transformed to a branch network, c) another possibility of dividing the demand in the design stage

The original demand in every split node selected has to be divided between the original node and its "twin node" according to some rule with a rational number of alternatives (e.g., Fig. 1b and 1c). The number of alternatives depends on the demand rate – a high demand in a node requires more alternatives. For networks with more loops, a combination of alternatives for every loop should be considered. Because we are now talking about designing the network, it is necessary to propose diameters for every alternative of the branch network which we get by this procedure. This is accomplished by LP. The cheapest one is an optimum which an algorithm is searching for. The search for the best possibility of splitting the loops is guided by the GA, which proposes one possible branch network for every chromosome in which both the splitting nodes and parameters for splitting the demand in these nodes are coded. The branch network (or this chromosome) is evaluated by accomplishing its design by LP, which is nested in the fitness function of the GA. This represents single iteration of the algorithm. The formulating of the chromosome and establishing the so-called Loop Links matrix (*LL*) and Loop Splitting Options matrix (*LSO*), both of which help to organize searching for the best loop-splitting locations on the network, and finally the R and R_s parameters for splitting the demands in the nodes where the loops are divided. There is the same number of genes as the number of loops in a network. Genes are coded as integer numbers, which indicate a row number in an *LSO* matrix. This matrix defines all the splitting possibilities for every loop. The second half of the genes in a chromosome is also coded as an integer value, and this value defines the ratio by which the demand in the original split node should be divided between the original node and its "twin node" after splitting the particular loop in its location. The proposed algorithm in its basic form is summarized in following diagram on

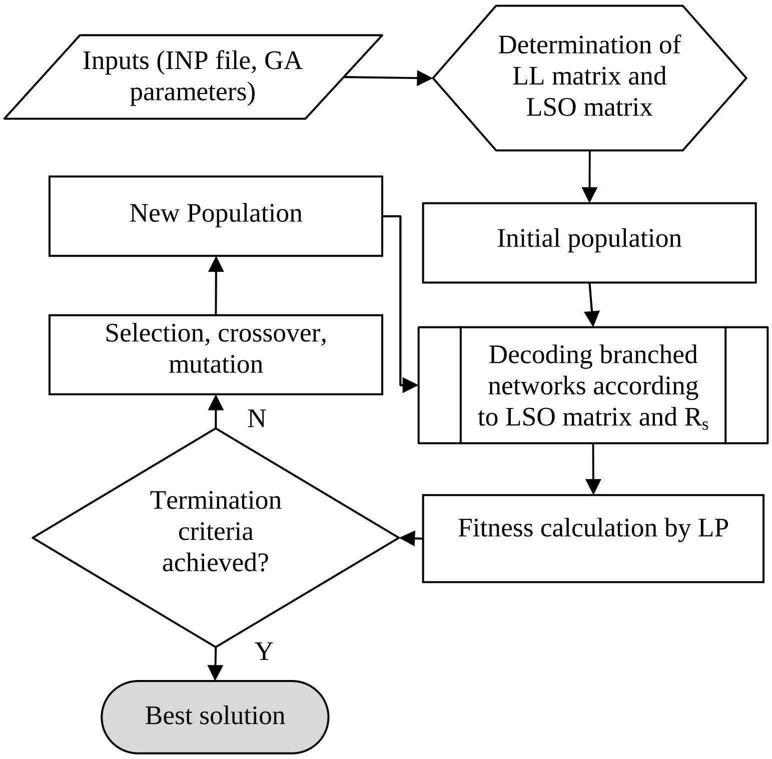

Fig. 2. The scheme of the GALP algorithm

In the last part of this section two remarks on the details of the proposed algorithm follow:

Firstly, one more condition should be added to the LP model in the case of using it in the context of solving looped networks.

The principle of the conservation of energy dictates that the difference in energy between two points must be the same regardless of the path that is taken. Thus, the difference in energy at any two points connected in a network is equal to the energy gains from the pumps and the energy losses in the pipes and fittings that occur in the path between them. This equation can be written for any open path between any two points. Of particular interest are paths around loops because the changes in energy must sum to zero. A linear programming model must also pay attention to this principle, so its formulation, which is expressed by formulas 1- 4 for the branched network only, have to be expanded by the loop condition in the proposed methodology; however each one of them was split into two branches by the described method. There should be the same pressure in the node in which the loop was breaking and in its twin node after proposing the diameters in the network. Therefore, while the LP model is being built, the following conditions should be added, which ensure that the same pressure must be in the original and corresponding dummy twin nodes:

$$Ei = A_{11}\, X_{11} + A_{12}\, X_{12} + ...+ A_{xy}X_{xy} \tag{5}$$

$$E_{ti} = A_{11}\, X_{11} + A_{12}\, X_{12} + ...+ A_{xy}X_{x} \tag{6}$$

$$Ei - Eti = 0 \tag{7}$$

where: E_i, E_{ti} energy losses from the source (pump, reservoir) to the split node on loop *i* and to its twin node,

A_{mn} is hydraulic loss in section *m* and diameter *n*, which belongs to the specified path,

X_{ij} is unknown length of the selected diameter *j* on section *i.*

The second remark on the details of the proposed algorithm relates to its compatibility with EPANET. In an obvious simulation-based GA approach a hydraulic network solver handles the pressure and velocity constraints and simultaneously evaluates the hydraulic performance of each trial solution. The most commonly used simulation model to analyze the network in such a manner is EPANET. In the proposed method simulation by EPANET (or another hydraulic engine) is not incorporated in the core of the objective function (or fitness function), but linear programming is used instead. But for the sake of compatibility with other optimization models, the computation of the friction head losses A_{mn} (Eq. 2) is executed by EPANET by calling its functions through the EPANET Toolkit [18].

3 Results

The performance of this model was evaluated by the optimization of the well-known Hanoi network and triple Hanoi water supply networks. The first problem is taken from the literature and the second problem was introduced by the authors for the sake of evaluating the proposed method for greater problem. The global optimum for the Hanoi network is known because the long usage of this benchmark in the optimization methods development community. The triple Hanoi network is derived from it in such a way that her global optimums could also be evaluated. On this basis it is possible to compare the results obtained in testing runs with the known global optimums. The water distribution trunk network in Hanoi Vietnam was first introduced by Fujiwara and Khang [9]. The parameters of this network are well known and can be found in the work of Fujiwara and Khang [9]. The network consists of three loops, so three **LSO** matrices were determined by the algorithm. Roulette wheel selection was used to choose the parents for the next generation. A one-point crossover was used because of the relatively short chromosome, and the probability of the selected pair of strings being subjected to the crossover operator was taken as $p_c=0.9$. The mutation rate was set to be $p_m = 0.1$.A very simple penalty function was used: if the LP the final producer of every partial solution) does not find a solution (which could happen in some configurations), the algorithm gives this solution a significantly higher cost than the highest cost in the previous generation. That is why the run-time plots and other statistical evaluations of the computation will be described hereinafter only for the triple Hanoi network, which is larger (100 pipes).

In the triple Hanoi water distribution network all the corresponding parameters for the nodes and lines are the same as in the original Hanoi network on all three (single Hanoi network) parts except for four pipes, the head in the reservoir and the demand in one node. These changes were made for the sake of obtaining the same pressure in nodes 3, 33 and 63 (with a diameter of 1016 mm on pipes 1, 2, 35 and 68, which will certainly be proposed here by any optimization method because of the large flow in them) as in the original Hanoi network in node 3. In such conditions the same diameters should be the optimal solution for the corresponding pipes as in the original Hanoi network. These are the changes mentioned: the head in the reservoir is set to 105 m; the length of pipe 1 is 1 m; the length of pipe 2 is 1786.50 m; and the lengths of pipes 35 and 68 are 1641.69 m. In junction 3 the demand is equal to zero. Under these conditions the *reference* (global) optimal solution of the triple Hanoi network could be evaluated as follows:

$$C_{TH} = 3 * C_H - 3 * L_1 * C_1 - 3 * L_2 * C_1 + (1 + 1786.5 + (2*1641.69)) * C_1. \qquad (8)$$

where:
- C_{TH} optimal cost of the triple Hanoi network,
- C_H reference optimal cost of the Hanoi network ($6,057,744),
- L_1 length of the first pipe on the original network (100m),
- L_2 length of the second pipe on the original network (100m),
- C_1 unit price of diameter 1016 mm ($278.28).

For our solution, which is the best solution of the basic Hanoi network compared to the results known to the author, this means that the optimal solution of the triple Hanoi network should be $18,373,697.49. A comparison of the results obtained by the proposed method (which can propose two diameters on some sections) with those published in the literature and obtained by a discrete diameter design (only one diameter is proposed for a section) is a bit problematic, and it is necessary to accomplish it comparatively to the corresponding global minimums for the discrete and split pipe designs. A discrete diameter design has somewhat limited possibilities in comparison with a split pipe design, which can propose two diameters for some sections of the network from the point of view of the resulting cost. Discrete diameter solutions for this reason always cost somewhat more than split pipe design solutions. That is why the *percentual differences* were computed from the reference global minimum - in the case of GALP it is the reference global minimum for a split pipe design, and in the case of a discrete diameter design, another reference global minimum will be taken as the basis.

So we will take as a reference (the best) discrete diameter design solution obtained by [10]. This author used the harmony search method and found a feasible solution at a cost of $6,081,087, and it is the best feasible solution for a discrete diameter design which has been published in the literature. Only solutions which are feasible in terms of the allowable nodal pressures computed by the EPANET network solver are taken into consideration. When this is taken as the reference (optimal) discrete diameter design of the Hanoi network, the reference (global) optimal *discrete* diameter solution for the triple Hanoi network should be \$18,443,867.49.

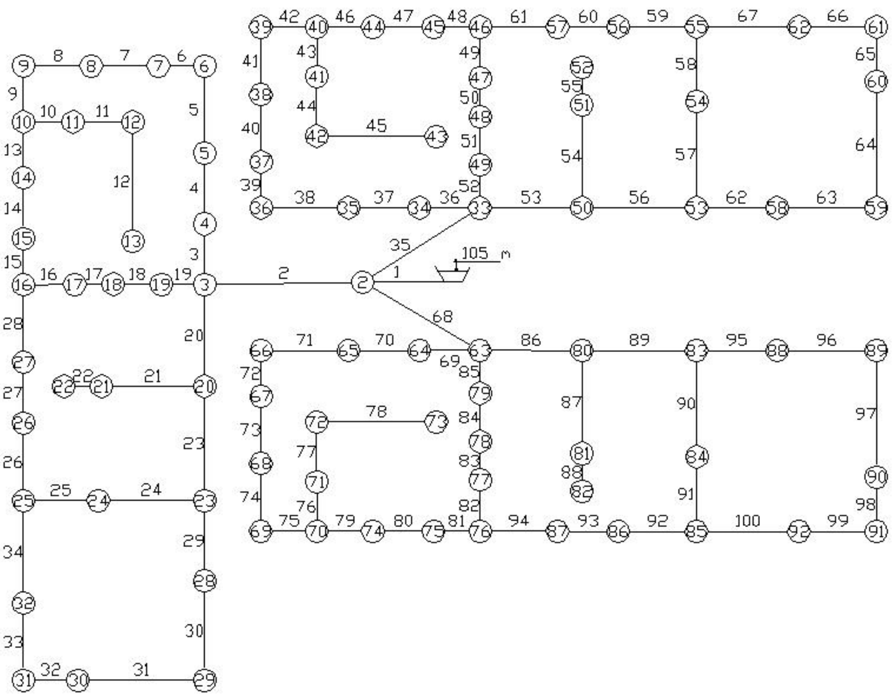

Fig. 3. Triple Hanoi network

The best results obtained with GALP, the GA optimization model OptiDesigner and the HSNet model based on the Harmony Search methodology are summarized in Table 1. The results in Table 1 demonstrate that the proposed GALP methodology gives significantly better results in terms of closeness to the global minimum. This is a consequence of the reduction of search space for GA in a GALP context. There are three loops in the basic Hanoi network. This means that the chromosome for the Hanoi network consists of 6 genes. There are 14 possibilities for splitting the first loop, 8 for the second and 7 for the third. The R_s factor used was set to be 0.1, so there are 9 possibilities for splitting the demand in the split node in every loop (01; 0.2; ...; 0.9). This means that there are 14*8*7*9*9*9 = 571,536 possibilities of which the search space of GALP for the basic Hanoi network consists. The search space of the GA for the same problem is 6^{34} = 2.86512E+26, which is significantly greater. In the case of the triple Hanoi network there are 571.536^3 = 1.86694E+17 possibilities for GALP and 6^{100} = 6.53319E+77 possibilities for GA. In this case it can be seen that a three times greater problem has a smaller search space when using GALP in comparison to when GA is used alone.

GALP is the method proposed by the authors of this paper; references to other methods are GA [2], Harmony search [10] and OptiDesigner – http://www.optiwater.com/optidesigner.html. Other methods were also tested and compared to GALP, but they provide similar or worst results than those in the Table 1

Table 1. Comparison of the best results when applying various methods

Method	Hanoi	triple Hanoi	Deviation from reference global optimum [%]
GALP	6,057,697*	18,394,255	0.04
GA	6,081,087	19,269,160	4.47
OptiDesigner	6,115,055	**	**
Harmony search	6,081,087*	18,839,302	2.67

*reference global solution
** a feasible solution was not found

4 Conclusions

The design of an optimal water distribution network is a complex task. Various deterministic and heuristic algorithms have been proposed and attempted for solving this problem.

Authors have proposed a method in which heuristic algorithms (the genetic algorithm) are incorporated, but the final solution is produced by linear programming. This method is described in the paper and was successfully tested on the benchmark networks. It was determined that the method gives results that are more trustworthy in terms of closeness to a global minimum. This is because of the fact that involving LP in an algorithm reduces the search space for heuristics very dramatically on most network configurations. Although one iteration of the proposed method is a little more computationally intensive than in the case of using only heuristic algorithm, this reduction is finally more important for the effectiveness of the algorithm.

The method is proposed as an alternative to existing methods, mainly when a least-cost design is to be solved. Its extension to solving multi-objective tasks should be evaluated in future research. Nevertheless, the authors supposes it should be tested too, if it is not better to use a more precise method for this single objective task and then refine the output from the least-cost design according to other criteria (or with other demand pattern) by applying a simulation model. The design is always multi objective, but that does not mean that this multi-objectivity must be covered in one computation with too many objectives (and less precision). That is why the author believes that this method can also be practically useful also in the stage described and tested in this paper.

In this work only the basic GA method was used in the stochastic part of the algorithm. It gives good results, but there is a possibility open to replace it with some of the other and more effective heuristic methods which are available in the optimization community. The author expects that this can even refine the method in the future. The effect of such a refinement will mainly be revealed when significantly larger networks than those tested here will be solved.

Acknowledgement

This study was supported by the Scientific Grant Agency of the Ministry of Education of the Slovak Republic and the Slovak Academy of Sciences, Grant No. 1/0496/08 and 1/0585/08 and by the Slovak Research and Development Agency, Grant No. APVV-0443-07.

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A85**

Automated Calibration of Irrigation Projects Simulation Model by Harmony Search Optimization

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Abstract. The paper deals with calibration of the simulation models of hydraulic part of an irrigation project. This model can be used in design, reconstruction, enlargement or maintenance of the pressurized irrigation systems. Calibration of a water distribution model is a process consisting of comparison of pressures and flows predicted with observed pressures and flows for known operating conditions (i.e., pump operation, tank levels, pressure-reducing valve settings), and adjustment of the input data for the model to improve agreement between observed and predicted values. In practice, given a set or sets of measured state variables, engineers apply trial and error techniques with their judgment to vary the parameters and accomplish this task. Trial and error techniques are tedious and have no guarantee of reasonable results. The paper introduces the methodology of determination of calibrated parameters automatically. Described methodology of calibration is based on optimizing procedures using the harmony search approach.

Keywords: Irrigation, simulation model, calibration, harmony search

1 Introduction

Simulation models are used for designing of the new irrigation systems, reconstruction or modernization of existing system or maintenance of different tasks while managing them. Several simulation models have been developed for analyzing hydraulic networks of irrigation and drinking water systems. Among all those models, the three most commonly used for irrigation networks are: EPANET [8], COPAM [6], and GESTAR [1]. In this paper, EPANET was used because it is a reliable model that has been applied worldwide for various types of water distribution networks.

When simulation model is to be applied, there should be assurance that results from the model must bear close resemblance to the actual performance of the hydraulic system. However, various factors always cause difference between reality and results, including errors and presuppositions of simulation model design period, human and tool errors, false simplifying of a design, error in determining border conditions and so on. So, to be sure of correct model function for different operating conditions, model calibration is necessary.

Ormsbee and Lingireddy [7] state that, in general, a network model calibration effort should encompass seven basic steps:

1. Identifying the intended use of the model
2. Determining initial estimates of the model parameters
3. Collecting calibration data
4. Evaluating the model results
5. Performing the macro-level calibration
6. Performing the sensitivity analysis
7. Performing the micro-level calibration

Deviations between results of the model application and the field observations may be caused by several factors, including:

- Erroneous model parameters (pipe roughness values and nodal demand distribution);
- Erroneous network data (pipe diameters, lengths, etc.);
- Incorrect network geometry (pipes connected to the wrong nodes);
- Incorrect pressure zones boundary definitions;
- Errors in boundary conditions (incorrect pressure regulating valve settings, tank water levels, pump curves, etc.);
- Errors in historical operating records (i.e., pumps starting and stopping at the wrong times);
- Measurement equipment errors (i.e., pressure gauges not properly calibrated);
- Measurement error (i.e., reading the wrong values from measurement instruments).

If measured state variable values are significantly different from the modeled values, the cause for the difference probably is in input data. The only way to adequately address such macro level errors is to systematically review the data associated with the model and compare them with the real - field data. This is macro-level calibration.

The calibration process in its micro-level means mainly the determination of pipeline roughness coefficients, after several years of a system operation. It should cover also the specification of any additional data and maximum possible elimination of errors, obstructing the attainment of conformity between the model and reality. There are many parameters that are uncertain and affect values of pressures in junctions or pipes flow rate. Traditional model calibration of a water distribution model is based on a trial-and-error procedure, by which modeler first estimates the values of model parameters, then runs the model to obtain a predicted pressure and flow, and finally compares the simulated values to the observed data. If the predicted data does not compare closely with the observed data, the engineer returns to the model, makes some adjustments to the model parameters, and runs it again to produce a new set of simulation results. This may have to be repeated many times to make sure that the model produces a close-enough prediction of the water distribution network in the real world. The traditional calibration technique is, among other things, quite time consuming. In addition, a typical network representation of a water network may include hundreds or thousands of links and nodes which should modeler deal with. The paper introduces the methodology of determination of calibration parameters automatically. Described methodology is based on optimizing procedures using the harmony search methodology.

2 Methods

EPANET [8] simulation model that analyzes hydraulic in water distribution a system was used in this work. Like other mentioned network models, EPANET abstracts an actual distribution system into a network of links connected together at their end-points called nodes. Links can represent pipes, pumps, or control valves. Nodes can be junctions, reservoirs, or tanks. Junctions are points where pipes join together and where water either enters or leaves the network. Reservoirs represent fixed head boundary locations, such as treatment works or groundwater aquifers. Tanks are storage facilities where the volume and water level can change over time. Junctions can also contain emitters (or sprinklers) which make the outflow rate dependent on the pressure.

EPANET receives input data of the network being modeled through a input file. The input file describes the properties and connectivity of the components in a pipe network. Based on the information contained in an input file, EPANET will compute flows, pressures and other parameters throughout the network. EPANET can be used for many different kinds of applications in distribution systems analysis.

EPANET exists in two basic forms – as standalone application with graphical user interface and as EPANET Toolkit. The EPANET Programmer's Toolkit is a dynamic link library (DLL) of functions that allow software developers to customize or call EPANET's computational engine according to their own needs. The toolkit is useful for developing specialized applications that require running many network analyses, which is the case of simulation model calibration. EPANET Toolkit and hereinafter described harmony search algorithm principles were used for developing software for calibration purposes of hydraulic model of the irrigation network by means of the Visual Basic programming language.

Harmony search algorithm [5] proposed to use in this paper was adopted from musical process of finding “pleasant harmonies” by improvisation. For instance, when several notes from different musical instruments are played simultaneously on a random basis and this process is repeated, there is a possibility to find better harmonies. In harmony search methodology (HS), these better harmonies are saved in a certain size of memory by replacing the worst harmony in the memory until the predefined maximum number of improvisation, generating a new harmony, is reached. The basic scheme of harmony search optimization procedure is shown in figure 1.

Fundamental five steps of a HS could be summarized as follows:

Step 1: Design variable and algorithm parameters initialization

Step 2: Harmony memory initialization

Step 3: Generation of a new harmony

Step 4: Harmony memory update if needed
Step 5: Improvisation stopping criterion check

Step 1: Design variable and algorithm parameters initialization. The optimization is expressed as follows:

$$\text{Minimize } f(x) \tag{1}$$

$$\text{Subject to } x_i \in X_{i,j}\ ;\ i = 1,2,...,N\ ;\ j = 1,2,...,K \tag{2}$$

where *f* (*x*) is an objective function; *x* is the vector of calibrated variables (mainly roughness coefficients of pipes), X_i is the set of the possible values of each variable which is bounded by the pre-defined range; *N* is the total number of searching variables; *K* is the number of possible values for the variables (for instance we design 10 possible roughness coefficients for particular pipe or emitter coefficients for particular irrigator working on field during measurements). Model of the irrigation network with lengths and connections of the pipes and other necessary data is read by harmony search optimization calibration model in the form of the EPANET INP file [8].

Four harmony search algorithm parameters that need to be initialized are harmony memory size (HMS), harmony memory considering rate (HMCR), pitch adjusting rate (PAR), and maximum number of improvisations (NI).

Step 2: Harmony memory initialization. The harmony memory is a memory location (matrix) where solution vectors (sets of roughness coefficients or other calibrated parameters) and corresponding objective function values are stored. The initial HM memory consists of HMS different randomly generated solution vectors. Each solution vector such contains randomly generated pipe roughness values or other calibrated parameters of EPANET model (taken from pre-defined range of the possible values). All the harmonies found are stored in the harmony memory which has a form of (HMS) × (N+1) matrix. Columns one through N store design variable values, and the last column contains the objective function values.

In the case of the simulation model calibration various objective functions can be used. In this work was following equation chosen:

$$min \quad \frac{\sum_{n=1}^{N}\left(1-\frac{Pcalc_n}{Pobs_n}\right)^2}{N_P} + k \times \frac{\sum_{n=1}^{N}\left(1-\frac{Qcalc_n}{Qobs_n}\right)^2}{N_Q} \tag{3}$$

$Pobs_n$	measured pressure i within the system (Mpa)
$Pcalc_n$	pressure calculated with EPANET (Mpa)
$Qobs_n$	measured flow after the pump station (l/s)
$Qcalc_n$	flow calculated by EPANET after the pump station (l/s)
k	weight coefficient stress the importance of the requirement to comply with measured flows after the pump station
N_p, N_q	number of junction nodes with observed pressures and flows

Step 3: Generation of a new harmony. In this step evolution of new solutions is accomplished. Improvisation or generation of a new harmony (generation of new - better combination of roughness coefficients or other calibrated parameters) is performed based on three rules: 1. memory consideration, 2. pitch adjustment, and 3. random selection. This is main part of algorithm – similar as crossing, mutation or selection in the case of genetic algorithms. These general harmony search algorithm principles can be found in GEEM [5].

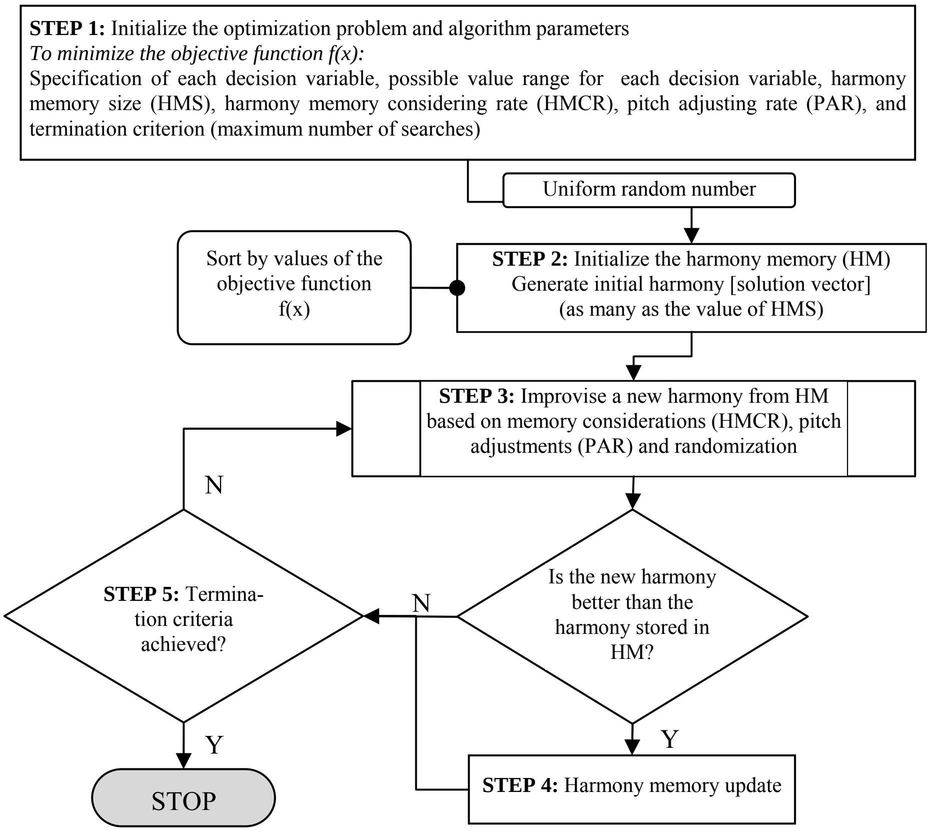

Fig.1. Optimization procedure by Harmony search algorithm

Step 4: Harmony memory update. In this step EPANET model is running with data taken from the new harmony where are coded. By comparing results of computation with observed values (Eq. 3) one gets value of the objective function. If the new objective function value is lesser than the worst objective function value in the harmony memory, the worst harmony vector is replaced by the new harmony vector. Lesser objective function value means, that in the case of new harmony smaller differences are between modeled and observed pressures or flows by using parameters coded in new harmony.

Step 5: Stopping Criterion. A conditional statement is applied to judge whether this harmony search loop needs to repeat or stop. In the case of calibration problem harmony search stops if no improvement in comparing observed and modeled values by objective function (Eq. 3) was found on last five hundred iterations.

3 Results and Discussion

Described procedure of irrigation system calibration was applied in the irrigation system Kuty (South-west Slovakia). Its layout is shown in the Figure 2. This is one of the relatively old irrigation facilities with large area coverage in Slovakia, with applied sprinkler irrigation and an underground pressurized water network. Layout of the irrigation network is branched and it consists from approximately 25 km of pipes (steel, cast iron and some new PVC pipes). Its construction was completed in the middle of the 1960s, and thus the whole facility is coming close to the end of its service life. For this reason it can serve as a suitable model for testing the proposed methods of calibration. Parameters such as roughness coefficients, pump operation characteristics etc., were changed and values of them should be from this reason searched for. The irrigated area amounts to 942 ha. The irrigation system consists of irrigation water take off complex located at the irrigation inlet to the irrigation pump station; pump station itself, pressurized network for the delivery of the irrigation water and from the sprinklers. Presented work is aimed primary on verification of the methodology for evolving reliability of simulation model for this irrigation system. Simulation model could be than used for the rehabilitation design or for finding a best operation rules under various conditions (new crops, new irrigators on system etc.). The process of micro calibration included in this case changing emitter coefficients of irrigators (see EPANET manual), fine-tuning the roughness of pipes and altering pump operating characteristics. Considering the scale of the input and output data the results of the calculation is described only generally, detailed information could be provided by author of this article. During some hydraulic situations insufficient pressures were detected within the system, hampering for instance the irrigation of asparagus and potatoes in the area of branches A3, A4 and in the area of an enlargement of the original system (end parts of branches A3, A4 – Fig. 2). Since the described site is very much suitable for growing the said crops, farmers plan to enlarge the acreage of these crops. Gradually one can expect the further increase in growing these and other crops, with the subsequent enlargement simultaneously irrigated area. This is caused by the fact that modification of the growing crops leads more towards a "monoculture" growing pattern. It could be expected that such events will occur more frequently, because of globalization of the economy, trade and farming, which results in an environment where it becomes more convenient to specialize in a narrower assortment of crops which are cheaper to produce and more effectively to be placed on the market. From irrigators point of view such a situation is more demanding in terms of the pipeline network capacity, which seems to be insufficient already now. Experience of irrigation users as well as measurements made by author of the article in irrigation system Kuty have shown that during the operation of the irrigation system an insufficient pressure in hydrants often occurs.

A good solution of low pressures is for instance the creation of loop in some parts of the pipeline network, e.g. making loop from branches A3 and A4 or from other parts of the system. Such a solution is optimal from costs viewpoint as well as in terms of achieving required technical parameters (pressures). It also simplifies work during the operation and seems to be favorable also in the events of failure (in such cases of malfunction in pipe system it provides an alternative way to transport water to hydrants). Such solution of a reconstruction cannot be accomplished without composing, calibration and application of a simulation model. An example of use of such model in the optimization of a reconstruction design is not aim of present work (author is focused only on calibration of this tool which is necessary for accomplishing this task) and it is described for instance in [3].

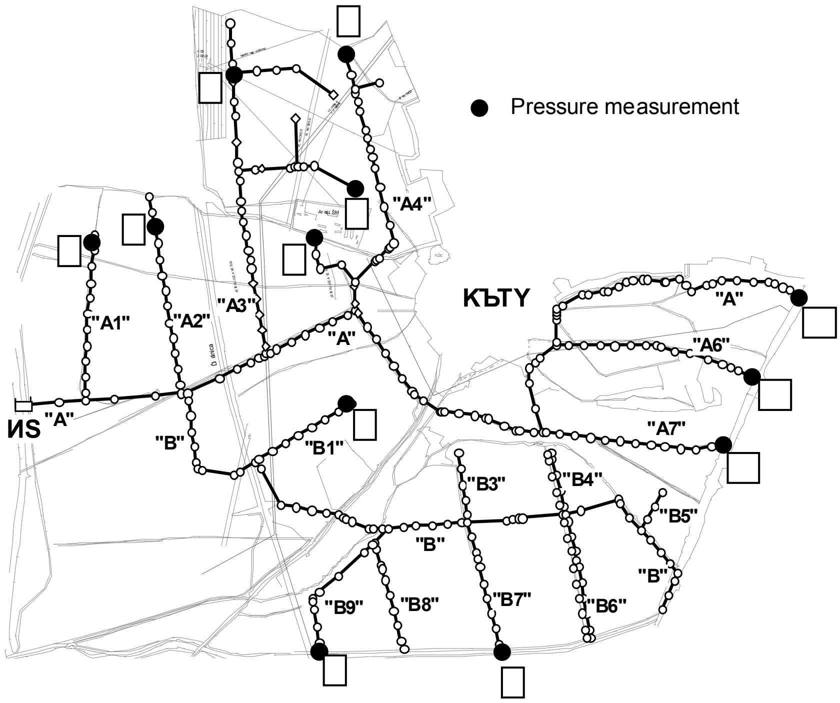

Fig. 2. Layout of an irrigation system in which calibration of simulation model was verified

Hydraulic condition was analyzed using measurements performed both in pump station as well as in irrigation pipeline network. In order to analyze hydraulic behavior of the system for this purpose a more detailed measurements as well as calibrated simulation model was needed. In the first step preliminary hydraulic model was developed. There are 316 junctions and 315 pipes in this model. In preliminary model some simplifications were made – for instance irrigators were modeled with a constant demand, not in pressure dependent regime. Measurements and this tool make possible the identification of various failures, operation malfunctions etc., repair of which is inexpensive (e.g. partially closed network valve was detected on branch A3. This was signalized by significantly big difference between measured and computed pressures on it). Elimination of such problems was the first step and at the same time the first profit generated by calibration effort. If some reparations are not possible, they should be also modeled as any other hydraulics phenomenon in the system. This was completed in so-called macro calibration part of the process which was described hereinbefore.

In addition to that, another problem often occurs during the operation of irrigation system, and it was the incorrect working scheme of irrigators (e.g. the high concentration of sprinklers in some parts of irrigation system), which also results in insufficient pressures. Since such events took place within the described system, an operator should be instructed and given recommendations to reorganize the irrigation system usage. For evolving such rules more detailed calibration (micro calibration) procedure was applied described in chapter about methodology.

Calibration works in irrigation system Kuty included the measurement of pressures in the outlet from a pump station, flow straight after the pump station and measurement of pressures in typical points of an irrigation system (Fig. 2). Placement of irrigators, its type and other parameters (sprinkler nozzle diameter) were also checked. Based on specifications declared by the producer it was important to define the sprinkler characteristic curve. Because this entry depended on the technical condition of the equipment, parameters of sprinkler pressure-flow characteristic curve were also calculated in the context of the calibration computation (as emitter coefficients in EPANET).

During the very first calculation run pipeline roughness coefficients were taken separately for each section and this made possible the identification of different anomalies within the distribution system, caused by errors in data set. In subsequent program runs groups of pipeline sections were created with assumed identical roughness coefficients using the criteria of identical pipeline diameter and material.

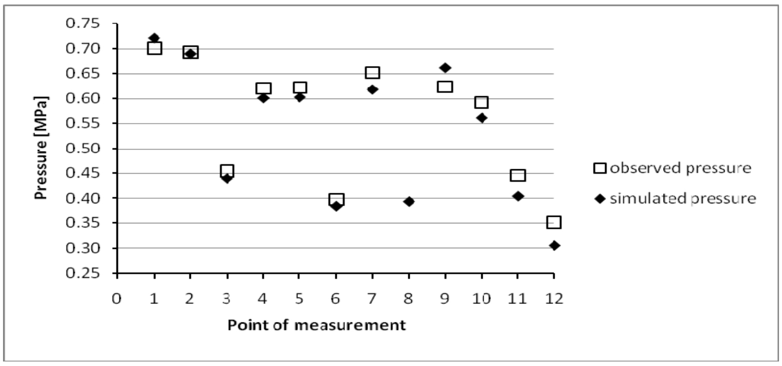

Fig. 3. Calculation results in the system calibrated for the testing operation condition

Calculation results in the system calibrated for the testing operation condition (other set of observed pressures and flows as was used in calibration process) revealed disparities in pressures up to 4,8% and in flows up to 4,9%. Such results, which were obtained, represent a good conformity. A larger disparity between observed and simulated flows was caused e.g. by oscillation of this entry due to the instability of the flow. As it was mentioned above, the accuracy of calibration depends not only on the roughness coefficients set-up but also on other parameters, which can generate various errors. More accurate results can be achieved using additional specifications, such as section length; determined by map readings in scale 1:2000 (no numeric data were available). The more accurate section length could be determined either from longitudinal profiles or by doing direct measurements in the field. The outcome confirms the suitability of the method even in cases with insufficient data available, which is often the case in practice.

4 Conclusions

The paper deals with the use of simulation models in design and maintenance of pressurized irrigation systems. In order to use simulation models e.g. for the design of the reconstruction of these systems one has to perform their calibration. Traditionally, model calibration of hydraulic network models has been a manual task, the modeler making changes to the parameter values on a trial-and-error basis to achieve an effect which he considers will result in an overall convergence between field and model data. The paper introduces the methodology of determination of calibration parameters automatically. Described methodology is based on optimizing procedures using the harmony search methodology. Model calibration is really an optimization problem, although most modelers would perhaps not immediately recognize it as such. The objective is to produce a mathematical model whose predictions agree closely with the field observations. However, there are vast numbers of potential combinations of parameter values, which could be investigated before reaching a good calibration. There is, therefore, scope for achieving a calibration much more efficiently and consistently using an optimization technique (e.g. HS) rather than the traditional trial-and-error adjustment adopted by most network modelers.

Acknowledgement

This study was supported by the Scientific Grant Agency of the Ministry of Education of the Slovak Republic and the Slovak Academy of Sciences, Grants No. 1/0496/08, 1/0585/08 and by the Slovak Research and Development Agency, Grant No. APVV-0443-07.

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A87**

Hydrologic and Hydraulic Aspects of Flood Protection in Urban Region

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Abstract. The goal of the contribution is to present a complex possibility of the modelling the runoff processes in an urban region. The complexity of the approach involved mutual integration of results of partial mathematical models- rainfall- runoff model, sewage system model, 1-D hydrodynamic modelling of open channel flow together with partially covered flows, 2-D hydrodynamic modelling of flooding the town residential area. According to the modelling process appropriate preventive measures for flood protection improvement have been designed.

Keywords: Flood protection, hydrologic analysis, DTM, open channel flow, numerical modelling, polder design

1 Introduction

The Rača region is one of the most important and dynamically developed urban parts of Bratislava City. This region is lying directly under the Small Carpathians – the beginning of Carpathian Mountains in Central Europe. The Rača region has been attacked for many years by storm rainfalls events and during the snowmelt period. The situation became more critical recently after huge urban development in the Rača region. A comprehensive and detailed hydrologic and hydraulic analysis has been worked out to obtain more information for design of appropriate flood protection measures which have to decrease or completely eliminate the flood threatening in the urban region. Formation of the runoff from slopes of Small Carpathians is followed by concentration of discharge in open as well as covered channels connected with the sewage system, too. The consequence of such extreme hydrologic situation is the surface flooding of streets, full covered profiles of channelled rivers under the streets as well as full sewage systems as it was published from Danube water board authority [1].

Presented results have been worked out by two departments of the Faculty of Civil Engineering STU in Bratislava in cooperation with DHI Slovakia, Ltd. responsible for hydraulic modelling works using the most modern simulation tools. Several flood situations have been analysed and modelled using rainfall-runoff modelling connected with already mentioned partial hydrodynamic models.

2 Analysis of Design Intensities of Short-term Rainfalls

For determination of maximum intensities of storm rainfalls which can in the Rača urban region appear, all available data about the rainfalls with duration of 15 and 60 min as well as daily intensities of substitute rainfalls with periodicity p = 0.01 have been elaborated and compared. Several methodological procedures used in Slovakia have been compared with similar procedures for more remote regions. The comparison is illustrated in the Fig. 1.

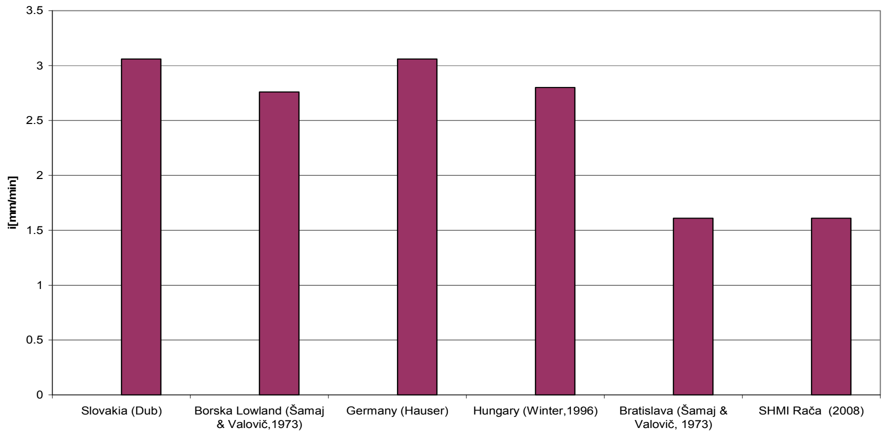

Fig. 1. Comparison of rainfall intensities with duration 15 min and frequency p=0.01 estimated using various methods

Intensities of maximum daily rainfall intensities which are characteristic for regional rainfall events have been determined after several Slovak authors [5], [8]. These values were compared with the results of similar determination for Upper Austria [4] or with regional estimation in Austria (1948-1994) after authors Gutknecht, Watzinger [2]. Values of maximum daily intensities determined after mentioned authors are very similar (Fig. 2). Highest values have been estimated after Schimpf [4]. For daily rainfall amounts intensities with periodicity 0.01 are varying meanly at the 0.054 mm.min^{-1} value. According to mentioned data resulting designed values of rainfall intensities with periodicity p=0.01 have been elaborated for the Rača region [6] and they have been used for subsequent modelling of extreme runoff scenarios in Bratislava – Rača town residential area. They are illustrated in the Fig. 3. There is only one gauging station in the observed region on the main Račiansky creek where from measured data about discharges in 1968–2006 were available for the hydrologic analysis. In frame of the statistical analysis the attention has been put on analysis of mean monthly and annual discharges, on analysis of seasonal occurrence of mean monthly discharges, on trend analysis of mean annual discharges and as well as on analysis of cyclic and seasonal component of mean monthly discharges.

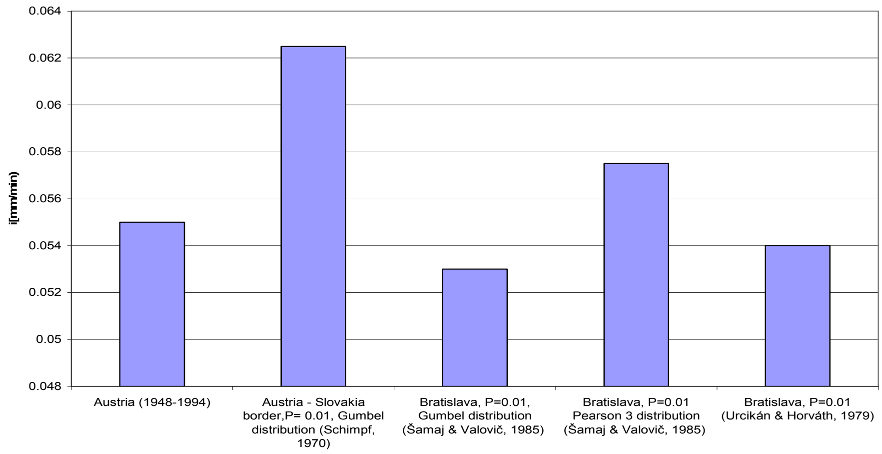

Fig. 2. Comparison daily rainfall intensities with the frequency p= 0.01 estimated using various methods

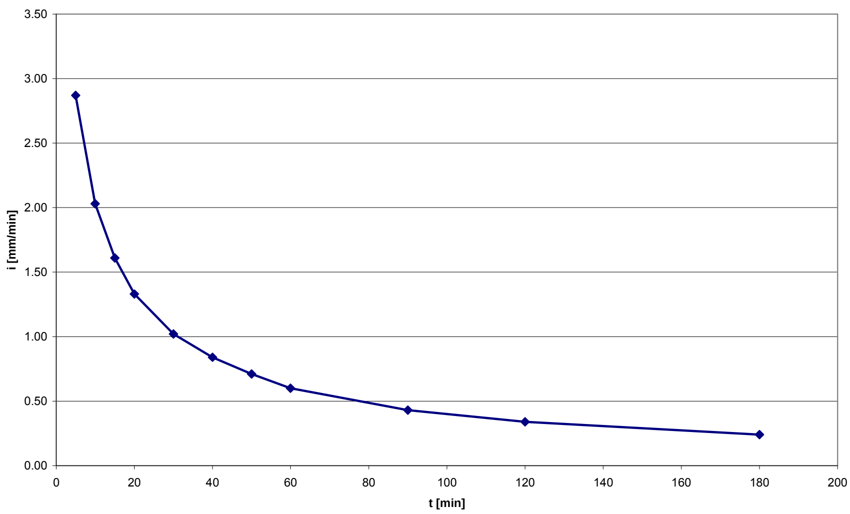

Fig. 3. IDF curve (periodicity p=0.01) for the Rača residential area (SHMI, [6])

3 Hydrologic Modelling of Extreme Runoff Events

For modelling of runoff from the spring snowmelt a hydrologic rainfall-runoff model developed at the STU has been used. The model divides the river basin into several sub-basins using them as hydrologic units (Fig. 4). The model is working with a daily (or hourly) time step consisting of sub-model for water accumulation and snowmelt (snow sub-model), sub-model for soil water content with evaluation of actual evapotranspiration (soil sub-model), sub-model for evaluation of sub-basin response and for runoff transformation in frame of the sub-basin (runoff model) and sub-model for discharge transformation in the river bed. The scheme of the model is illustrated in the Fig.4 [7].

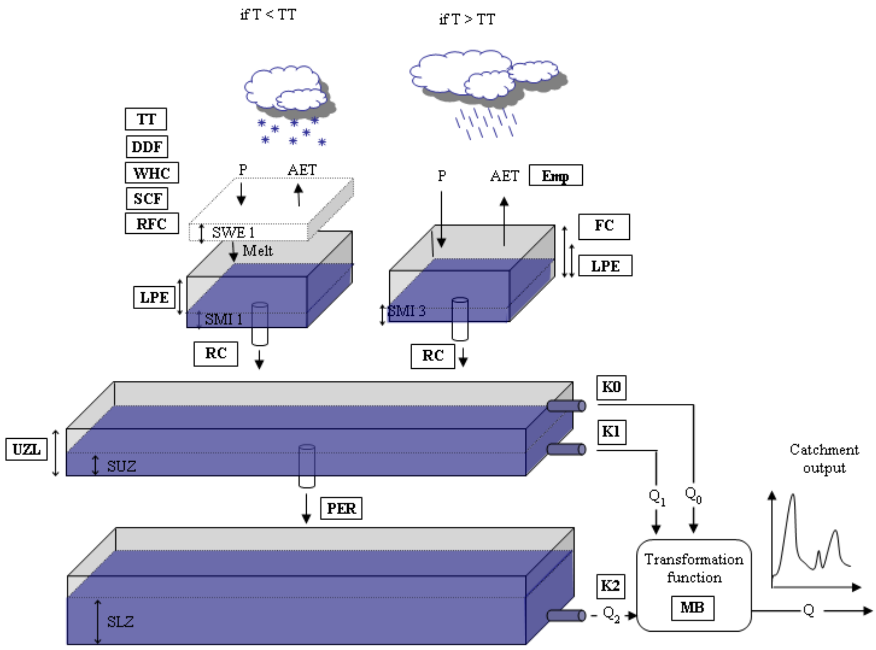

Fig. 4. The scheme of the applied rainfall-runoff model: framed symbols represent model parameters; non-framed symbols represent the model input or output variables

Legend: P - precipitation [mm], AET - actual evapotranspiration [mm], SWE - actual snow water equivalent [mm], SMI - soil moisture [mm], DDF - degree-day factor [mm/ºC/day-1], TT - threshold air temperature [ºC], WHC - water holding capacity [-], RFC - refreezing coefficient [-], SCF - snow correction factor [-], FC - maximum field capacity [mm], LPE - limit of potential evapotranspiration [-], RC - recharge coefficient [-], Emp - empirical parameter for evapotranspiration [-], K0 - recession coefficient for surface runoff (Q_o) [-], K1 - recession coefficient for subsurface runoff (Q_1) [-], K2 - recession coefficient for base flow (Q_2) [-], UZL -

limit for upper zone [mm], PER - percolation parameter [mm/day], MB - parameter of runoff retardation [day].

Analysing a historical series of mean daily discharges on the Vajnory gauging station parameters for calibration of the rainfall-runoff model have been obtained. The attention has been focused on the highest flood after snowmelt period in 2006. In Fig.5 the comparison of measured and simulated mean daily discharges is illustrated on the main Račiansky creek in the profile of the gauging station.

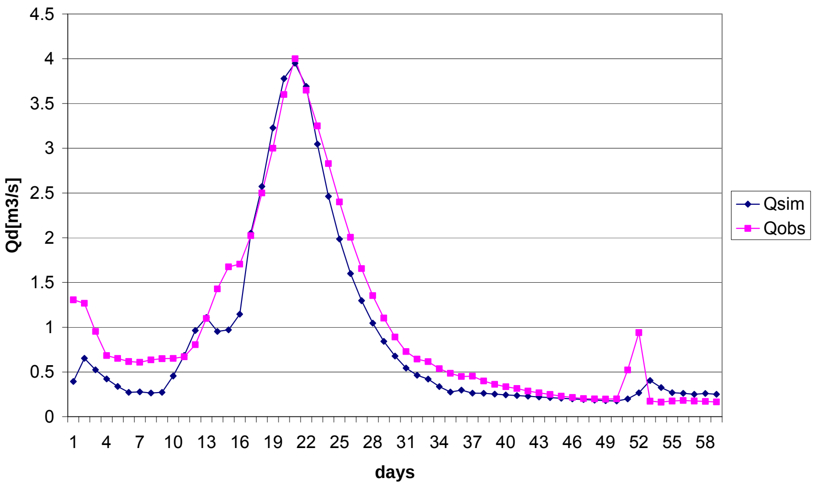

Fig. 5. Comparison of the simulated and observed mean daily discharges at the Vajnory gauging station on Račiansky creek

4 Hydrodynamic Modelling in Rača Residential Part

4.1 Hydraulic Modelling of Covered Reaches of River Beds

The second part of the research was modelling of the most critical point – hydrodynamic modelling of water flow in river beds which are in the residential part covered and mostly under streets. It would not be so strange the problem is that all these reaches are coupled and absolutely capacitive insufficient. The modelling of covered channel flow (using MOUSE model, version 2007) has determined the maximum hydraulic capacity of individual covered parts of creeks in Rača residential part and defined most critical parts where flow rates exceed the maximum capacity of covered parts and the water flows out on the surface [7].

4.2 Hydraulic Modelling of Open Channel Flow

Lower parts of creeks flowing through Rača have been modelled as one-dimensional open channel flow using MIKE 11 model. This model has been used for calculation of water level on lower parts of covered river beds, calculation of water level regime during the flood period as well as calculation of water elevation with respect to flooding of streets in the urban region.

Fig. 6. The scheme of covered parts of river beds in Rača

4.3 Integrated Modelling of Surface Flooding in Rača

For modelling of water flow on the surface in densely built-up parts of Rača region MIKE 21 modelling tool with triangular flexible computational mesh consisting of more than 48 000 triangular elements (Fig. 7). Buildings, greater objects and on some places even house blocks have been excluded from the modelling process completely. For appropriate calculation the elevation system from DTM has been introduced into the model. Lowest parts were approx. 134 m a.s.l., the highest 220 m a.s.l., i.e. about 86 m on length of 1.45 km. For assembly of mathematical modelling it was necessary to create a map describing the roughness conditions of the region (i.e. the resistance magnitude of the terrain against the flowing water).

Two flood situations have been simulated using the MIKE 21 modelling tool. First was a flash flood with culmination Q_{100} (according to SHMI) and the second was a spring flood after during and after snowmelt period. Boundary conditions have been introduced according to hydrological data from Slovak hydro-meteorological institute [6]. There were water levels of five creeks which create the possible flood-threatening of the urban region. The results of flooding simulation by Q_{100} flood wave using MIKE 21 have shown that in lower parts of the Rača region serious problems with flooded streets occurred. The flood map of the Rača urban region is illustrated in Fig. 8. From this map streets could be determined which are under the water, what is the depth and velocity of the flowing water [7].

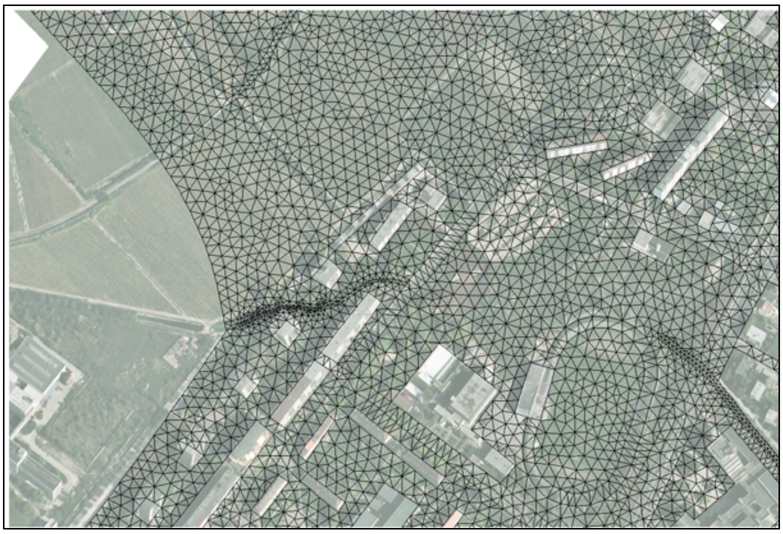

Fig. 7. Detail of the triangular computational mesh

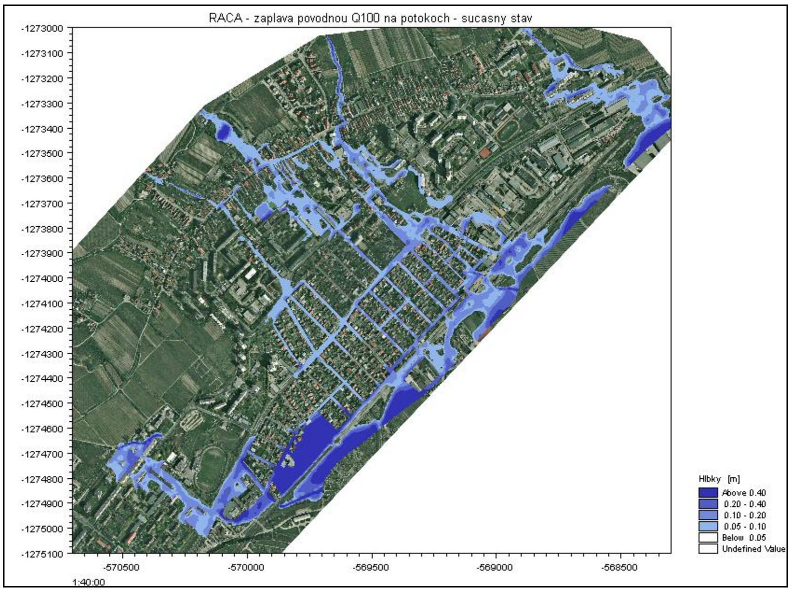

Fig. 8. Flood map of the Rača urban region

5 Measures Introduced for Flood Protection

For mitigation of flood response several measures have been designed. Different alternatives and combinations have been taken into consideration and been tested using above mentioned models. Following measures and their efficiency were tested:

- increase of cross-sections of covered river beds under the streets;
- river lining of open river beds;
- water transfer among the creeks;
- construction of polders in Small Carpathian mountain region above the town to transform flood flow rates.

Most effective, most efficient and economic appeared the last mentioned measure, i.e., the build up of polders in mountain region above the Rača region. Therefore the discovery of appropriate profiles for such polders, their hydraulic and hydrologic analysis was the most important goal of the project [7].

To illustrate that there are such profiles available in the mountain region following figures will prove this fact. For each of the polder main characteristics – area, volume and height have been evaluated as well as the transformation calculations of flood waves according to Q_{100} input data [6]. One of proposed profiles on Bansky creek is shown in a digital form in Fig. 9. The result of tranformation calculation of flood wave Q_{100}=7,0 $m^3.s^{-1}$ is shown in Fig. 10. It has been tranformed to Q =0,76 $m^3.s^{-1}$ discharge value [7].

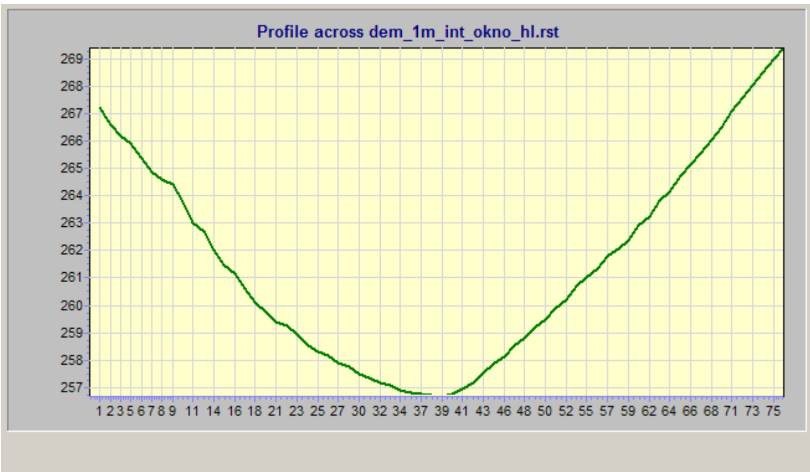

Fig. 9. Cross-section of the polder profile on Bansky creek

Similar to shown data calculations have been performed for all other polders in the mountain region above Rača. They have been analysed as individual polders and not as a cascade. Such a solution would bring very favourable results but unfortunately they do not solve the flood protection problem of Rača region completely.

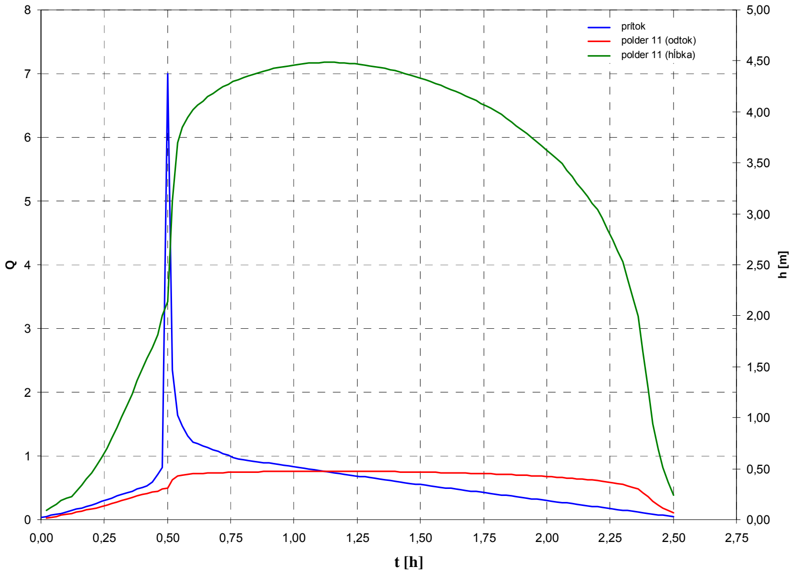

Fig. 10. Transformation of the hundred year discharge on Bansky creek [1], [7].

6 Conclusions

Submitted paper is prepared on base of most recent available hydrologic data according to SHMI database as well as on base of latest digital morphological data (DTM, ortophotomap). Analysing surface water flow in flood situations in Rača most recent modelling methods in hydrology and hydraulics have been utilized. Several analyses as well as prognoses of water level regime in open channel as well as covered channel flow have been elaborated to follow and simulate the flood wave progress in the Rača residential region.

In cooperation with Danube water board authority variants of flood protection measures have been elaborated and evaluated. Very important for building polders in mountain region above Rača is the fact that proposed polders will absorb according to their volumes flash flood waves and they will cause different concentration times of flood waves on particular creeks in the solved region. Calculations of flood wave transformation have shown desirable results, the polder construction will help considerably but they do not solve the flood protection problem completely. The reason for this statement is the fact that there are still creeks where we it is not able to build a polder, at all.

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International Symposium on Water Management and Hydraulic Engineering

Ohrid/Macedonia, 1-5 September 2009

Paper: **A88**

Application of Hydrodynamic Numeric Models on Complex Sediment Transport Processes

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Abstract. This paper discusses the usability of numerical hydrodynamic models for simulation of complex sediment transport processes in rivers. Therefore, a comparison of physical and numerical models has been done. It will be shown; that even small changes have essential influence on the results of local sediment deposition processes. This comparative study is performed by simultaneous tests on numeric and on physical models. Thus, a hydraulic scale model of a stand-alone bed load-adjusting facility was used in order to provide a proper prototype for complex bed load transport processes. The scale model was used for detailed control measurements to evaluate the results of the numerical simulation.

Due to the low water depths, by the numerical model the three dimensional currents could be disregarded. Hence a model based on 2-d depth averaged shallow water equations was chosen. The software used for the numerical analysis is characterized by the calculating performance, stability and easy handling. Thus, a quick execution of the numerical simulation was possible; regardless of the high density computer mesh implemented.The comparative data analysis between physical and numerical models confirms the two dimensional numerical models as a reliable tool for prediction of sediment transport processes. However, locally, the results of sediment transport processes may depend essentially on small scale processes of high turbulence. In such cases, only very detailed numerical analyses with extraordinary long calculation time, or physical models, may afford correct results.

Consequently, it is proposed to use a comparative hydraulic scale model for sediment transport by the complex flow cases in order to obtain reliable results.

Keywords: Sediment transport, bed load retention, numerical and physical model, hybrid modeling, Hydro-GS_2D, hydraulic engineering

1 Introduction

Sediment transport has an essential impact on hydraulic and ecological processes. Thus, prediction of sediment transport processes in natural streams induces nowadays more and more interest: In economical aspects like flood water protection, water resource management and hydraulic engineering on the one hand, in ecological fields like river restoration and ecological integral planning on the other hand. The insight in these processes is the background for alternative sustainable approaches in hydraulic engineering.

The numerical models, widely applied for prediction of water levels and velocities in open channels, in the last few years are more often used to solve problems of bed load transport processes in open channels.

For the longer flow courses and for prediction of general tendencies, usually simple, one dimensional models are used. These models compute one average flow velocity and water level in each surveyed cross section. The potential sediment transport is, inter alia, a function of these parameters. Consequently, neither discrepancy in lateral nor in local limited longitudinal extension attracts interest.

For more detailed investigations, multidimensional computer models are used. In hydraulic engineering, mostly two-dimensional models find application. By dint of these more advanced approach, also more detailed analysis are possible. But these two-dimensional, as well as three-dimensional numerical models are based on simplifications. For two-dimensional models one simplification is clear to find. They do not include the third dimension. But there are several more simplifications in two- as well as in three-dimensional models. So, usually turbulence is integrated through approximations and hydrostatical pressure distribution is assumed. Spatial and temporal discretisation is also always a simplification of the reality.

2 Intention of Investigation

The intention of this paper is a critical assessment of the usability of numerical models for the prediction of bed load transport processes.

Recently, the numerical models, mainly two-dimensional, are often used in the field of hydraulic engineering. This means practically, that it is not so often necessary to arrange and elaborate physical models. In this way, the hydraulic analysis can be conducted much faster and easier. In consideration of these new approaches, a very important question should be explored: How reliable is the output of these numerical models for complex sediment transport processes?

Serious evaluation of model results is without substantial comparative data not possible. Measurements are often either not possible due to environmental conditions (e.g. during flood events), or the studied area is just being designed. In both cases, physical models can support the evaluation of numerical models by additional data. This data cannot replace data acquisition in nature, but it can afford a basic verification of the results obtained by the numerical model.

Therefore, we decided to carry out a comparative study of a numerical and physical model of complex bed load transport processes. So, it was possible to arrange a very detailed examination of the results of the numerical model.

3 Test Accomplishment

3.1 Numerical Model

The scope of the study was to use a very common numerical model, which is often being applied for practical problems in hydraulic engineering. The most important characteristics were the calculating performance, user-friendly-interface and the reliability.

According to these specifications, the software packages of Hydro-GS 2D and SMS 9.2 were used. The selected software is very common and widely used. The Surface Water Modeling System (SMS) offers a clear environment for one-, two, and three-dimensional hydrodynamic modeling and includes a pre- and post-processor for surface water modeling and design. Hydro-GS 2D affords high calculating performance and reliability.

Hydro-GS 2D features two-dimensional hydrodynamic simulation of water bodies. The calculation-procedure integrated in HyroGS-2D is based on the depth averaged shallow water equations.

Coming from the physical equations of conservation of mass, energy and momentum the equations of Navier Stokes can be formed. To solve turbulent currents numerical, a temporal averaging is necessary. This results in the Reynolds equations. When the velocities and momenta in vertical direction can be neglected, the two-dimensional shallow water equations can be used:

$$\partial t\begin{bmatrix} H \\ u\cdot h \\ v\cdot h \end{bmatrix} + \partial x\begin{bmatrix} u\cdot h \\ u^2\cdot h+0{,}5\cdot g\cdot h^2-\nu\cdot h\cdot\dfrac{\partial u}{\partial x} \\ u\cdot v\cdot h-\nu\cdot h\cdot\dfrac{\partial v}{\partial x} \end{bmatrix} + \partial y\begin{bmatrix} v\cdot h \\ u\cdot v\cdot h-\nu\cdot h\cdot\dfrac{\partial u}{\partial y} \\ v^2\cdot h+0{,}5\cdot g\cdot h^2-\nu\cdot h\cdot\dfrac{\partial v}{\partial y} \end{bmatrix} + \begin{bmatrix} 0 \\ g\cdot h(I_{Ex}-I_{Sx}) \\ g\cdot h(I_{Ey}-I_{Sy}) \end{bmatrix} = 0 \tag{1}$$

With the two-dimensional shallow water equations, the horizontal flow is computed in HydroGS-2D [1]. For application of these simplified equations some preconditions must be fulfilled:

- Shallow water is dominant. Wavelength is much longer than the amplitude.
- Velocity and momentum in vertical direction can be neglected.
- Due to the low stream line deflection, the pressure can be supposed as hydrostatical.

Thus the velocities in horizontal x and y direction are provided. Velocities and momenta in vertical z direction are not regarded. Turbulence modeling is done by the Boussinesq approximation, which is based on the concept of eddy viscosity [1].

$$\nu = \nu_0 + c_\mu \cdot u^* \cdot h \tag{2}$$

In this connection ν_0 is a constant value, which can be defined for each element. The second part of the equation shows the eddy viscosity, due to the bed shear stress. The tractive force is u*, water depth is h and the constant c_μ has been established in tests as to lie between 0.3 and 0.9. So it was implemented into the software with 0.6 [2].

The simulation of bed load transport by the HYDRO_GS-2D is founded on the extended Meyer-Peter & Müller equation. By balancing all entering and leaving sediment loads of each control element, according to DVWK [3], changes on the riverbed are detected.

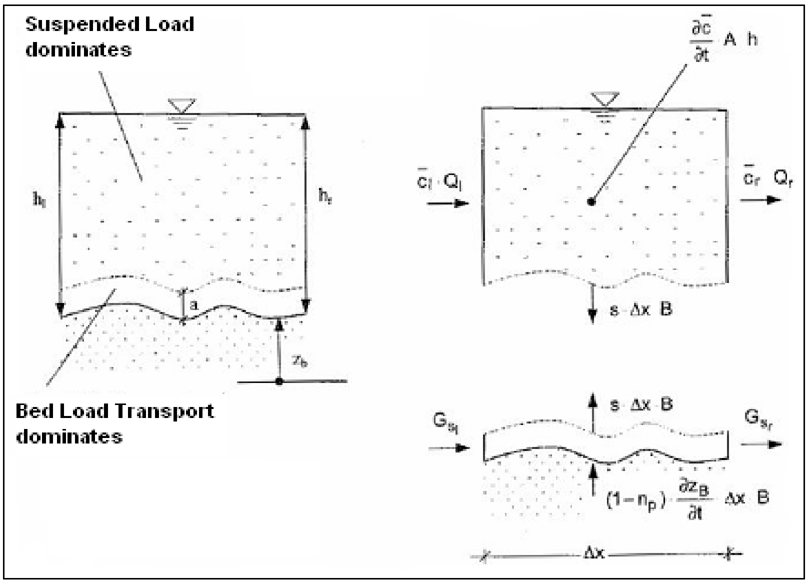

Fig. 1. Vertical classification of water body by bed load and suspended load in one-dimensional formulation [3]

3.2 Physical Scale Model

For experimental validation of the numerical results, a physical scale model was used. The primal aim of the model was the design and optimization of a bed load retention structure. The horizontal scale was defined as 1:80 and the vertical scale as 1:26.67 triple inflated. During the tests, discharge of one- until the hundred-yearly floodwater has been tested. The surface of the model has been made out of concrete and only for illustrational purposes, the floodplains were colored. Dark green was used for wood and light green for meadow and fields. To simulate the natural roughness in woody areas, stones with diameters between three and five centimeters were assembled. The physical scale model of the river has had a length of approximately 18.75 m and was build with a “quasi-movable” riverbed. This means that the granulometry of the bottom sediments satisfied the roughness condition for the necessary water levels similarity (bed forms, armoring layer). They were also mobile in order to indicate erosion or deposition tendencies by high discharges, but they did not represent a part of the bed load. The river bed has been modeled by coarser sediment (grain size 17 millimeter) than the material which has been used to indicate bed load transport process. For the simulation of bed load transport processes basalt sand with a grain size of 2 millimeters was admitted to the inflowing water. The quantity of added sediment depended on the water discharge.

For the comparison of the results with the ones coming from the numerical model, during the test runs, photos were taken and after each test the surface of channel bed has been scanned by laser. Additionally, water levels and velocities were measured and compared with the results from the numerical model.

3.3 Implementation

The numerical model was a replication of the physical model and not of the nature. This means that the dimensions in the numerical model accord with the physical scale model. Due to this fact, it was possible to compare the numerical with the physical model without any uncertainties on the scale.

When this investigation was done, the used numerical model allowed only one grain size for sediment transport simulation. However, for the physical model two different materials were used. Coarser material was used to shape the river bed. To indicate the bed load transport processes dark material with a smaller grain size was admitted to the water during the test run. In this way, the admitted material was much easier transported than the material of the quasi moveable riverbed. This involves different degradation behavior of the materials. This challenge was solved by using a parameter, which limited the maximum degradation per time unit. So it was possible to achieve realistic degradations of the quasi-moveable bed and aggradations of the admitted bed load material.

The calibration and validation of the model were done at different discharges by comparing water levels, velocities and bed load transport processes. The congruence of the numerical simulation with the physical model during calibration was convincing.

4 Test Results

Basing on the plan view and cross sectional areas of the deposition, the output of the numerical models was compared to the physical scale model. In this manner it was possible to evaluate the horizontal expansion as well as the vertical layer size of deposited material. It was essential to validate the numerical outcomes on all cubic expansions. Only in this way it was possible to identify spatial differences between the models.

Following, the flow velocities and water levels were measured on the physical model. These parameters were used to calibrate the numerical model. The accordance of the numerical and the physical models was corresponding for the measured hydrodynamic data. As a consequence they were not further discussed by the evaluation of the numerical model.

In this paper, only the test run with the annual flood water discharge is exemplarily shown. More detailed test results were presented by Steiner [6].

4.1 Annual Flood Water Discharge

A discharge of 0.245 m³/s represents the yearly flood water discharge in the scale model. At this discharge the water level in the channel is at the maximum. Barley inundation of the floodplain occurs. The test run is illustrated by the following figures

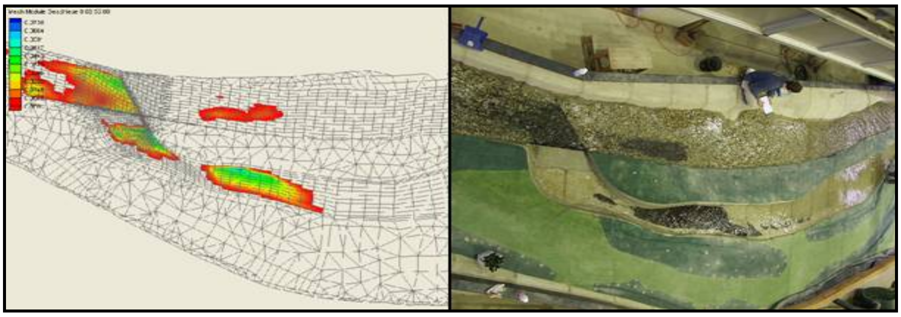

Fig. 2. Time: 75 minutes; Total sediment load added: 45 kg

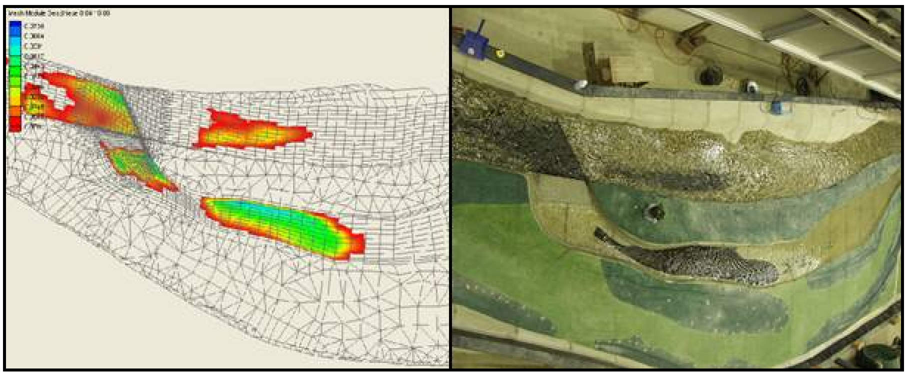

Fig. 3. Time:150 minutes; Total sediment load added: 90 kg

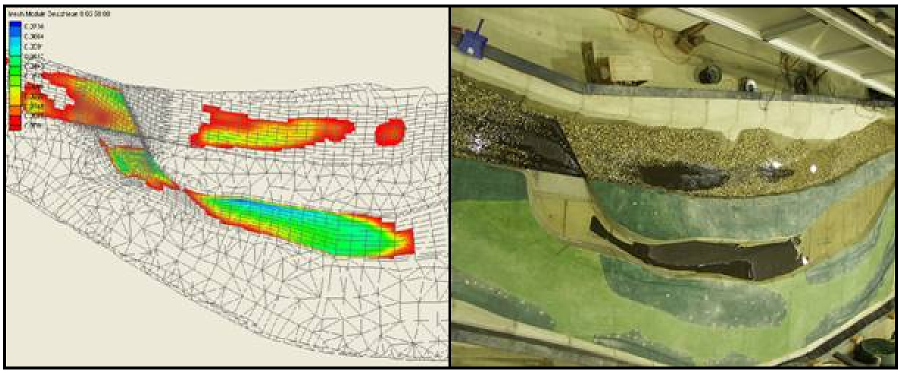

Fig. 4. Time: 250 minutes; Total sediment load added:150 kg

At the series of the illustrations, a good correlation of numerical and physical model can be seen. Both, the spatial and the temporal sedimentation distribution of the numerical model comply with the physical model. Only at the intake area of the side arm, the aggradations of sediments are overestimated by the numerical model. This can be seen at the following Profiles.

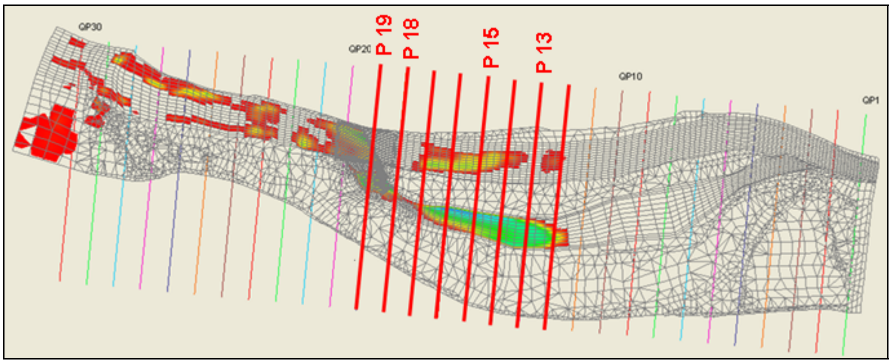

Fig. 5. Map of the area including the location of the profiles.

The upper illustration shows the location of the profiles. Due to the fact that most areas of the numerical simulation comply very well with the physical model, only diverging sections are presented. At the other profile, very small or nearly no discrepancies can bee seen. Following, four representative profiles are shown below. The profiles include the derived output of the numerical model "Hydro_GS-2D", as well as the depositions on the physical scale model. The profiles were scanned by a laser, after finishing the test run.

At the sidearm intake area, the aggradations are locally overestimated by the numerical computation. In Profile 19, accumulations with an averaged layer thickness of 2 cm can be seen albeit no depositions in the physical model occur. In the downstream direction this actual situation is changing. At Profile 18, the aggradations of the numerical and the physical model are nearly equated. Continuing downstream, at the Profile 15 and Profile 13, the sedimentary deposition is persistently overestimated by the numerical model. In order to understand this process, it is necessary to regard the divergences in Profile 19. In the numerical model, the additional accumulation in the intake area has a decreasing impact on the bed load extraction into the side arm and changes the flow field in this area. This results with less sediment transport and consequently less deposition in the side arm. The difference in the geometry in Profile 18 is an indication of the changed flow field.

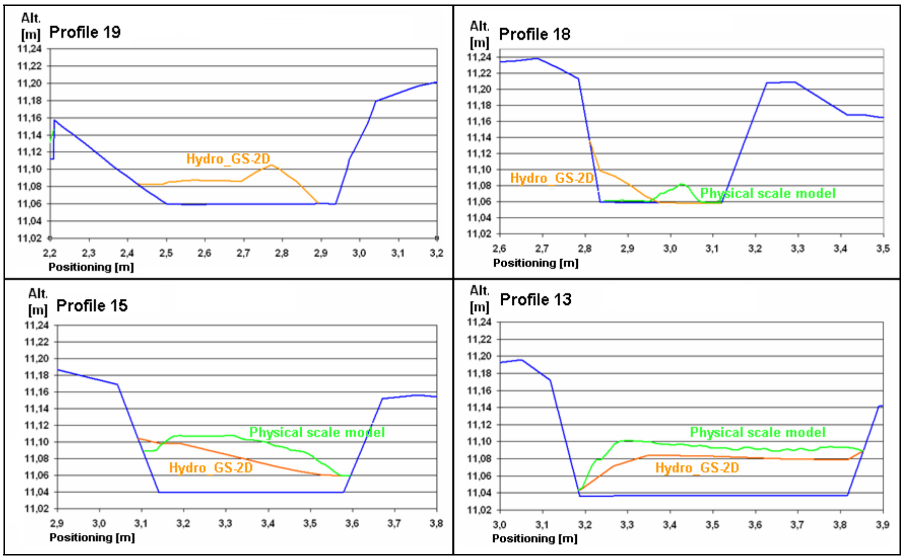

Fig. 6. Profiles of the sidearm: Comparative illustrations of aggradations in the physical and the numerical model

The accuracy of the numerical model was very good, only in the intake area of the sidearm, several differences were observed. Thus the question on cause and effect arises. In the physical scale model, pronounced turbulences in this region could be observed. However the water depth was low. The procedure integrated in the numerical model, Hydro_GS-2D, is based on the numerical solution of the 2D current equations with Finite-volume-Discretization. Yet, due to the low water depth, no dominant three dimensional currents could be observed at the physical model. Therefore, the application of a two-dimensional model has been feasible. Turbulence modeling in the numerical simulation is done by the Boussinesq approximation, which is based on the concept of eddy viscosity. Thereby the reproduction of the water levels in the numerical model functions in a satisfying manner. However, due to the high intense of spatial and temporal variation of the flow conditions, the real appearing velocities could not be reproduced by the numerical model. Even by refining the computation mesh, no significant correction could be established. Thus it is supposed, that a much higher temporal and spatial resolution would be necessary to simulate these activities. However, in the physical model, the essential influence of the turbulences on the sediment transport could be observed. From this follows, that in regions of high turbulences, only very extensive computations can predict sediment transport processes reliably.

5 Conclusions

This investigation is an obvious example of the influence of small hydrodynamic discrepancies on complex bed load transport processes in numerical and physical models. The prediction of water levels can be done by numerical models very reliably. Further on local bed load transport processes are predicted very exactly. Only in the regions of high turbulences, transport processes cannot be detected adequately by common numerical models. In these exceptional regions, intricate turbulent flow processes dominate, which have an essential impact on the sediment transport.

For open channels, containing complex hydraulic structures, neither three-dimensional (3D) hydrostatic models are able to predict the 3D flow and transport phenomena accurately. In recent years, several 3D-models, that incorporate sediment transport and, in some cases, have the capability to predict the bed topography evolution or the equilibrium bathymetry have been proposed [7]. Yet, every numerical model uses simplifications. Therefore there is always a residual risk of discrepancies between the computation and nature. If only rare reference data is available, shortcomings of the computation cannot be identified.

This investigation shows clearly the limitations of the numerical simulation. In single task, it is possible to use high sophisticated three-dimensional models which can afford satisfactory results. However, it is difficult to verify the output of the computation. Consequently, it is usually not possible to make sure, whether the used numerical model is also the adequate one.

Thus, it is suggested to use a combination of a numerical and a physical model for sophisticated hydraulic tasks. On the one hand, it is possible to verify the results of the numerical model and to survey additional data on the physical model on the other hand, is it much easier and more demonstrative to test specific prototypes on a physical model.

Hybrid modeling provides trustworthy results and makes use of the benefits of two the worlds: physical and numerical modeling.

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A89**

Hydraulic Flushing of Alpine Reservoirs – Model Study

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Abstract. Dams represent obstacles in rivers which modify natural flow conditions, as well as their sediment transport capacity. As a consequence, the reservoir water storage volume is continuously reduced. Technical, as well as ecological aspects of these impoundments are severely disturbed by accumulated sediments. One of the instruments for sediment management in reservoirs is sediment flushing. Extremely high concentrations of released sediment and the lack of predictive methods for a better flushing operation, result with adverse impacts on the river morphology and the aquatic ecosystem. A hydraulic model study has been conducted at the Institute for Water Management, Hydrology and Hydraulic Engineering (IWHW), BOKU, Vienna, to identify essential relations among flushing procedures and sediment release.

Keywords: Hydraulic flushing, sediment management, scale model

1 Introduction

With the aim to quantify the sediment outflow during the flushing process, in this model study, some hydro-morphological correlations and a simple one-dimensional sediment transport model have been verified. This model study was conducted with the aim to answer to the question - which parameters are significant to describe the sediment flushing process by the reservoirs in Alpine regions. The typical shape of the sediment accumulation in a reservoir is the delta formation. As the transported sediments approach closer to the dam, the shear stress at the bottom diminishes and so by deposition of the sediment a formation of delta begins. MORRIS and FAN [1] zoned the sediment deposits of a reservoir into three main regions (Fig. 1). The geometry of the delta can be divided into the top-set bed – consisting of gravel and large grain sized sediments, the fore-set bed – building the front of the delta, and finally - the bottom-set bed, which is made of the fine, far transported sediment.

Dependent upon the geometry of the reservoir, hydrological aspects, properties of the incoming sediment, reservoir operations and etc., the resulting shape of the accumulation zones can vary in regard to the specific reservoir conditions. After all, deltaic depositions will migrate towards the dam. The depositional pattern tends to become wedge-shaped. Consequently, in long term, the wedge-shaped deposition could be the equilibrium state for any reservoir [2].

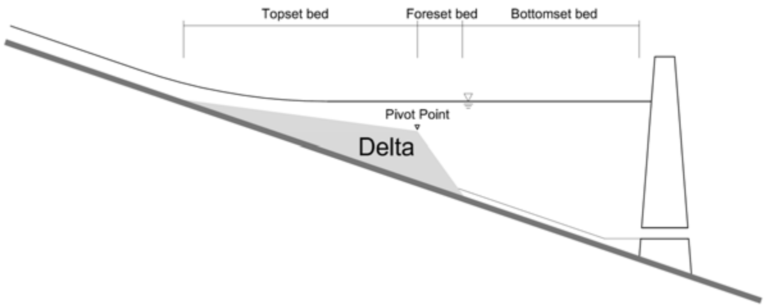

Fig. 1. Schema of a deltaic deposition zone [1]

Aside from the depositional impacts in the reservoir, the upstream and downstream stretches of the river system are influenced by the storing of sediment, due to the dam construction:

- Accumulation of sediments in the upstream reach results with rising of the bottom level;
- Lack of sediment in the downstream part causes lowering of the river bed downstream of the dam site.

The impacts on the reservoir operations are caused by the loss of water storage volume and the negative effects on the dam construction as well as the dam function:

- Storage loss has severe impacts on the usable water capacity and the flood retention;
- Intakes and Outlets can be blocked by deposited materials;
- Abrasion of the turbine blades by sediment particles.

An integrated approach to the sediment management, that includes all the feasible strategies, is required in order to balance the sediment budget across the reservoirs [3]. This implies that the dam and the impoundment are operated in a manner, consistent with the preservation of sustainable long-term benefits, rather than the present strategy of developing and operating a reservoir, as a non-sustainable source of water supply [3].

2 State of the Art – Hydraulic Flushing

Options to manage sediment can be arranged in three common ways [4]:

- Minimizing the sediment loads entering into the reservoirs;
- Minimizing the deposition of the sediment in reservoirs;
- Removing the accumulated sediments from reservoirs.

In the most cases, a combination of these methods has to be performed, to achieve the sediment budget. Hydraulic flushing is used for scouring of deposited sediments out from the reservoirs by the use of low level outlets in a dam, by lowering, or without lowering the water levels and thus increasing the water flow velocities through the reservoir. Flushing can also be classified as flushing under pressure and free-flow flushing [5].

Under the pressure flushing, water is released through the bottom outlets, while the water level in the reservoir is kept high. Free flow flushing means that the reservoir has been emptied and the inflowing water from upstream is routed through the reservoir, resembling to the natural flow conditions [5].

The effectiveness and the process of flushing depend on the depositional pattern of the sediment. [2] shows the differences between the deltaic and wedge-shaped accumulation and describes the flushing process in the following way (Fig. 2). The numbers on the lines correspond to the numbered water levels of the longitudinal section.

The flushing process under the wedge-shaped depositions is shown in Fig. 2. In contrast to the delta shape in Fig. 2, the deposition zone has reached the dam site. If the bottom outlet gate is opened, the outflow erodes a stable flushing cone in front of the outlet. By lowering the water level from 0 to 1, the shape of the erosion in the region of the outlet does not change. This process is called the pressure flushing. When the water surface level is about the same level as the bed surface near the gate (level 1 to level 2), the flowing water starts to erode the rim of the flushing cone and retrogressive erosion will occur to create a flushing channel towards the upstream direction [2].

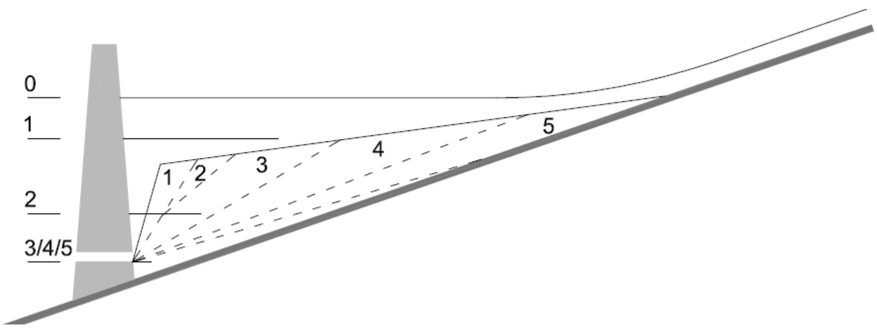

Fig. 2. Retrograde flushing channel erosion under conditions of wedge-shaped deposition [2]

2.1 Channel Formation

In order to work with one dimensional sediment transport models, it is necessary to estimate the developing alterable width of the flushing channel. A simple method to calculate the equilibrium channel width is by using the following relation:

$$B_e = K \cdot Q^{0,5} \tag{1}$$

where: B_e is the computed channel width, Q is the main discharge and K is a factor to be estimated. By this simple approach, the stable or equilibrium width can be estimated, if the factor K is known. For analysis of the complete flushing process, the development of the equilibrium channel characteristics has to be simulated. Therefore, the concept of [2] has been applied:

In the dimensionless width parameter, the time factor is supposed to be taken into account. It may be expressed as an exponential function defined by

$$\frac{(B - B_0)}{(B_e - B_0)} = 1 - \exp[-\xi \cdot (t - t_0)] \tag{2}$$

where: B_0 is the initial channel width; t is the time factor in minute, and t_0 is the time when the flow condition turns into free-surface flow condition; ξ is a coefficient to be estimated [2]. With that procedure, the width development of the flushing channel can be observed by hydraulic model tests. This data can be used to calculate the amount of flushed sediment by simple sediment transport formula by the TUM (Tsinghua University Method).

2.2 Sediment Transport

- TUM – Tsinghua University Method

$$Q_S = E \cdot \frac{Q^{1,6} \cdot I^{1,2}}{B^{0,6}} \tag{3}$$

where:
- Q_S sediment outflow in mass per time
- Q dominant discharge
- I bed slope (account: bed slope = water surface gradient)
- B channel width
- E sediment specific factor

This widely spread function is based on the sediment transport formula by WUHAN, which was especially adapted to calculate the transport of fine and sandy sediments. On the Tsinghua University in China, the function of WUHAN was taken and combined with the MANNING-STRICKLER formula, to obtain a simple and user friendly connection. Because of the adaptation to the fine sediments, it has not yet been determined whether this formula could be also used for the transport estimation of the coarser particles.

2.3 Flushing Efficiency

The efficiency of the flushing process can be expressed in different ways. In this work, the effectiveness of the flushing tests was defined by:

$$F_e = \frac{V_{Se} - V_{Si}}{V_W} \tag{4}$$

where: F_e flushing efficiency
V_{Se} volume of flushed sediment
V_{Si} volume of the inflowing sediment to the reservoir (in this test = 0)
V_w required amount of water volume

The efficiency was calculated as an average value of a defined period.

3 Hydraulic Model

The aim of this hydraulic scale model is to calibrate and verify the expressed functions by the data achieved from the model tests. In this manner, conclusions of the conformity between the observed results and the one dimensional transport model can be done. Fig. 3 illustrates the setup of the model.

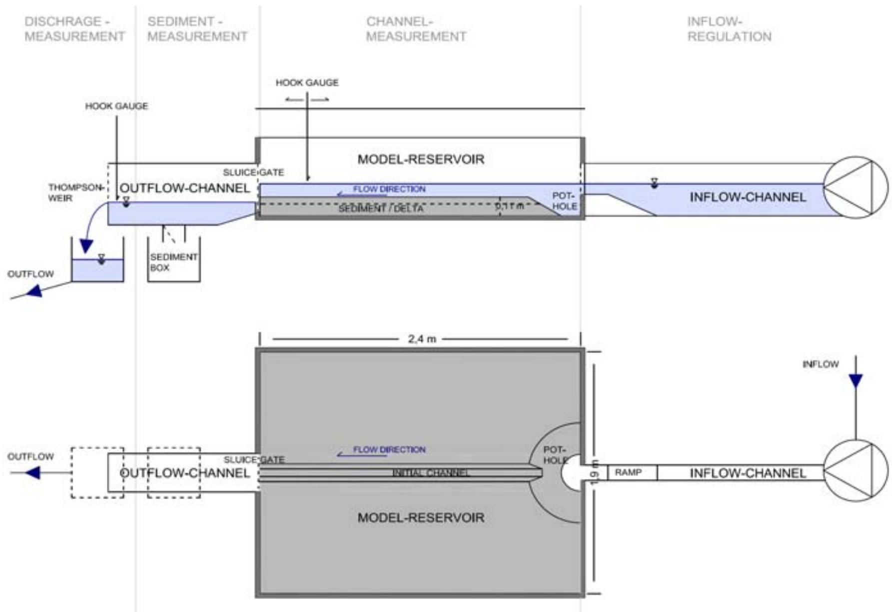

Fig. 3. Schema of the model setup

In order to simulate Alpine conditions on the model, coarse sediment material was chosen. Although in Alpine reservoirs both - fine and coarse sediment appear, the effects of sorting and layering have been ignored. This simplification was done in order to avoid the ambiguity of results, related to the high number of different parameters. The chosen grain size of the basalt sediment particles was about 3.0 mm by d_{90}.

Eight model runs under different conditions were conducted (Tab. 1). The parameter that has been varied was the discharge, in order to observe the correlation between the outflow and the rate of sediment transport, as well as the effectiveness of the flushing operation. In addition to the five measured test runs, three further studies were performed in order to create a photo-series on the flushing event. In this way, qualitative observation of two runs: the one with and the other one without a prepared initial channel in the deposition sediment zone could be realized.

Table 1. List of the model runs

RUN	DISCHARGE	SITUATION	ANALYSES
1	1,74 l/s	Initial channel	Measurement
2	1,99 l/s	Initial channel	Measurement
3	2,27 l/s	Initial channel	Measurement
4	2,44 l/s	Initial channel	Measurement
5	2,54 l/s	Initial channel	Measurement
F1	1,74 l/s	Initial channel	Photo-series
F4	2,44 l/s	Initial channel	Photo-series
F4-I	2,44 l/s	no Initial channel	Photo-series

- Discharge
- Sediment transport
- Water surface slope
- Channel width

All these measurements were done intermittently. The values of the flushing sediment concentration were calculated from the volume of the accumulated sediment divided by the measurement interval.

4 Results of the Model Tests

Due to the similar flushing characteristics of each model run - the results of the model study are illustrated in the diagram describing the run 4.

Measured values (Run 4):

- Temporal sequence of: Discharge/Sediment transport/Flushing efficiency by using Eq. (4).
- Temporal sequence of: Sediment transport / Accumulated sediment transport
- Temporal sequence of: Flushing channel width development

The Fig. 4 illustrates the test results of the Run 4.

Calculated values (Run 4):

- Equilibrium width of the flushing channel (calculated by data of Run 1 to Run 5) by using formula (1).
- Width development of the flushing channel (calculated by data of Run 1 to Run 5)
- Sediment transport by using the TUM formula (Run 4)

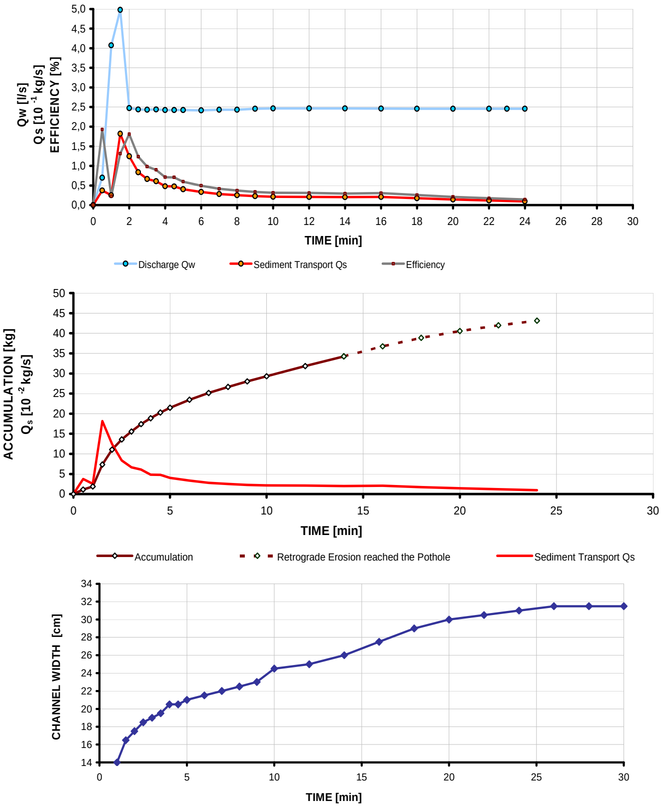

Fig. 4. Temporal behavior of the observed discharge, the sediment transport and the flushing efficiency-formula (4), the accumulated flushed sediments and of the width development

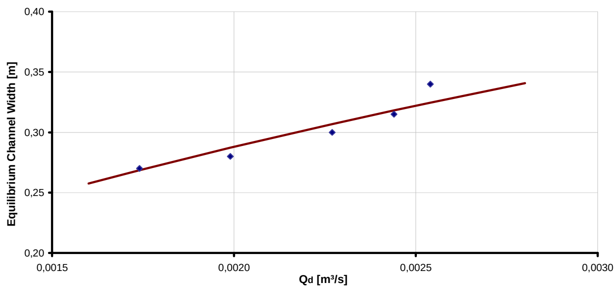

Fig. 5. Relation between the dominant discharge Q_d and the equilibrium channel width by evaluation of Run 1 to Run 5. With this information the formula (1) can be calibrated.

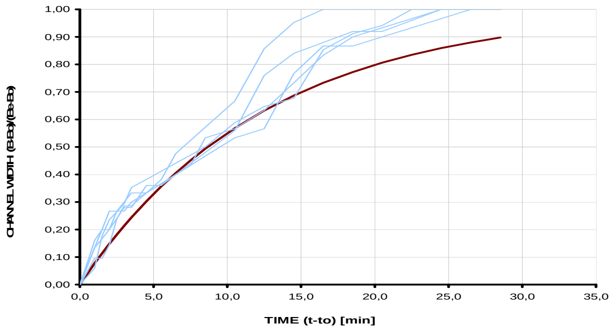

Fig. 6. Comparison of the measured data with the calculated data on the width-development (Run 1 to Run 5)

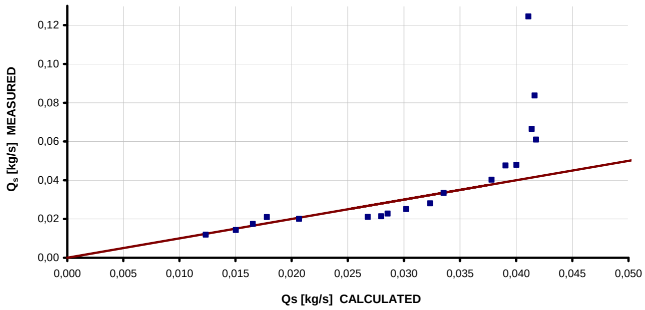

Fig. 7. Comparison of the measured data with calculated data by the TUM – the diagonal line represents exact fitting of the values. Deviant data in the right part of the diagram derive from the beginning of the flushing process – to compare with the photo-series.

Primarily, the developing flushing process is shown by the results of the photo-series (F4) under the conditions of the discharge of Run 4.

As seen in the photo-series, the flushing operation consists of several phases. This circumstance is also documented in the measured data of the model test, as well as in the prepared diagrams. By comparison of the Fig. 4 with the results of MORRIS and FAN [1], a significant affinity between the sediment concentration shown in the literature and the efficiency curve of this model study can be noticed. The first, and relatively high peak of the flushing efficiency, depends on the low water outflow by opening of the sluice gate for the first 1,5 cm. A stable cone of erosion has been shaped, initiating a high flushing efficiency, because of the low discharge at the start of the model test – compare with photo-series F4; time: 00:30 – 01:30 (min). By further opening of the sluicing gate and reaching the free-flow condition, a high rise in sediment transport has been effected, at which the drain off from the lateral floodplains widened the erosion zone dramatically – compare with photo-series F4; time: 02:30 (min). To that time, the flushing process can not be described by a one-dimensional transport formula, which is shown by the underestimating of the sediment transport in the diagrams Fig. 7.

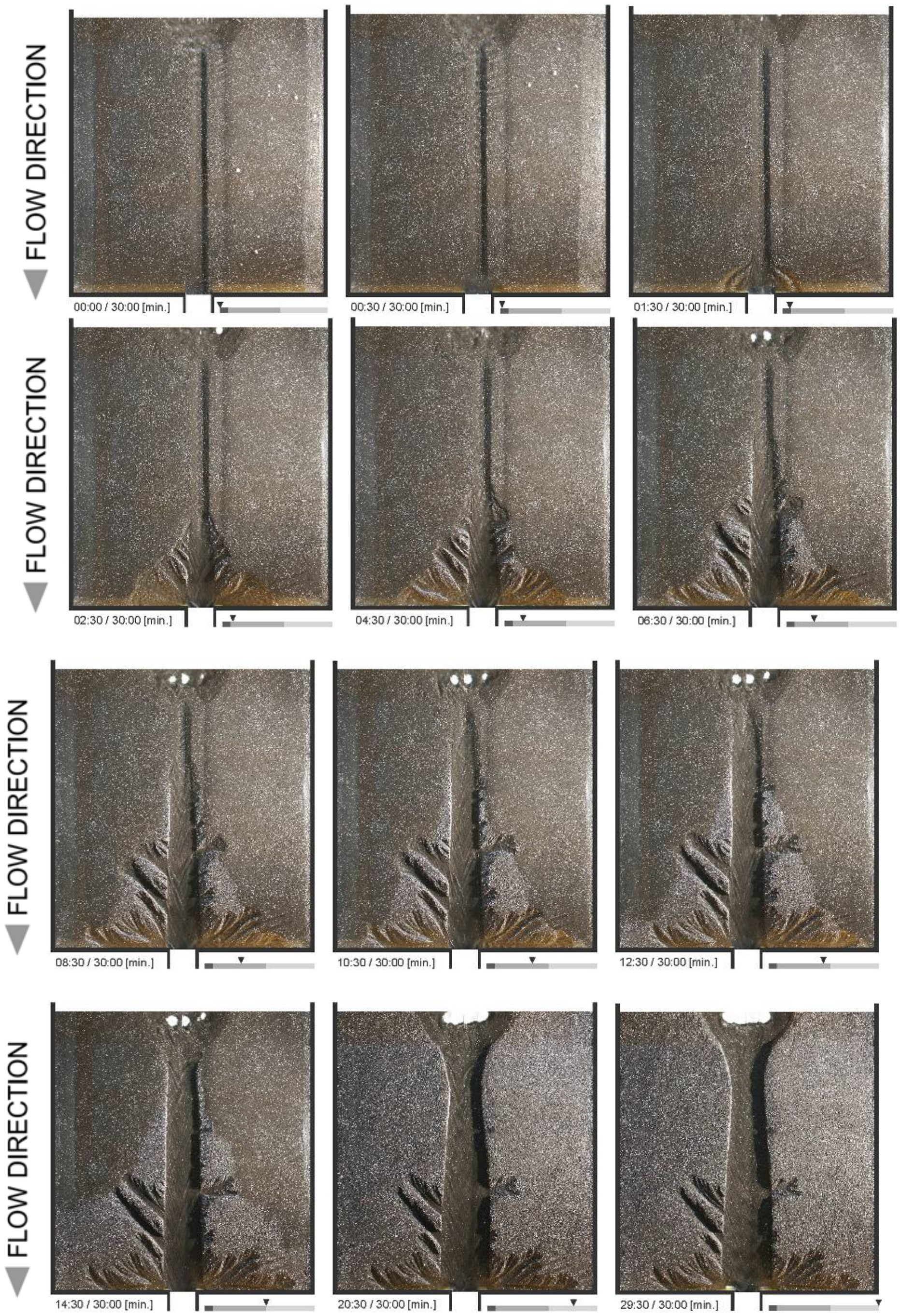

Fig. 8. Photo-series F4 (2.44 l/s)

When the water level in the model reservoir reaches the point of the initial channel, the retrograde channel erosion set in. As a consequence, the flushing channel deepened and widened until the back cutting line of erosion was entering the pothole – compare with photo-series F4; time: 12:30 (min). In this phase, the channel erosion complies with the formula of the TUM (Fig. 7). At the point of reaching the pothole, the channel widened rashly, which causes the misfit of the flushing channel width development at that time – compare with photo-series F4; time: 14:00 (min) and MORRIS and FAN [1]

Results of the model run without an initial channel

To give an insight into the developing channel structure by horizontal sediment surface without existence of an initial channel, the following photo-series of the run F4-I is attached.

As seen in the photo-series F4-I, the channel definition is much more confuse as in the case of the tests with a prepared initial channel. This series confirms the difference between the maintenance flushing and the channel formation as illustrated in MORRIS and FAN [1]. Anyhow, the adoption of the flushing channel is justified by the fact, that the surface of the sediment deposits in a reservoir is influenced by the flow of water in the impoundment. Depending on the conditions in the reservoir, there will always be a small channel defining structure in the delta. The main problem of the initial channel arrangement lies in the concrete dimension, without any adjustment on the hydrologic situation. In this manner, both – the runs with and the runs without the initial channel – represent real situations. In this way, the initial situation has been relevant for the analysis of the measured data.

5 Conclusions

The monitoring of the sediment flushing process in situ may be “well“ reproduced on the physical scale model. Under the model conditions, in the phase of the free surface flushing, several processes could be observed, for example concentration of flow into the flushing channel and retrograde channel erosion. The phase of the retrograde channel erosion could also be simulated correctly, by the applied 1-D sediment transport model. Nevertheless, the processes at the very beginning of the free surface flow phase, where the sediment transport process is governed not only by the lateral erosion of the flushing channel but also by the erosion process from the delta side areas, that could not be well described by the (TUM) relationship. Therefore, in this area, the 1-D analysis is not leading to the satisfying results. However, it should be stressed, that through the selected square layout of the reservoir model, the lateral erosion and sediment transport processes are unequally stronger weighted, than by the stretched reservoir forms, which could often been found in the Alpine regions. In the case of the stretched reservoir geometry, where the main flushed sediment volume results from the retrograde flushing channel erosion, may this be nearly assessed. Preferably good estimation of the sediment concentrations, flushing efficiency, and duration may thus be applied as argumentation basis for the feasibility studies of the sediment management methods in reservoirs and as a contribution for a sustainable resources management.

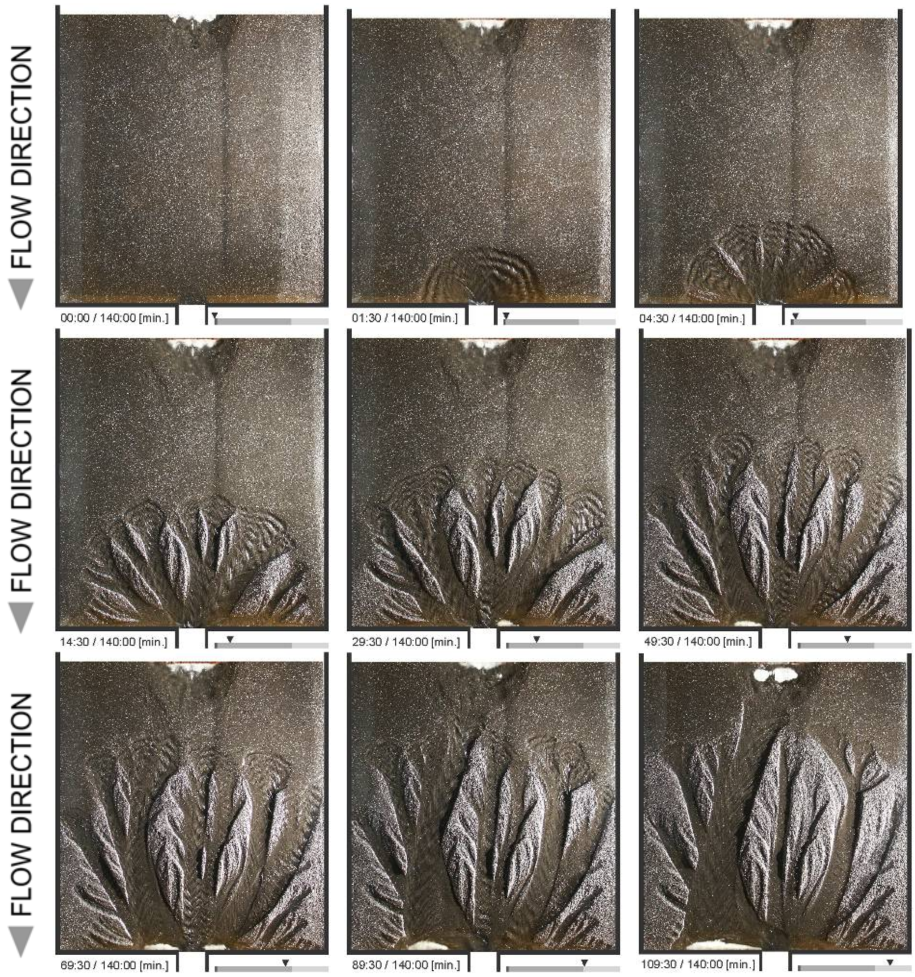

Fig. 9. Photo-series F4-I (2.44 l/s)

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A93**

Model of Preparing the Operation of Hydropower Plants in Slovak Electricity Supply System

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Abstract. The optimal operation of an electrical power supply subsystem results from respect for the effectiveness criterion for the control of an industrial enterprise. It is essential for the operation of the hydro power plants in Slovak electricity supply system. The fundamental principle of this operation is optimal hydropower utilization of the Slovak water courses. The paper describes a programmatic model for a cost-effective use of the hydropower plant's hydropotential in covering the daily load chart requirements with respect to the preoperation of other energetic sources in Slovakia.

Keywords: Electrical supply system, hydropower plant, supporting services, objective function, optimization criterion, optimization method

1 Introduction

The efficient management of the subsystems of an electrical system (ES) must principally be based on respecting the fundamental criterion of efficient ES management. This is also the governing principle of managing the important hydroelectric system within the Slovenské elektrárne (SE) national utility company – the Váh Cascade (see Fig. 1.). The core management criterion for this subsystem is the cost-effective use of the hydropotential of the SE's water sources, while facilitating all the marginal water management and energy conditions.

The preparation of hydro power plant (HPP) operation seeks to ensure reliable and efficient operation of the hydroelectric system. Planning and preparation of operation within the Dispatching Management System of the Vodné elektrárne Trenčín utility company (VET) is defined as "computations and modelling of hydro plant operation based on the input of coordinating signals from higher management level in intervals (year, month, day) and respecting the hydraulic conditions and requirements of ES to efficiently use the hydroelectric potential". One of the important elements in the HPP's pre-operating process is the computation and operation at modelling using the hydromodel of the Váh cascade ("hydro-model").

The hydro-model represents a programmatic model for a cost-effective use of the HPP Váh Cascade's hydropotential in covering the Daily Load Chart requirements with respect to the pre-operation of other SE sources (thermo-hydro coordination). The hydro-model's algorithms are programmed and tuned in the ADA95 program language environment and are currently implemented by means of an external DLL library into the complex IT environment supporting the pre-operating process of the HPPs. The hydro-model's main task is the processing of hydrological inputs and the modeling of hydraulic states and hydraulic bonds of the hydro power plant cascade on the Váh and Orava rivers (the Váh Cascade). On the output side, a performance plan at the individual HPPs is being generated with due account of their regulatory reserve and in the determined time roster, as proposed under the main efficiency criterion, namely, that the production plan should reflect as closely as possible the required shape of the business chart.

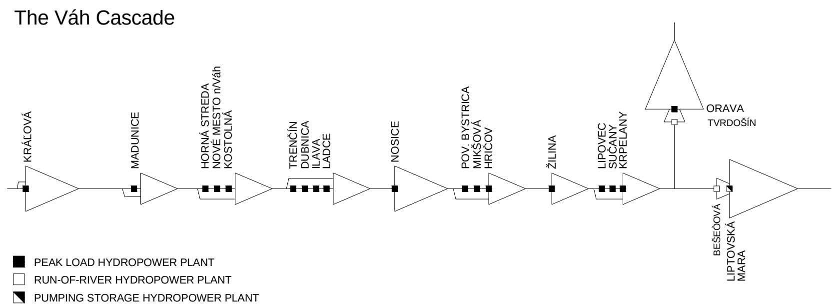


Fig. 1. Scheme of the Váh Cascade

2 The Hydro-model's Underlying Schemes

Depending on the manner of operation (the manner of the water efficiency) and the waterwork's (WW) conceptual solution, the individual HPPs of the Váh Cascade can be associated with two underlying schemes:

- Regulatory scheme
- Accumulation scheme

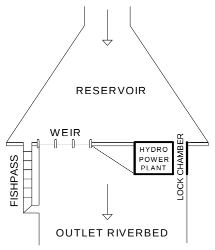
2.1 Regulatory Scheme

In operative terms, these HPPs are built on reservoirs with short-term flow regulations (daily or weekly). With respect to the waterwork's conceptual solution, the regulatory schemes can be divided into weir-surrounding schemes (Fig. 2) and channel schemes (Fig. 3).

The weir-surrounding regulatory HPPs of the Váh Cascade include HPP Žilina, HPP Kráľová and HPP Nosice. Among the regulatory channel HPPs of the Váh Cascade are the following HPP groups: Krpeľany - Sučany - Lipovec, Hričov - Mikšová - Považská Bystrica, Ladce - Ilava - Dubnica - Trenčín, Kostolná - Nové Mesto n/Váh. - Horná Streda, and the HPP Madunice which stands alone.

2.2 Accumulation Scheme

In terms of operations (water efficiency), these HPPs or pumped-storage HPPs are built at large accumulation reservoirs with long-term flow regulation (annual or more years). The accumulating reservoirs are connected to compensation reservoirs with downstream low-performance HPPs that are designed to generate electricity at given flow rates under these HPPs. The governing HPP of such a group is the HPP or pumped-storage HPP built in a direct hydraulic bond to the accumulation reservoir (Fig. 4). The accumulation schemes of the Váh Cascade HPPs include HPP: Orava – Tvrdošín and pumped-storage HPP Liptovská Mara – Bešeňová.



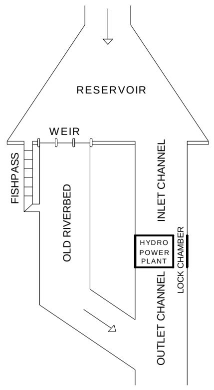


Fig. 2. Weir-surrounding scheme **Fig. 3.**. Channel scheme

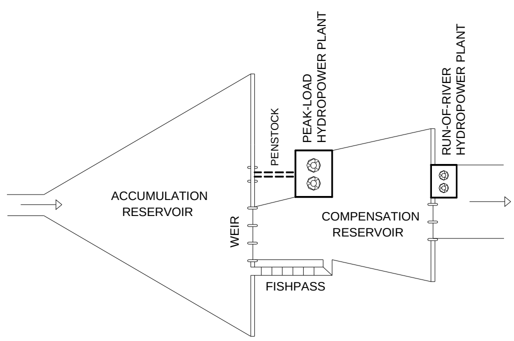


Fig. 4. Accumulation scheme

3 Underlying Computation Scheme

The hydro-model's basic computation scheme is shown in Fig. 5.

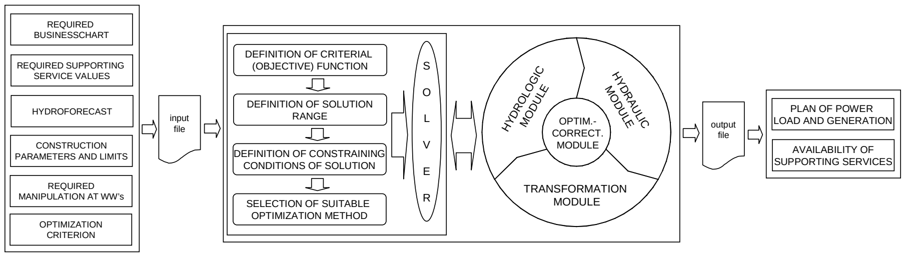


Fig. 5. Basic computation scheme of the solution

The hydro-model's main inputs are as follows:
- required business chart or evaluation;
- required values of supporting services (PpS) (i.e., primary regulation of power output (PR+/-), secondary regulation of power output (SR) and tertiary regulation of power output (TR+/-));
- hydrological forecasts from the Slovak Institute of Hydrometeorology Institute (SHMÚ);
- WW's construction parameters and limits;
- required manipulation at water structures;
- optimization criterion.

The hydro-model's main outputs are as follows:
- plan of power output and generation at individual HPPs of the Váh Cascade,
- supporting services (PpS) offered.

As a fundamental prerequisite, the search for the most efficient solution of any system must be based on derivation of the optimization criterion that, as concerns the HPP subsystem, must comply with the basic criteria of efficient management of the entire ES. The overriding optimization criterion of the pre-operation of the HPPs of the Váh Cascade is to achieve the closest business chart/proposed performance ratio as required. The optimization criterion is then defined by the so-called criterial (objective) function. The objective function can be written as:

$$min\ Z = \sum_{i=1}^{n} \left(N_i - P_i\right)^2 \ \sum_{k=1}^{m} \sum_{i=1}^{n} \left(N_i - \sum_{k=1}^{k-1} {}^{k}P_i \right) . {}^{k}Q_i \tag{1}$$

where:

i - index of time period in the planning problem, i=1, 2, ..., n

N_i - required power output in period i,

P_i - total power output produced from the HPPs of the Váh cascade in period i.

Optimization will thus be understood as minimizing the objective function $Z(P)$, which is represented by the dependent variable of parameters, whose optimum values $P^*=(P^*_1, P^*_2, ..., P^*_n)$ are to be sought. The tasks of this formulation can be resolved by using theoretical methods of optimum system management. One suitable way to solve the given optimization task which is usable for drafting the pre-operation of the HPP system is the use of a HPP system simulation model and the optimization method in the simulation model. The choice of a suitable optimization method should take due account of the fact that the accuracy of the computations in the management efficiency solution will be higher than the accuracy of the possible acquisition of the input data. In view of the above and in an effort to achieve the utmost efficiency of the HPP pre-operating process (in particular, regarding the hydro-model's "velocity"), the objective function had necessarily been transformed to linear function.

$$max\ Z' = \sum_{k=1}^{m} \sum_{i=1}^{n} \left(N_i - \sum_{k=1}^{k-1} {}^{k}P_i \right) . {}^{k}Q_i \qquad (2)$$

subject to:
${}^{k}Q_{MIN\ i} <= {}^{k}Qi <= {}^{k}Q_{MAX\ i}$
${}^{k}P_{MIN\ i} <= {}^{k}Pi <= {}^{k}P_{MAX\ i}$
${}^{k}V_{MIN\ i} <= {}^{k}Vi < = {}^{k}V_{MAX\ i}$
${}^{k}V_{i} = {}^{k}V_{i-1} + {}^{k}I_i - {}^{k}Q_i.t$
where:
t - time period,
k - index of HPP, k=1, 2, ..., m (set of downstream)
${}^{k}Q_i$ - discharge for HPP k in period i,
${}^{k}Q_{MIN\ i}$ - minimum discharge for HPP k in period i,
${}^{k}Q_{MAX\ I}$ - maximum discharge for HPP k in period i,
${}^{k}P_i$ - power output produced by HPP k in period i,
${}^{k}P_{MIN\ i}$ - minimum power output limit of HPP k in period i,
${}^{k}P_{MAX\ i}$ - maximum power output limit of HPP k in period i,
${}^{k}V_i$ - storage volume of the reservoir k at the end of period i,
${}^{k}V_{MIN\ i}$ - minimum storage of the reservoir k in period i,
${}^{k}V_{MAX\ i}$ - maximum storage of the reservoir k in period i,
${}^{k}I_i$ - net inflow volume to the reservoir k in period i, including seepage and evaporation losses, and diversions.

As all coefficients of the task (coefficients of all the functions) are real numbers, the problem of the HPP's efficiency of operations could have been transformed to a linear programming task that can briefly be stated as the following matrix:

for *k = 1 To m*
$max\ z=(c)^T.Q$ in compliance with $A.Q \leq b$, $Q \geq 0$
next *k*

where,
A - matrix of structural coefficients,
c - vector of evaluations,
Q - vector of solutions (optimum flow through HPPs profile),
b - vector of restrictions.

In the hydro-model, the solution of the defined task of the linear programming is arranged by using the Simplex method. The parameters for computing the values of the structural coefficients of the "A" matrix and the "b" vector of the restrictions (the vector of the right sides) are arrived at by computations made in the hydromodel's additional modules.

3.1 Hydrological Module

The hydrological module provides data as to the flow management in the individual elements of the system. Its task (output) is to provide real data on available water flows for the individual HPPs. The hydrological module is based on the following:

- system's elements/flow management impact rate;
- determination of the priority bonds among the system's reservoirs.

Based on the setting of the possible bonds (interactions) and the system's elements/flow management impact rate, the hydromodel can be divided as follows:

- sections: profile of the reservoir's intake – HPP intake profile;
- sections: outlet profile from the group-ultimate HPP – intake profile of the next HPP group;
- sections: weir (profile of an idle outlet from the HPP group) – the orifice of the ultimate HPP's outlet channel to the old riverbed.

The hydrological module serves to provide data on flows in the sections between the individual HPPs or HPP groups that, although hydraulically uncoupled, have a substantial impact on the time sequence of the regulated flows, and, as a result, impact the engagement of the individual HPPs in time. The "inefficient" sections include: Bešeňová – Krpeľany, Tvrdošín – Krpeľany, Lipovec – Žilina, Žilina – Hričov and Madunice – Kráľová. Additionally, the hydrological module also serves to provide data on flows in the sections located between a weir (the profile of an idle outlet from the HPP group) and the orifice of the group-ultimate HPP's outlet channel to the old riverbed.

3.2 Hydraulic Module

The purpose of the hydraulic module is to provide data on heads for the individual HPPs. It permits to determination the following characteristics:

- water surface level in the reservoir;
- hydraulic losses in the HPP intakes and outlets;
- hydraulic bonds of the individual HPPs.

The proposed structure of the hydraulic module stems from the configuration of the HPPs. In terms of solving the hydraulic bonds and losses, the module is divided as follows:

- reservoirs;
- channels divided into sections: reservoir – HPP, HPP – HPP, HPP – reservoir of the next HPP group (the downstream water surface level is impacted by the operation of the next HPP group), HPP – inefficiently used flow section;
- penstocks (section reservoir – HPP).

The job of the reservoir's hydraulic module is to set, on the basis of the supply volume, the reservoir's immediate water surface level or the reservoir's immediate water supply volume as based on the water surface level. The job of the hydraulic module of the channels is to set the hydraulic losses resulting from the water flow in these structures. The job of the hydraulic module of the pressure inlet channels (pipes) is to set the hydraulic losses resulting from the pressured water flow in these structures.

3.3 Transformation Module

The purpose of the transformation module is to transform, based on the results of both the hydrological and hydraulic modules, the flow rates (hydrological modules) and heads (hydraulic module) to electrical power. Additionally, the transformation module also has the job of determining the values of the flow rates equivalent to the required supporting service values. The transformation of the required regulatory outputs to the equivalent flow rates is apparent from the scheme in Fig. 6.

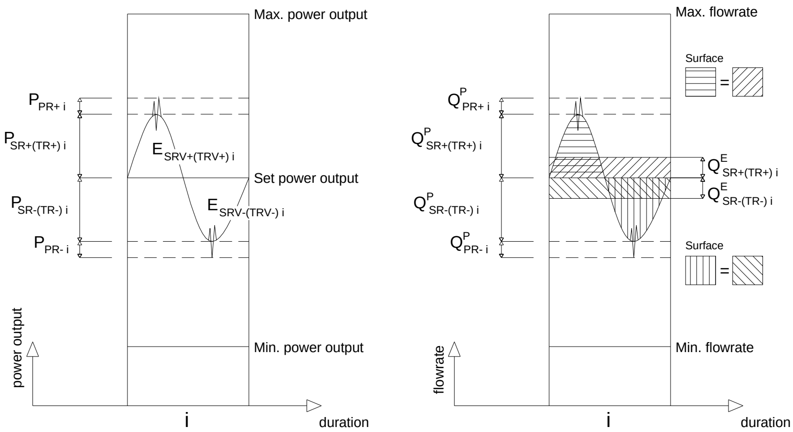


Fig. 6. Transformation of the required regulatory outputs to the equivalent flow rates

3.4 Module of Optimization and Correction

The purpose of the optimization module is to propose, based on the existing conditions and marginal criteria of water management and energy management, as well as other criteria, the output plan for the individual HPPs of the Váh Cascade, while using the main optimization criterion. The HPP Output Plan with headroom for a regulatory reserve in the determined time roster is proposed for the stated optimization interval. The corrective part of the efficiency module serves to make corrections of the values of the structural coefficients of the *"A"* matrix and the *"b"* vector of restrictions for and after each computation of the values of the solution vector $Q=(Q_1,Q_2, ..., Q_n)$. Subsequently, more accurate values of the *"Q"* solution vector will be recomputed using the corrected values.

4 Conclusions

VET has been applying efficiency solutions for preparing operation and real-time operative management for more than 20 years. Nevertheless, the issue of efficient hydro power plan operations failed to be adequately solved. The complexity of efficient HPP management arises mainly in the cascades operation, where the individual HPPs are interconnected by complex hydraulic bonds. An additional problem is posed by the fact that efficient load distribution among the individual energy output facilities based on the criterion of regime efficiency has failed to be resolved by the Slovak Energy Dispatching Center (SED). Currently, the operation of the individual ES sources based on the system's estimated load balance and staff experience is being prepared. As a result, the coordinating signals for the management of the subsystems coming from the first level ES management SED tend to be inaccurate and mere estimates, in which case any accurate efficiency solution for management of the Váh Cascade HPPs fails to comply with the principle of intercompatibility of objectives on individual management levels, and fails to comply with the global objective as set by the first management level.

Acknowledgement

This work was supported by the Science and Technology Assistance Agency under Contract APVT No. 20-046602 and APVT No. 20-046302.

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A94**

Impact of New Trends in Programming and Marketing of Electricity to the Method of Operation of the Hydropower Plants in Slovak Electricity Supply System

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Abstract. A considerable attention has been paid to the optimal utilization of the dynamic characteristics of the hydropower plants in electric supply system of Slovakia. For more than 20 years, simplified optimization solutions have been used for the operation planning as well as the real time operative control. The issue of the assuring the support services in the field of the primary, secondary and tertiary regulation of power output has been increasingly occurring by the operation of the hydropower plants in the last years. The article describes suitable methods for the solutions of the problems of the hydropower plant control related to the new tendencies in the planning and the market with electrical energy, while primary taking into account their generality in the aspect of the planning and providing the support services.

Keywords: Electrical supply system, hydropower-thermal system, hydropower plant, primary power regulation, secondary power regulation, tertiary power regulation, volume equivalent

1 Introduction

To generate and to supply the electrical supply system (ESS) with as much power as is being demanded at the same moment (including transmission losses) is important mainly because the electric power can not be stored in a larger scale and subsequently used in the time of increased power demand.

The sources, which are assuring the balance of the energy system between the power generation and the power demand, provide the so-called support services (PpS) to the electric supply system.

The support services can be divided into:

- primary power regulation (PRV),
- secondary power regulation (SRV),
- tertiary power regulation (TRV),
- quick and the cold dispatch backup.

The power in the ESS which is reserved for the support services assuring is called the **regulation (power) backup**.

2 Efficient Load Distributions in a Hydropower-Thermal System

Whether at the production of the base power or the assuring of the PpS, the objective of the power producer should be to maintain the maximum operational effectiveness of the power system. The simplest and the most common criterion for the optimal allocation of the load between the power sources of the system is the achievement of the minimal production costs with the respect to the limiting conditions, which are linked to this criterion in the specific time and space [2]. Based on the previous, for a hydropower-thermal system, which consists of m thermal and n hydropower plants, can be the total fuel costs Nc in a regulation period T expressed as following:

$$N_c = \sum_{j=1}^{T}\sum_{i=1}^{m}\left({}^{s}N_{i,j}({}^{s}P_{i,j}) + {}^{PpS}N_{i,j}({}^{s}P_{i,j}, {}^{PpS+}P_{i,j}, {}^{PpS-}P_{i,j})\right) \rightarrow min \tag{1}$$

where:

j	the index of the time interval of the solution within the total regulation time T, j=1 2, .., T,
i	the index of the thermal power plant, i=1, 2, .., m,
${}^{s}N_{i,j}$	the fuel costs of the i thermal power plant during the j hour at the base power generation,
${}^{s}P_{i,j}$	the power output (base power) of the i thermal power plant (TPP) during the j hour (MW),
${}^{PpS}N_{i,j}$	the fuel costs of the i thermal power plant during the j hour, related to the providing of the PpS (costs/hour),
${}^{PpS+}P_{i,j}$	positive regulation power output of the i thermal power plant during the j hour (MW),
${}^{PpS-}P_{i,j}$	negative regulation power output of the i thermal power plant during the j hour (MW),

For the balance of the power outputs (base power) in the j hour of the regulation period T can be written:

$${}^{s}P_j = \sum_{i=1}^{m} {}^{s}P_{i,j} + \sum_{k=1}^{n} {}^{s}P_{k,j} \tag{2}$$

where:

k	index of the hydropower plant (HPP), k=1, 2, .., n,
${}^{s}P_j$	total required base power during the j hour (MW),
${}^{s}P_{k,j}$	base power of the k hydropower plant during the j hour (MW),

For the balance of the power backup during the *j* hour stands in the positive or negative course of the deviation following:

$$ {}^{PpS+}P_j = \sum_{i=1}^{m} {}^{PpS+}P_{i,j} + \sum_{k=1}^{n} {}^{PpS+}P_{k,j} \text{ or } {}^{PpS-}P_j = \sum_{i=1}^{m} {}^{PpS-}P_{i,j} + \sum_{k=1}^{n} {}^{PpS-}P_{k,j} \tag{3} $$

where:

${}^{PpS+}P_j$ total required positive regulation backup during the *j* hour (MW),
${}^{PpS+}P_{k,j}$ positive regulation backup of the *k* hydropower plant during the *j* hour (MW),
${}^{PpS-}P_j$ total required negative regulation backup during the *j* hour (MW),
${}^{PpS-}P_{k,j}$ negative regulation backup of the *k* hydropower plant during the *j* hour (MW),
The diversion of the load between thermal and hydropower plants with the respect to the entire boundary water management and energetic conditions and fulfilling the criterion (2) will be optimal and the total fuel costs in the energetic system will be minimal.

2.1 Load Distribution between the Energy Sources of the Slovenské Elektrárne, Co.

The system of the energetic sources of the Slovenské elektrárne company, which operates 2 nuclear, 2 thermal and 34 hydropower plants, can also be considered as a hydropower-thermal system. Apart from the complicated analytic expression of the costs related to the providing of the support services, the complexity of the optimal distribution of the load between the TPPs and the HPPs is caused mostly by the operation of the hydropower plants in the cascade (the Váh Cascade), where the hydropower plants are interconnected between themselves by complicated hydraulic links.

The complexity of the support services distribution is also caused by the fact that the ability of providing support services for the ESS depends on the type of the hydropower plant, as well. To express the suitability of the particular types of the hydropower plants for the PpS providing is a very complex and complicated task.

In the real process of the planning of the energetic system's operation, the distribution of the support services between particular sources is rather based on the experience of the dispatch operators.

In the first step, the percentage of the generation unit's ability to provide power backup with taking into account the backup's use for the providing of the support services of a higher rank is specified (e.g., for estimating the ability of the unit to provide the power backup for the tertiary regulation, the backup for the primary and the secondary regulation is taken into account). In the next step, the base power load distribution between the HPP and the TPP is based on the regime efficiency criterion expressed as follows:

$$ F = \sum_{j=1}^{T} \sum_{k=1}^{n} b_j \left({}^{s}P_j - \sum_{k=1}^{k-1} {}^{s}P_{k,j} \right) . {}^{s}P_{k,j} \rightarrow max \tag{4} $$

where b_j is the relative increase of the fuel costs during the j hour for all of the TPP (costs/MWh).

The values of the ${}^{s}P_{k,j}$ are determined by the maximization of the objective function F supplemented by the limiting conditions, which are mostly based on the constraints in the operation manuals of the water structures or given by the constructional and operational parameters of the HPPs.

$$\overset{\min}{P}_{k,j} + {}^{PpS-}P_{k,j} \leq {}^{s}P_{k,j} \leq \overset{\max}{P}_{k,j} - {}^{PpS+}P_{k,j} \tag{5}$$

$$\overset{\min}{HN}_{k,j} \leq HN_{k,j} \leq \overset{\max}{HN}_{k,j} \tag{6}$$

$$HN_{k,T} = {}^{poz}HN_{k,T} \tag{7}$$

$$V_{k,j} = V_{k,j-1} + {}^{prítok}V_{k,j} - {}^{odber}V_{k,j} \tag{8}$$

where:

$\overset{\min}{P}_{k,j}$, $\overset{\max}{P}_{k,j}$	min and max attainable power output of the k HPP during the j hour (MW),
$\overset{\min}{HN}_{k,j}$, $\overset{\max}{HN}_{k,j}$	min and the max operational water level in the reservoir of the k HPP during the j hour (m a.s.l.),
$HN_{k,j}$	water level in the reservoir of the k HPP in the end of the j hour (m a.s.l.),
$HN_{k,T}$	water level in the reservoir of the k HPP in the end of the T (m a.s.l.),
${}^{poz}HN_{k,T}$	required water level in the reservoir of the k HPP in the end of the T (m a.s.l.),
$V_{k,j}$	storage volume of the reservoir of the k HPP in the end of the j hour (m^3),
${}^{prítok}V_{k,j}$	total volume of water flown into the reservoir of the k HPP in the j hour reduced by the evaporation losses, leakage and other not energetic withdrawals (m^3),
${}^{odber}V_{k,j}$	total volume of water withdrawn for energetic purposes from the reservoir of the k HPP during the j hour (m^3).

2.2 The Effect of Providing the Support Services on the Balance of a Reservoir's Storage Volume

The scheme in the fig.1 shows that the value of the ${}^{odber}V_{k,j}$ is based on the nature of the provided PpS. Based on its nature, for the *primary power regulation* is the PRV+ or the PRV- volume equivalent neglected. For its planning, only the assigning of the required power backup is taken into account.

For the *secondary power regulation*, the volume of water required for the SRV+ or SRV- can be expressed as follows:

$$^{SRV+(-)}V_{k,j} = \int_{j-1}^{j} {}^{SRV+(-)}Q_k(t).dt \tag{9}$$

where $^{SRV+(-)}Q_k$ is the actual discharge equivalent of the SRV+(-) of the k HPP [m^3.s^{-1}]

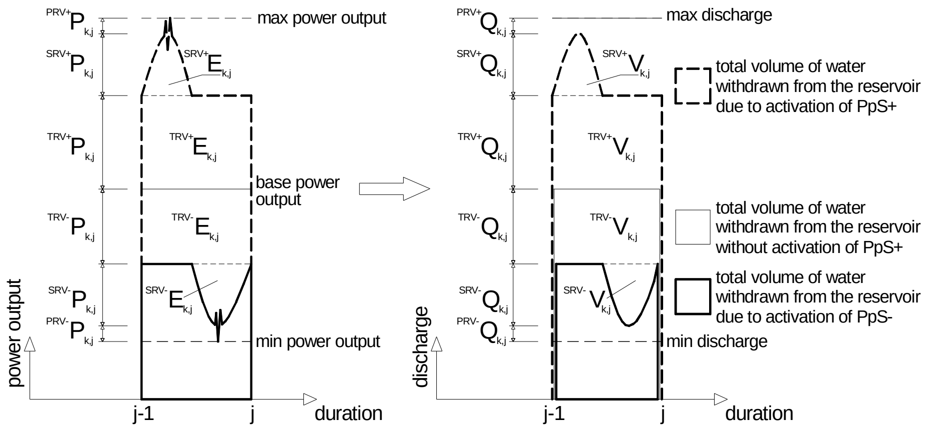

Fig. 1. The estimation of the water volume withdrawn for the energetic purposes of the k HPP's reservoir during the j hour

For the providing of the *tertiary power regulation* is also necessary to take into account the volume change of the water withdrawn from the reservoir. The expression for the TRV+ or TRV- volume equivalent is as follows:

$$^{TRV+(-)}V_{k,j} = \int_{j-1}^{j} {}^{TRV+(-)}k.{}^{TRV+(-)}Q_k(t).dt \tag{10}$$

where:

$^{TRV+(-)}Q_k$ actual TRV+(-) discharge equivalent of the *k* HPP (m^3.s^{-1}),

$^{TRV+(-)}k$ TRV's utilization coefficient, expressing the uncertainity rate of the moment of the tertiary power regulation load in the total regulation period *T*. It can acquire values in the range <0~1>. If the $^{TRV+(-)}k$ value equals 1, it means that the TRV is assumed to be activated in every hour of the *T* [-]

Then, for the computation of the $^{odber}V_{k,j}$ stand following:

$$^{odber}V_{k,j} = {}^{0}V_{k,j} + {}^{SRV+}V_{k,j} - {}^{SRV-}V_{k,j} + {}^{TRV+}V_{k,j} \tag{11}$$

or

$$^{odber}V_{k,j} = {}^{0}V_{k,j} + {}^{SRV+}V_{k,j} - {}^{SRV-}V_{k,j} - {}^{TRV-}V_{k,j} \tag{12}$$

The effect of providing the support services on a reservoir's operation water level is shown on the scheme in Fig. 2. Meeting the (13) and (14) boundary conditions for both limiting water level regimes should maintain a "safety pillow" for providing the planned support services in a range of the total regulation period T.

$$\overset{\min}{HN}_{k,j} \leq {}^{PpS+}HN_{k,j} \leq \overset{\max}{HN}_{k,j} \tag{13}$$

$$\overset{\min}{HN}_{k,j} \leq {}^{PpS-}HN_{k,j} \leq \overset{\max}{HN}_{k,j} \tag{14}$$

where:

${}^{PpS+}HN_{k,j}$ water level inf the reservoir of the k HPP at the end of the j hour when the PpS+ is activated (m a.s.l.),

${}^{PpS-}HN_{k,j}$ water level inf the reservoir of the k HPP at the end of the j hour when the PpS–

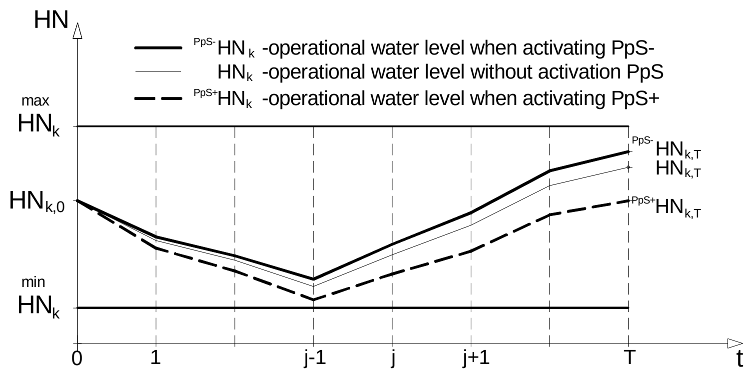

Fig. 2. The effect of activating the support services on the operation water level regime in a reservoir

The above shows that an important factor for assessment of the PpS planning is the real estimation of the water levels when activating the particular PpS. This estimation depends on determination of the ${}^{SRV+(-)}k$ and ${}^{TRV+(-)}k$ coefficients. These have to be determined so that they describe as precisely as possible the share of the particular PpS on the changes of the reservoir's available storage volume, guarantee providing the PpS during the whole regulation period and at the same time do not restrain the regulation abilitiy of the reservoir.

According to the operators of the Dispatch centre of the hydropower plants in Trenčín, for the estimations of the limiting operational water levels, the values of the ${}^{SRV+(-)}k$ between 0~0,1 and ${}^{TRV+(-)}k$ between 0,2~0,25 give the most real results.

3 Conclusions

The described methodology of the planning of the support services on the hydropower plants has been implemented into the model of the planning and operative control of the operations of the hydropower plants since 2007. The model is a part of the complex information system of the operations planning of the energetic sources of the Slovenské elektrárne, Co..

In the model, the solution of the optimization objective, which is described by the criterion function (4), is based on the modified simplex method with the corrective algorithm enabling a quick convergence to the optimal result. Although the behaviour of the criterion function is nonlinear, for the selection of this method has been the determining criterion the request of the shortest computational time of the optimization as possible.

The methodology of the optimal load allocation between the thermal and the hydropower plants with the reserving of power backup for the support services assuring, which is based on the energy producer so called regime efficiency, is universal only in the case that the ESS operator is identical to the power producer or the case that the power producer's capacity totally covers the demands of the whole ES. In other cases, the conditions of the optimal load allocation much more complicated. It is caused mostly by the fact that the base power and the support services have a substitute character. It means that the increased production of the one product requires the decrease of the production of the second product. Thus a situation may occur that some sources with low marginal costs (e.g. hydropower plants) will be allocated from the base power generation to the providing of the PpS. This will decrease the system operator's costs but at the same time it will lead to the increase the prices in the energy market, because these sources will be replaced by the ones with higher costs. The decrease of the PpS costs of the system operator would be at the expense of the consumers in the energy market. The situation is complicated also by the facts that the price for the particular types of the support services is not defined and the support services market, which would generate their prizes, is not created. The prize for particular types of the support services is defied by a temporary apparatus, which is based on the amount of the finances available to the supplier of the PpS - the operator of the electric supply system after the confirmation from the regulatory office.

Acknowledgement

This work was supported by the Science and Technology Assistance Agency under Contract APVT No. 20-046602 and APVT No. 20-046302.

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A96**

Flood Protection of Adamovské Kochanovce

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Abstract. The contribution contains reasons and the way of the flood protection solution of Adamovské Kochanovce - beautiful village with rich history, many cultural monuments and natural reserve. This village is often attacked by floods, which cause high economical, cultural and social damages. After reconnaissance and an agreement with village management the problem has been solved by building a polder above the village for controlling part of the Adamovsky Creek. The goal of the project is to design according to hydrological data the dimension parameters of the polder, i.e. the height of the dam, outlet structure capacity, spillway capacity, stilling basin and necessary river bed lining. The attention is given to stability problems, as well.

Keywords: Floods, flood protection, retention dam, design, hydraulic calculations

1 Introduction

Floods are the natural part of the hydrologic cycle. In some countries man got up to exploit overflowed water to his credit, in the other places he fights against it uselessly. Floods have many negative effects. Primary effects mean physical damages (damaged structures, bridges, sewage systems, buildings, cars...) and casualties (people and livestock die due to drowning or subsequent epidemics and diseases). Secondary effects mean e.g. contamination of water, unhygienic conditions, dying of non-tolerant species of flora. Tertiary effects mean long-term economic effects.

There are many ways of the flood protection - active or passive. But we can not generalize their application, because each locality is different and there are many parameters entering the design. On the bigger streams and rivers, the flood protection is realized by dikes and water schemes. But on the small streams such solution is ecologically and economically impossible. Therefore building retention dams called "polders" is very progressive solution nowadays. "Polder" is an effective water structure, which has no noisy effect in the nature and it can be even aesthetic part of the country.

As many villages located at the root of a mountain, Adamovské Kochanovce village has big troubles with floods, too. Troubles occur especially during the short summer storms with high intensity and in the spring, when snow is melting and a rain comes. After an agreement with village management, I decided to solve the flood protection of this village by building a „polder"on the Adamovsky Creek and regulation of a part of this stream.

Polder is a type of retention reservoir, which has to retain the flood wave and prevent the odd amount of water from flowing out the river bed. Village will be protected against the static impacts of the water (overflowing) as well as dynamic impacts (erosion, sediment transport, damaging of the roads and bridges).

2 Input Data for Flood Protection Design

There are many input data for the flood protection design, especially:

- *description of a place (city/town)* (inhabitants, monuments, relics, nature reserves, springs, wetlands, ...)

- *site character* (description of the catchment, land use, inclinations, length of the stream, objects on the stream ...),

- *map data* (map lists number 35 12 24, 35 12 25, 35 14 04 and 35 14 05 from Geodetic and Land Register Institute Bratislava in 1:10 000 scale were used for this project (design)),

- *hydrologic data* (rainfall, runoff and temperature data, in this project (design) these data were used from the gauging station Trenčín. Design flood wave was calculated from information about the catchment, land use, temperatures, rainfall and runoff.),

- *geological, pedological and morphological data* [1] (information about bedrock, landslides, soil types and soil sorts ...)

- *geodetic data* (information about existed geodetic points, topographical survey of the area)

-*hydraulic data* (roughness of the stream – Manning's n, slope, capacity of the objects

3 Design Method

Maps had to be converted and adjusted according to our requirements. Then I had to choose the profile of the planned polder – specific site, where this polder has to be built. I decided to build a dam 350 m above a bridge in a village, to provide enough space to retain water, but not to endanger any people or buildings.

Afterwards, design flood wave had to be calculated – that means the retention volume of a polder, volume of water which polder is able to retain. I calculated the flood wave using DesQ 5.2 Software by CN numbers method. CN numbers represent runoff. This method uses information about rainfall and runoff, land-use and morphology of a catchment.

Designed flood wave of Adamovsky Creek has culmination Q_{100} of 7.2 $m^3.s^{-1}$, time of rise of flood wave is 4 hours, and time of recession of flood wave is 9 hours. Volume of the flood wave is approx. 140 000 m^3. Designed flood wave is in Fig. 1.

Retention volume of a polder is represented by ABC area in Fig. 2. Point C means capacity of a channel under the dam Q_K (calculated later). In the first step it is useful to match point A with C by straight line and simply measure the area ABC, which represents the retention volume of a polder provided – 56 200 m^3.

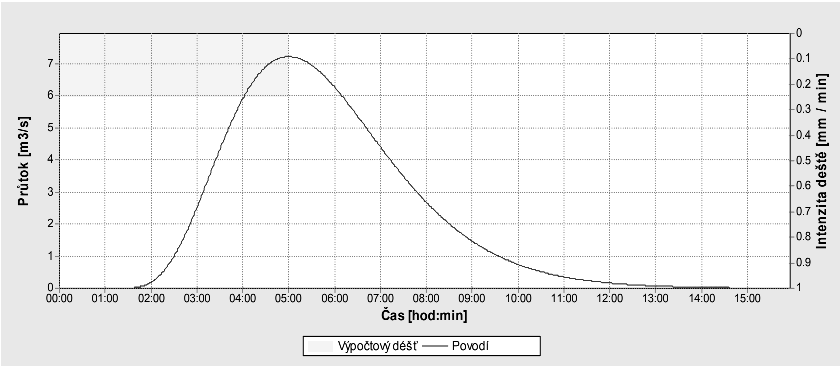

Fig. 1. Designed flood wave with culmination 7.2 $m^3.s^{-1}$

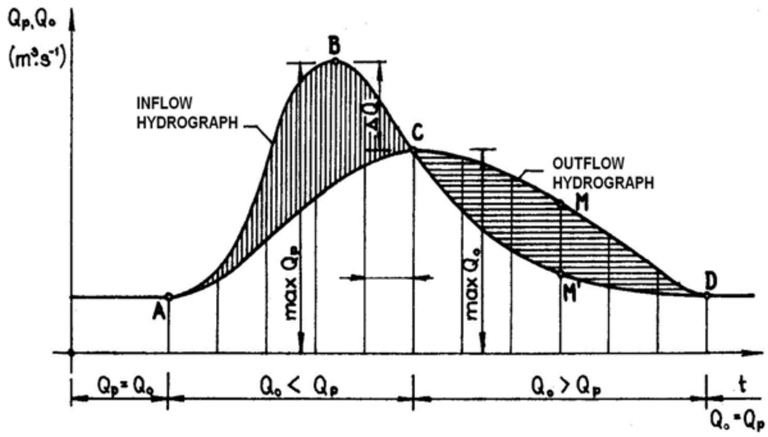

Fig. 2. Attenuation and lagging of routed hydrograph. Inflow Q_P and outflow Q_O hydrographs. Q = f (time). Water entering storage is hatched vertically; water leaving storage is hatched horizontally

The height of a dam required was subtracted from the reservoir surface elevation curve (Fig. 3) and the reservoir storage elevation curve (Fig. 4). To ensure required

retention volume 56 200 m^3, the dam has to be 5 m high. Values of retention surfaces and retention volumes belonging to various heights of a dam are in Table 1.
Material of a dam is designed to be concrete with quarry stone lining.

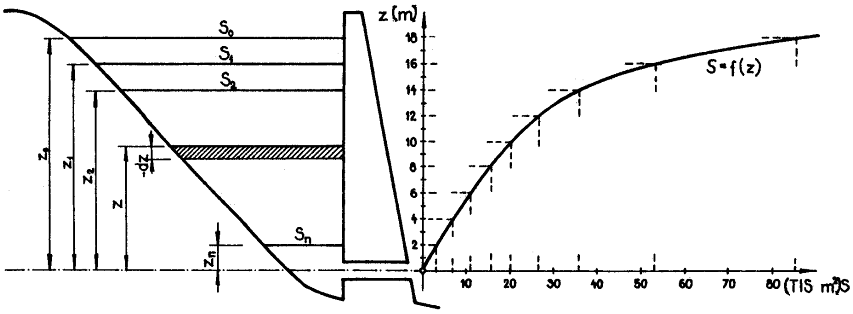

Fig. 3. Reservoir scheme and the reservoir surface elevation curve. Reservoir surface S is depending on a height of a dam z

Table 1. Reservoir surfaces and volumes according to the height of a dam on Adamovsky Creek in Adamovské Kochanovce

Dam height H	Reservoir surface S	Reservoir volume V
[m]	[m^2]	[m^3]
0	0	0
1	3198.5	1599.2
2	6397.0	6397.0
3	11514.6	15352.8
4	20726.2	31473.1
5	28914.4	56293.4
6	42220.1	91860.6

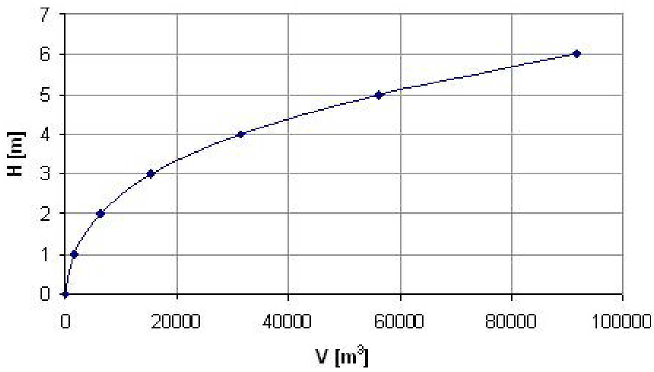

Fig. 4. Reservoir storage elevation curve, volume of the retention space V is depending on the dam height H

4 Rating Curve

Calculation of discharge in Adamovsky Creek (Fig. 6) was performed by Chezy´s formula (1) provided uniform flow [5]:

$$Q = C \cdot A\sqrt{R \cdot i_o} \tag{1}$$

$$C = \frac{1}{n} \cdot R^{\frac{1}{6}} \tag{2}$$

where: R is the hydraulic radius A/O (m), A is cross-sectional area (m^2), O is wetted perimeter (m), n is Manning's roughness coefficient, i_0 is longitudinal slope (-) and C is Chezy´s coefficient ($m^{0,5}.s^{-1}$).

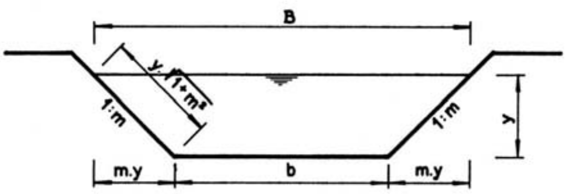

Fig. 5. Trapezoid channel with its basic parameters

Designed channel cross section is a trapezoid (Fig. 5) with bottom width b=1 m, bank slopes 1:2 (m=2) and longitudinal slope i_o=1.9%. Hydraulic roughness is entered by Manning roughness coefficient n=0,033 (it means bank fortification is quarry stone).

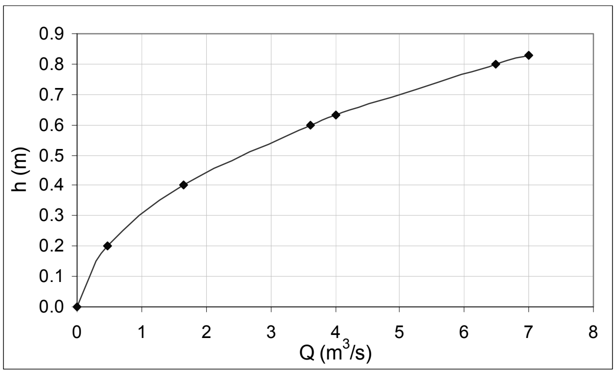

Fig. 6. Rating curve Q=f(h)

5 Channel Capacity Discharge under the Dam

It is necessary to find a critical cross section (area with the lowest capacity) on the channel. There are potentially three critical places under the designed polder. The culvert under the bridge, with capacity of 11.2 $m^3.s^{-1}$, the spillway of the anti-fire retention dam with capacity of 11.96 $m^3.s^{-1}$ and the pipe under the village with capacity of 4.48 $m^3.s^{-1}$. The lowest value from mentioned above is 4.48 $m^3.s^{-1}$, therefore I designed the channel capacity discharge $Q_K = 4.0\ m^3.s^{-1}$ for river bed below the polder. Then the maximum depth in the river bed is 0.63 m (according to the rating curve values- Fig. 6).

6 Outlet, Spillway and Stilling Basin Design

Basic signatures used in calculation are:

- ground elevation of a river bed above and under the dam KDP = KDT (m a.s.l.);
- water level in the dam KH 9 (m a.s.l.);
- water level in the channel under the dam KD (m a.s.l.);
- maximum water level in the dam KH_{max} (m a.s.l.);
- level of spillway crest KP (m a.s.l.);
- elevation of upper edge of the outlet KHO (m a.s.l.);
- designed discharge Q_{100} (m^3s^{-1}) ;
- channel capacity discharge Q_K (m^3s^{-1}).

The spillway was designed in trapezoidal shape, bottom width 2 m, height 1 m and bank slope 1:1. The capacity of the spillway was calculated using general Dubuat equation for a spillway (3) as follows

$$Q_P = \frac{2}{3}\mu_p \cdot \sqrt{2g} \cdot \left(\frac{b}{h} + \frac{4}{5}m\right) \cdot h^{5/2} \tag{3}$$

We assume, that $Q_O = Q_K$ (outflow matches the capacity of the channel under the polder), what is the basic anticipation for designing of a retention dam. The width of the outlet b_o is 1 m and height a_o is 0,66 m (from outlet equations) [3].

$$a_o = \frac{Q_K}{\mu_v \cdot b_o \sqrt{2g \cdot H}} \tag{4}$$

Outlet capacity had to be calculated for 3 situations:

1. Water level is lower than the upper edge of the outlet, this event is solved as a discharge over broad – crested weirs expressed by the equation [3]

$$Q_o = \varphi \cdot b_o \cdot h_1 \sqrt{2g(h_0 - h_1)} \tag{5}$$

where: h_1=KH–KHO, h_0 is water level in the dam KH above bottom edge of the outlet; counted with velocity head (7), and φ is a coefficient of overfall discharge.

$$h_0 = z + \frac{\alpha \cdot v_0^2}{2g} \tag{6}$$

2. Partially submerged orifice was calculated as a total of free and submerged outlet as follows,

$$Q_o = \mu_v \cdot b_o \sqrt{2g}\left[\frac{2}{3}\left(H^{\frac{3}{2}} - h_1^{\frac{3}{2}}\right) + y_d\sqrt{H}\right] \tag{7}$$

where: h_1=KH–KHO, H=KH–KD, and y_d is water depth under the dam.
Orifice coefficient μ_v was calculated from Eq. (3):

$$\mu_v = -0{,}3376\left(\frac{h_1}{H}\right)^2 + 0{,}3772\left(\frac{h_1}{H}\right) + 0{,}6104 \tag{8}$$

3. Submerged orifice discharge was calculated from

$$Q_o = \mu_v \cdot a_o \cdot b_o\sqrt{2g \cdot H} \tag{9}$$

Flowing through the polder during the flood situation is characterized by rating curve of the object complex (outlet and overflow together), $Q = Q_O + Q_P = f(H)$ (Fig. 7.).

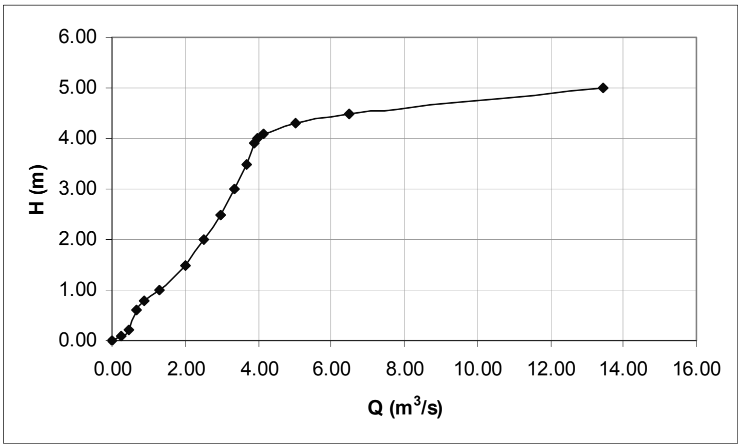

Fig. 7. Rating curve of the whole object

The stilling basin is a most common form of energy dissipater converting the supercritical flow from the spillway into subcritical flow compatible with the downstream channel regime. However, in this case supercritical flow occurs under the dam. Critical depth in the channel is 0.71 m, but water depth from the rating curve is 0.63 m. I designed only a stilling basin of minimum parameters – 0.3 m deep, 4 m wide and 5 m long.

7 Transformation of a Flood Wave

Transformation of a flood wave means reduction of a culmination discharge – flattening and extension of a flood wave peak (Fig. 8.) [5]. Calculation of transformation was made graphically by Klemeš method [4]. Principle of the calculation is in equation (10).

$$(Q_p - Q_o)dt = S.dH = dV \tag{10}$$

Building a retention dam on Adamovsky Creek lowered dangerous culmination discharge 7.2 $m^3.s^{-1}$ to 4 $m^3.s^{-1}$.

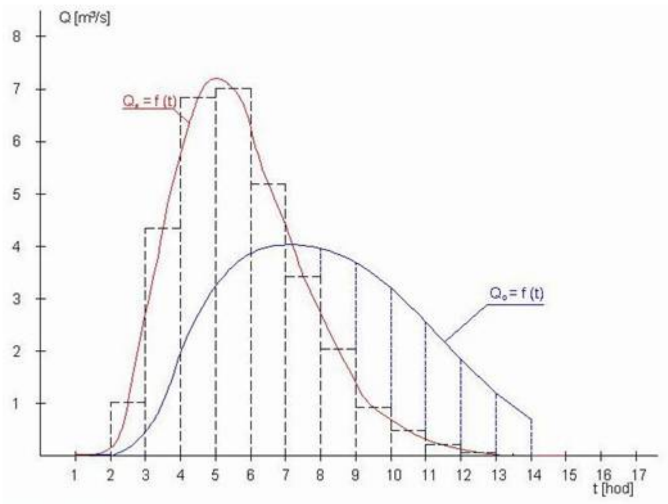

Fig. 8. Transformation of a flood wave on Adamovsky Creek by a retention reservoir

8 Stability

According to geological visual inspection, there are two types of soil: clayey gravel (GM) and clay with high plasticity (CH). Forasmuch as we don't know exact parameters, I calculated the stability for more values (Table 2). Exact results will be known only after detailed geological survey.

Slope stability was solved by Geo5 software using methods of Bishop, Petterson and Sarma. Slopes are unstable for the most dangerous situation – empty polder after the flood, so I suggested a gabion fortification.

Side-tilt stability is satisfying, but stability against horizontal movement is not, therefore I designed a stabilizing tooth in the footing bottom.

Consolidation settlement was also calculated for all the values. Load σ is 174.99 kPa and ground bearing capacity R will be subtracted from the charts after the geological survey.

Table 2. Soil parameters

soil name			clay		gravel
soil type			F8		G4
symbol			CH		GM
	mark	unit	min	max	
specific weight	γ	kN . m^{-3}	20	21	19.5
angle of internal friction	φ	°	15	19	28
cohesion	c	kPa	10	20	2
deformation modulus	E_{def}	MPa	4	6	60

9 Conclusions

Since 90´s of 20th century we notice increasing of total maximum daily precipitation. Analyses of hydrological and meteorological situations prove that floods on such catchments are caused by heavy rains with high intensity at upper part of a catchment.

Solution described in article reduces the culmination and protects the village against flood events. This polder is designed to retain flood with 100-year frequency. Channel arrangements under the dam construction keep the water from flowing out of the channel and prevent the creation of scours. Quarry stone fortification of the channel river bed prevents backward erosion and clogging the anti-fire retention dam, but it does not affect groundwater.

The polder is designed as a concrete structure, 5 m high, with an outlet structure in the bottom, which is 1 m wide and 0.66 m high and has got a trapezoidal spillway, which is 2 m wide and 1 m high. Unstable slopes are fortified with gabions [2], [7]. The polder will be made of natural materials; it will become an aesthetical part of the country. Total costs will be about 600 000 € (18 mil. SK).

The next phase of this project consists of a detailed topographical and geological survey, creating a DTM (digital terrain model), modelling of the flood situation in MIKE 11 and MIKE 21 software and drawing the flood maps. A flood map is a very good way, how to present the results to village governments and the public. Such solutions are often supported by European Union Funds.

During the planning, many problems can appear. Usually the biggest problem is purchasing detailed input data. Small streams such as Adamovsky Creek are not usually monitored, there are no information about n-year discharges or rainfall intensities, there is no topographical survey realized. It means that in the first phase of the project, we have to rely on uncertain sometimes even estimated values, which come into calculations.

We are always supposed to be on a safe side, even if it means bigger volumes of materials used. Because: "...if flood comes, there will always come a bigger one; that is the *Extreme value theory* based on experiences… " [6].

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A97**

Mathematical Model of Medzibodrozie WMS

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Abstract. This paper studies the problem of modeling hydrodynamic effects among interconnected river segments with an indirect flow control. Our ideas are applied to the problem of revitalizing River Tica, which requires guarantees on a controlled flow of water during the whole year. Mathematical modeling of relatively slow processes helps us to answer the question of how to transport water from River Latorica to River Velka Karcava on the border with Hungary using an indirect flow control through a series of sluice gates on River Tica.

Keywords: Mathematical modeling, water management system, natural channel, revitalization

1 Introduction

Mathematical model of the water management system (WMS) of River Tica was created as a part of the project INTERREG IIIA HU-SK-UA/05/01/041 (Fig. 1), which was supported by EU funds. Data in the model originate from parts of the project, which proposed a technical solution to the problem at the level of surface water [1] and described the impact of this solution on the regime of subsurface water levels [2].

Fig.1. The official logo of cross-boundary cooperation projects – INTERREG

1.1 Initial Ideas

At the beginning, we considered two basic variants of improving water discharge in the river channels of Tica and Velka Karcava on the boundary between Slovakia and Hungary:

a. improving water discharge in the connecting region using increased flood discharges from River Latorica, which are subsequently caught in the Tica River system, and later gradually transformed into an intake to River Velka Karcava
b. all-year improvement of water discharge in the WMS Tica from River Latorica, which requires the construction of a weir dam on River Latorica

In both cases, it is necessary to:

a. finalize the construction of the existing channel system, at least to a minimum extent (connecting gates – separation of existing channels, new channel segments)
b. build 6 (potentially reconstruct) stretches of channels and hydraulic objects on River Tica, which would allow for the control of water discharge

The first solution is similar to the one on the Hungarian side, where polder Czigand is filled during the time of flood discharges in River Tisza and gradually sluiced by the system of catch channels in the polder. The second solution, which is more suitable due to the character of the terrain on the Slovak side, guarantees regular improvements of water discharges in rivers Tica and Velka Karcava at the expense of one-time costs for building a weir lock hate. Note that this solution does not require the construction of locks for catching flood discharges on River Tica and increasing the volume of intake channels from River Latorica.

Due to aforementioned reasons, we focused on developing:

a. 1-dimensional grid model of the WMS Tica, which was expanded to 1.5-dimensional model of the main channel a inundations
b. balanced 2-dimensional model, which modeled the impact of spatial interactions of the 1.5-dimensional model due to percolation from the channel and evaporation of water compensated by rainfall
c. 2-dimensional hydrodynamic model for researching velocity fields and proposing the optimal modification of selected parts of River Tica to improve water discharge
d. Modification of models for dispatch control

2 1D Model of WMS

2.1 Mathematical Modeling of Unsteady Flow in Open Channels

The mathematical modeling of unsteady water flow in open channel is based on solution of 1D Saint-Venant solutions. Designation in solutions (1) and (2) corresponded with scheme of cross section on Fig. 2.

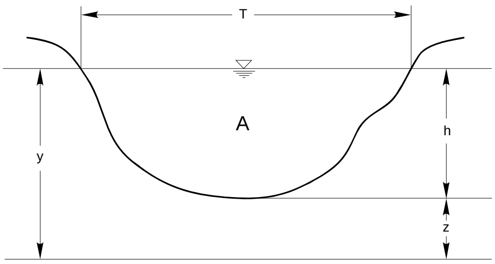

Fig. 2. Scheme of cross section

The continuity equation:

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} - q_{\ell} = 0 \tag{1}$$

where: *Q* water discharge in channel,
A cross section area,
q_l density of lateral intake (outtake),
x distance in direction of modeled channel,
t model time,
z level of bottom,
T width channel in water level.

The dynamical equation:

$$\frac{\partial(\rho\beta QV)}{\partial x} + \frac{\partial(\rho Q)}{\partial t} + gA\frac{\partial(\rho h)}{\partial x} = \rho gA(i_0 - i_e) + \rho q_{\ell} V_{\ell} \tag{2}$$

where: *V* average profile velocity of water,
g gravity constant,
ρ specific weight of water,
β correction factor due to non homogeneous velocity distribution,
h water depth in profile,
i_0 channel slope,
i_e energy line slope,
V_l part of lateral intake velocity in direction of axes x.

Correction factor due to non homogeneous distribution of velocity in profile is necessary introduced following transition from local velocities to average profile:

$$\beta = \frac{\iint_A v^2.dA}{V^2.A} = \frac{\sum_i A_i.v_i^2}{A.V^2} = \frac{\sum_i K_i.v_i}{K.V} \tag{3}$$

For numerical solution of equations was used Preismann implicit scheme based on finite element method. Starting water levels in modeled reach are obtain by modeling of permanent water flow in open channel.

For mathematical modeling of permanent water flow is useable method of computation from reach to reach and its corresponding relations between hydraulic characteristics. Parameters with index *d* are for downstream profile and with index *h* for upstream profile in modeling reach

$$\Delta z = Q^2 \left[\frac{1}{2g} \left[\left(\frac{\alpha_d}{S_d^2} - \frac{\alpha_h}{S_h^2} \right) + c. \left| \frac{\alpha_h}{S_h^2} - \frac{\alpha_d}{S_d^2} \right| \right] + \left(\frac{1}{K_d^2} + \frac{1}{K_h^2} \right) \frac{l}{2} \right] \tag{4}$$

where: Δz change of water level on reach $\Delta z = z_h - z_d$,
Q permanent water discharge,
α coefficient for non homogeneous velocity distribution,
c losses coefficient due to contraction or expansion of channel,
S cross section area,
l length of reach,
K conveyance of cross section.

2.2 2D Balanced Model of Environment Impact

Equation 1 enables to count in environment using variable of lateral intake density $\boldsymbol{q_l}$ [m^2/s]. Effect in equation 2 we can neglect with assumption $\boldsymbol{V_l = 0}$.

Infiltration losses

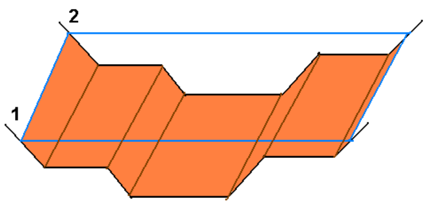

Fig. 3. Area of interaction between surface and ground waters

Water level in channel is in influence of ground water level. Impermeable background (IB) was considered in depth under terrain in interval 48-52 m. For computation there was used known equation for single sided infiltration from channel

$$q = -k \frac{h_1^2 - h_2^2}{L} \tag{5}$$

where: L range of depression groundwater level in (m),
h_1 water depth in channel measured from IB in (m),
h_2 ground water level measured from IB in (m),
k filtration coefficient, 0.0001 in (m/s).

Full losses on the discharge in computed reach due to infiltration are:

$$Q'_{ij} = 2\int_{x_i}^{x_j} q.dx = 2.\bar{q}_{ij}.(x_j - x_i) \qquad (6)$$

Evaporation losses and rainfall doping. Evaporation losses are directly proportional to area of free water surface S_{vh} and evaporation intensity from free water surface i_v :

$$q_v = -\frac{S_{vh}.i_v}{l}\frac{0.001}{86400} \qquad (7)$$

where: q_v density of lateral outflow due to evaporation in (m^2/s),
S_{vh} area of free water surface in (m^2),
i_v intensity of evaporation in (mm/day),
l distance between profiles in (m).

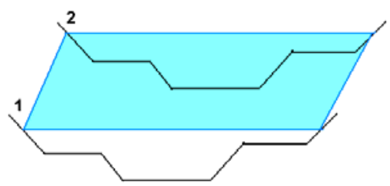

Fig.4. Area of evaporation losses and rainfall doping

Rainfall doping is directly proportional to free water surface S_{vh} and rainfall intensity i_z

$$q_z = \frac{S_{vh}.i_z}{l}\frac{0.001}{86400} \qquad (8)$$

where: q_v density of lateral inflow due to rainfall in (m^2/s),
S_{vh} area of free water surface in (m^2),
i_v rainfall intensity in (mm/day),
l distance between profiles in (m).

Full losses on water discharge in the reach are:

$$Q''_{ij} = \int_{x_i}^{x_j} b.(q_v + q_z)dx = (q_v + q_z)\frac{b_i + b_j}{2}\bar{q}_{ij}.(x_j - x_i) \qquad (9)$$

2.3 Setup of Net Model WMS Tice

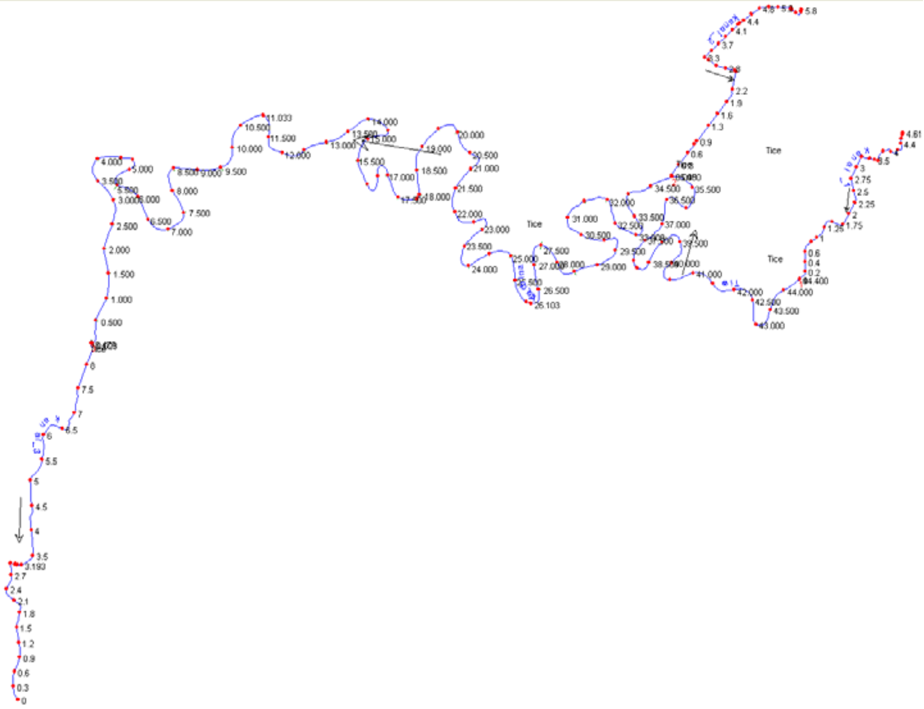

Fig.5. 1D net model

Table 1. Table of modeled reaches in Tica WMS

reach	channel	start at rkm	end at rkm	remark
1	channel 3	0.600	8.479	
2	Tice 12	0.023	4.760	
3	Tice 23	4.763	11.030	
4	Tice 34	11.033	17.910	
5	Tice 45	17.913	26.100	
6	Tice 56	26.103	38.170	
7	channel 2			inflow in 6-th reach
8	Tice 6K	38.173	44.400	join reach 8 a 9
9	channel 1	44.800	49.017	

1D net model consists from join reaches of Tice River system (see Table 1).

2.4 Discharge Control between Reaches in WMS Tica

As splitting objects and objects with discharge control were choose gates with total float area S=1,54 m^2 (2 circular tube with diameter 1000 mm). Total discharge *Q(t)* under submerged partial open outfall will be

$$Q(t) = \mu_v . S_e(t) . \sqrt{2g . H(t)} \tag{10}$$

where: μ_v outflow coefficient,
$S_e(t)$ effective outflow area changing in time,
$H(t)$ hydraulic gradient on object changing in time.

3 Modification of Models for Dispatch Control

This phase of solution was aimed to modification of mathematical model of WMS Tice:

a. Easy to use by operator WMS Tice, minimum quantity of input data, modification pre prepared operation scenarios – ***preprocessing*** (Fig. 6)
b. Maximum of in time presentation of action during manipulation with discharge on water management on objects this system (Fig. 7)
c. Start of modeling WMS at once – ***processing*** (complex process depend not only at inflows from River Latorica and outflow into Velka Karcava, but also at starting water level on individual reaches Tica River and open ratio of gates) (Fig. 7)
d. Possibility to graphic processing of results from modeling–***postprocessing*** (Fig. 8)

4 Conclusions

Described solution will allow us:

a. Optimize of water discharge control through revitalized part of Tica River
b. Modeling of emergency cases of breakdown discharges in Channel 1 and 2.
c. Modeling of extremely dry or wet periods

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Appendix

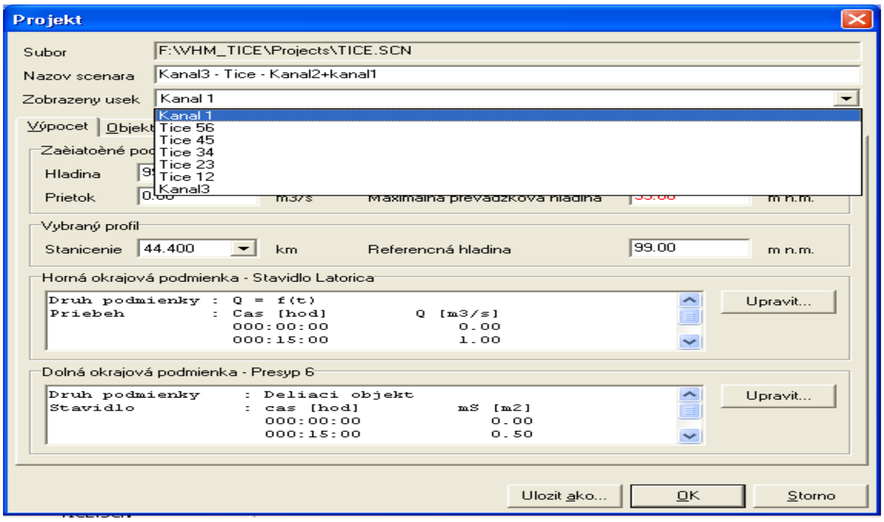

Fig. 6. Setup or modification of scenario

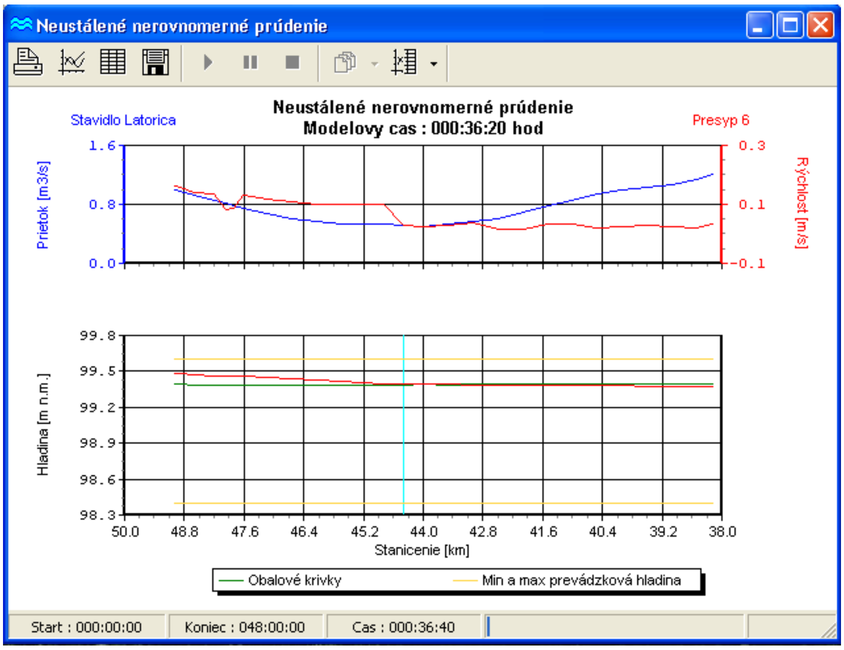

Fig. 7. Computing of water flow in WMS Tica

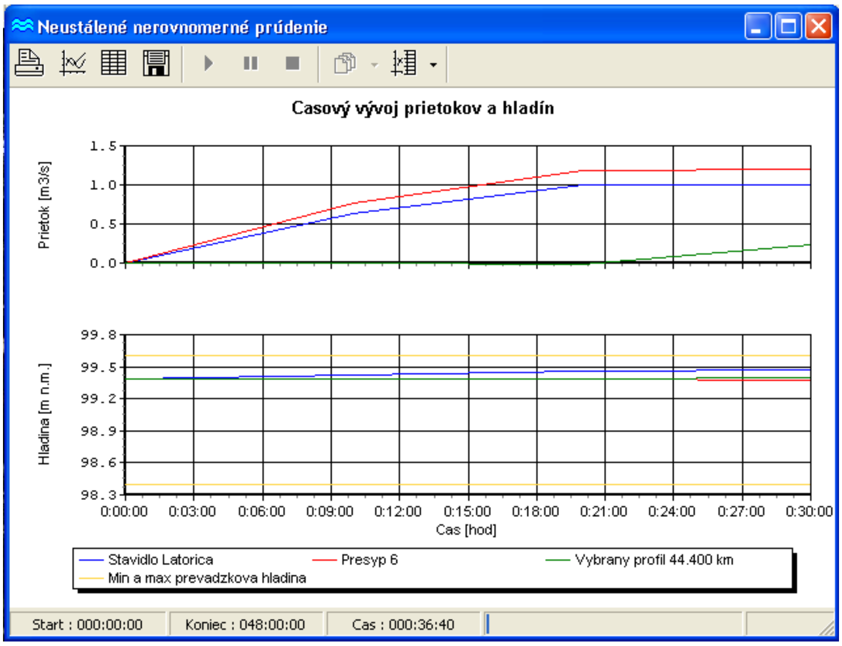

Fig. 8. Graphical processing of results from Tice WMS modeling

International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A98**

Predicting Breach Formation through Embankment Dam

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Abstract. The new large dam safety and failure impact assessment guidelines are being prepared for the Republic of Croatia. The breach formation process is complex, and depends upon a range of parameters. In this paper, the results of hydraulic analysis of Lipovačka Gradna and Jazbina (Croatia) dam failure have been presented. Special attention was paid to analyzing the first stage of breach growth when water flows through the initial breach in the dike crest and accelerates along the inner slope of the dike, eroding soil.

Key words: Dam failure, embankment dam, breach

1 Introduction

Flood induced by dam and levee breach happens occasionally throughout the world. Floods can induce serious loss of life and significant economic losses. To recognize the possible effects of dam breaks, a detailed knowledge of dam breakage processes and flood propagation is required. Warning time is the most important parameter affecting potential loss of life due to dam failure. Numerical and hydraulic models can be used to predict flood wave propagation and provide the information about the wave front arrival time, area to be flooded and water depth. Therefore, models are useful tool for developing evacuation plans and warning system for areas having potential flood risk.

When population centers are located close to dams, accurate prediction of breach parameters is crucial for development of effective emergency action plans. The development of a dam break is a complex process involving numerous uncertainties. Compared to the dam breach, the propagation of a flood wave can be modeled more accurately. However, the propagation of the dam break flood wave is a rare phenomenon and there is very little valid observation data available. Therefore, physical models are not only used to study the behavior of the prototype but also to produce necessary verification data for numerical models.

2 Development of a Breach

Embankment dams are subject to possible failure from either overtopping or piping water which erodes a breach through the dam. The breach formation is gradual with respect to time and width. As measured along the crest of the dam, breach is usually formed only along a portion of the dam's crest length. In many instances, the bottom of the breach progressively erodes downward until it reaches the bottom of the dam.

Understanding of the breach development process and the mathematical translation of this process into a model is of great importance for dike design. Simulation of erosion processes is based on relations between hydraulic energy dissipation rate and the erodibility factor. During the last decades, many methods have been developed to predict the hydrograph generated by a dam failure event [1] [2]. The methods are continuously improved and validated using the historical, laboratory and/or field experiment data.

Fig. 1. Formation of an opening in an embankment dam as a consequence of overtopping [3]

The size of the breach, as constituted by its depth and its width, and the rate of the breach formation determine the magnitude and shape of the resulting breach outflow hydrograph. This is of vital interest to engineers concerned with real-time forecasting or evacuation planning for floods produced by dam failures [4].

Recently developed models distinguish five stages in the breach developing process in sand-dikes [5] and clay-dikes [6]. In Stage I, flood water flows through the initial breach in the crest and starts the breach growth process by eroding soil away from the inner slope of the dike and possibly, depending on the flow velocity, along the dike crest. In this stage, water flows down and accelerates along the inner slope of the dike, getting more and more erosive. The breach flow attains its largest erosion potential in the vicinity of the toe of the dike. It can be expected that the dike breach erosion should often initiate close to the toe of the dike. Due to the larger erosion rate close to the toe of the dike compared to the upper part of the inner slope, the slope becomes steeper and steeper as time goes on. Stage I ends when a certain critical slope angle β_1 is achieved by the inner slope.

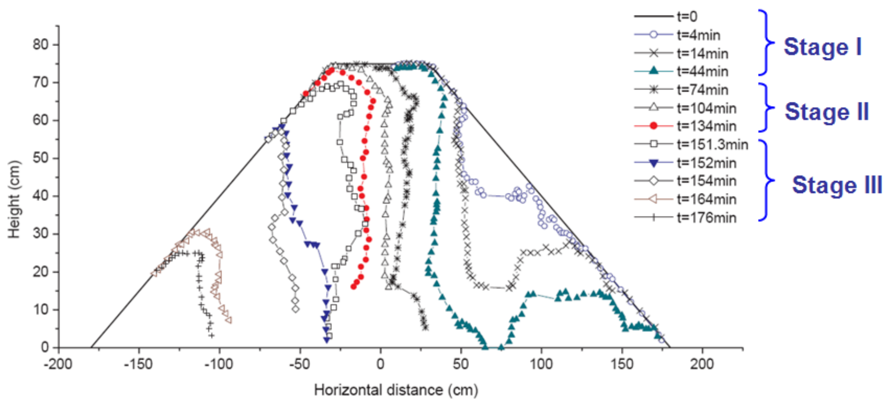

Fig. 2. Dike profile development [6])

Stage II begins after the inner slope of the dike arrives at the critical value β_1 (Fig. 2 and 3); the erosion of the crest is now intensive. Erosion of the outer slope occurs in Stage III. In Stages II and III, the dike body in the breach is eroded, until at the end of stage III the body of the dike has been completely washed away in the breach. In Stages IV and V the breach grows further, mainly laterally due to flow shear erosion along the side-slopes of the breach and the resulting discrete side-slope instability [6] .

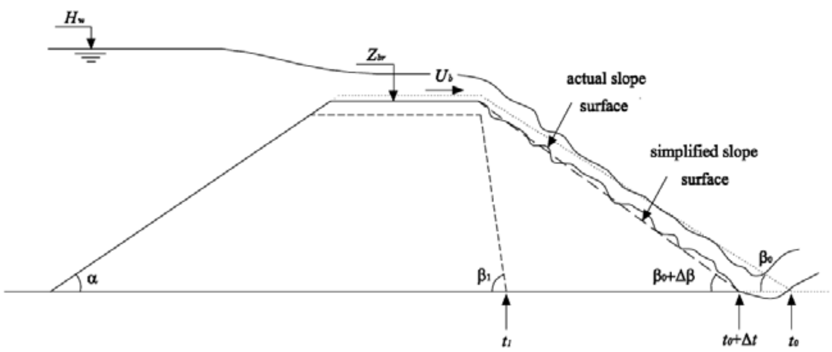

Fig. 3. Breach erosion in Stage I [6]

In the first Stage, it can be assumed that the breach acts as a broad-crested weir during the erosion process and therefore the discharge Q_{BR} through the breach is defined by the equation:

$$Q_{BR} = m\,B\sqrt{2g}\,h_p^{3/2} \tag{1}$$

where: m is the discharge coefficient, B is the width of the weir, and h_p is the depth of the stream over the crest.
The breach flow shear erosion occurs along the inner slope. Frequently used equation for the rate of soil erosion is:

$$E = M_e\,(\tau_b - \tau_c) \tag{2}$$

where: τ_b is bed shear stress, τ_c is critical shear stress and M_e is the erodibility factor. The bed shear stress is calculated by

$$\tau_b = \frac{1}{C^2}\rho g\, v^2 \tag{3}$$

in which C is the Chezy coefficient, ρ is water density and v is the cross-sectional average flow velocity.
Within one time step dt the increase of the slope angle $d\beta$ is calculated as:

$$d\beta = \frac{E}{x_e}\,dt \tag{4}$$

where x_e is the length at which normal flow occurs.

For modeling of breach growth in dikes, the key problem is the description of the rate of erosion of the dike by the flow. The crucial soil erodibility coefficient M_e used in existing erosion formulae is often stated as an experimentally or empirically determined constant [6].

In cases in which the breach flow velocity at the dike crest is not large and the erosion rate at the dike crest is small compared to that near the toe of the inner slope, the discharge through the breach (over the weir) depends strongly on the water level in the reservoir. That means that in Stage I there is no significant increase of discharge.

3 Case Study

A new numerical model was developed based on the new knowledge of modeling breach growth. The model was calibrated and validated with experimental data from laboratory experiments and used to predict the first stage of breach growth on Lipovačka Gradna and Jazbina Dam.

3.1 Lipovečka Gradna Dam

Lipovečka Gradna retention dam was built to protect the city of Samobor during a period of high water level. Following the initial project, the dam was planned to be built with tailings from a nearby quarry. As hydraulic data are not available for this type of material, data for a similar type of non-cohesive material were used for hydraulic simulation of the dam breach formation.

During the development of a model of breach formation in embankment dam, the fundamental problem is defining the intensity of erosion due to water flow velocity. The most important parameter is the erodibiliy factor M_e which is determined from experience or experimentally defined data. In this study, we used the Manning coefficient n = 0.03, critical shear stress τ_c= 1.5 N/m^2, and the erodifility factor M_e = 1*10^{-5} s-m/kg for the hydraulic estimation of the first phase of dam breach, as defined for similar material from previously published data (Zhu, 2006).

Results of the simulation show that the time required for erosion of the downstream portion of the dam is significantly larger than the time required for emptying the reservoir by outflow through the outlet and the spillway. That means that the progressive erosion will not follow in subsequent phases, and the dam breach formation will not occur.

It can be concluded that in the case when the volume of water in the retention basin is not large compared to the volume of the dam, it is not possible to reach the critical slope angle β_1 and the following Stages II and III. It means that the progressive erosion of the dam will cease.

3.2 Jazbina Dam

Jazbina retention dam was built to protect the west part of Zagreb during a period of high water level. The dam body volume is 279.723 m^3, storage volume to the spillway crest is 280.000 m^3, and the maximal volume that can occur in the retention is 530 000 m^3.

Figure 4 presents the cross-section of the Jazbina dam at the highest point, with the indication of downstream portion the dam washed away at the first stage of the dam breach formation. It can be calculated that more than 400 m^3 of the material needs to be washed away for the formation of the breach.

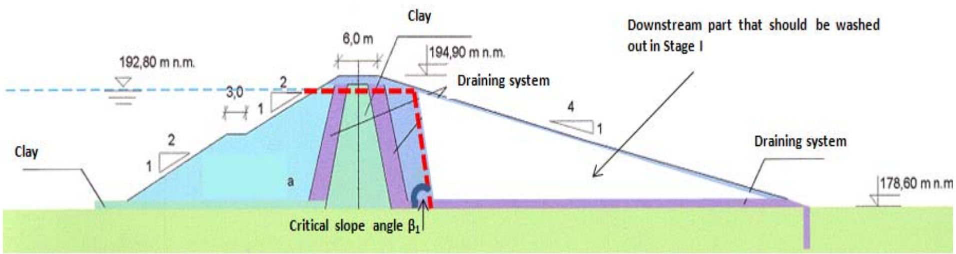

Fig. 4. Cross section of the Jazbina Dam indicationg the erosion zone at the first stage of erosion process

4 Conclusions

In this paper we present results of the analysis of possible dam failure of_Lipovečka gradna and Jazbina (Croatia) embankment dams.

Recently developed models distinguish five stages in the breach developing process in sand- and clay-dikes. In the Stage I water flows over the dike crest and accelerates along the inner slope of the dike, getting more and more erosive. In that stage the breach flow attains its largest erosion potential in the vicinity of the toe of the dike.

In case that the breach flow velocity at the dike crest is not large and the erosion rate at the dike crest is small compared to that near the toe of the inner slope, the discharge thought the breach (over the weir) depends strongly on the water level in the reservoir. That means that in the Stage I there is no significant increase of discharge.

Observed dams Jazbina and Lipovecka gradna are constructed to form retention basins. The volume of retained water is comparable in volume to the volume of the dam. Mathematical model of breach formation shows that it is not possible for the water in the retention basin to erode the downstream part of the dam. It means that the progressive erosion of the dam will not proceed past the Stage I.

The provided analysis shows that the embankment dams are exceptionally safe and acceptable (agreeable) solution for the construction of water retention structures in densely populated areas.

Considering that the available prototype as well as experimental data of dike failures (which are of high importance for model calibration) are scarce, we propose that the definition of specific hydraulic parameters is mandatory for the design of embankment dams in densely populated areas. Special attention should me made to investigate the erodibility factor M_e.

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A99**

Structural Design of Quay on River Drava near Osijek

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Abstract. This paper describes quay structure necessary for river transportation in Croatia. The 4×60m reinforced concrete quay on the river Drava at town Osijek has been designed. The quay structure will be constructed of in-situ made reinforced concrete. The quay will consist of four parts, three of them straight and one slightly curved. Each part will be 60 m long. The beginning of construction has been planned for the second half of this year (2009.). The reinforced concrete quay on river Drava near Osijek was analyzed with 102 different load combination in 2-D and 102 in 3-D model. All analysis was made according to the European norms. The analysis has confirmed that vertical and horizontal loading due to mobile cranes, ship impact forces and soil pressure are the most relevant for analysis of the structure. Prescribed quality of concrete was of type C30/37 and steel reinforcement of type B500B. Due to large horizontal forces of soil pressure on back wall, it was necessary to provide anchorage tendons at upper and at lower reinforced concrete beam grids combined with frictional plates at the back-wall of the structure.

Keywords: Drava River, quay structures, design, concrete, reinforcement, loading

1 Introduction

Ship landing stage in Osijek (Croatia) is placed on the right bank of river Drava and it is included in the system of the Danube international waterway. Inland waterway of river Drava is of IVth international class, as found in Review of the Classification of European Inland Waterways [9]. In the future, this part of, river Drava will be of Vb international class. Due to future development, all structures should be designed for that state (Vb).

2 Concrete Structure of Ship Landing Stage

2.1 General Information

The quay structure will consist of four parts, three of them straight and one slightly curved. Three straight parts of the quay structure are 60.0 m long and 10.5 m wide. Curved part has radius of 500 m. This curved part of quay structure consists of three straight segments, each 20.0 m long with 178^0 angle between two adjacent parts.

Working plateau will be at level of +87.5 m. The quay reinforced concrete structure will be 7.5 m height, and supported by longitudinal geotechnical diaphragm and drilled piles on a 5.0 m distance in longitudinal direction of a structure. Each of four parts of the structure could be marked with three longitudinal axes, marked as A (along the river Drava), B (in the middle of the structure), C (along the shore), and thirteen transversal axes, marked by numbers 1 to 13. The distance between the longitudinal axes is 5.25 m, and between transversal axes is 5.0 m.

The characteristic cross-sections of reinforced concrete structure with ships at extreme water levels are shown in figures 1, 2 and 3.

Several books and papers have been of great use for design of quay structure. These are: Bruun [2], Thoresen [3], Quinn [4], Haefen und Wasserstrassen [5], Kolund and Despot [6], Mazurkiewicz [7].

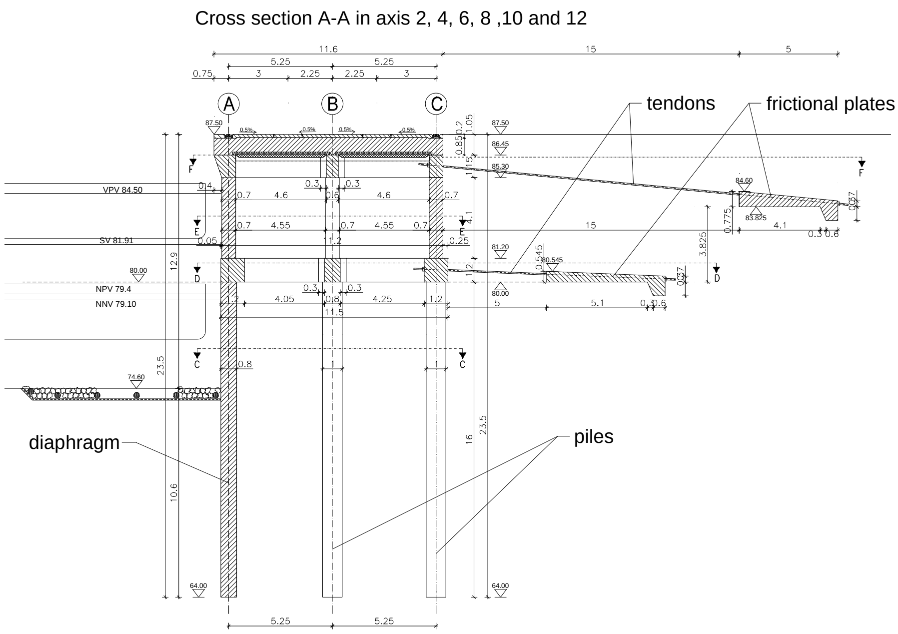

Fig. 1. The transversal cross section A-A of reinforced concrete quay structure, at the position of middle column (cross section position presented in Fig. 3)

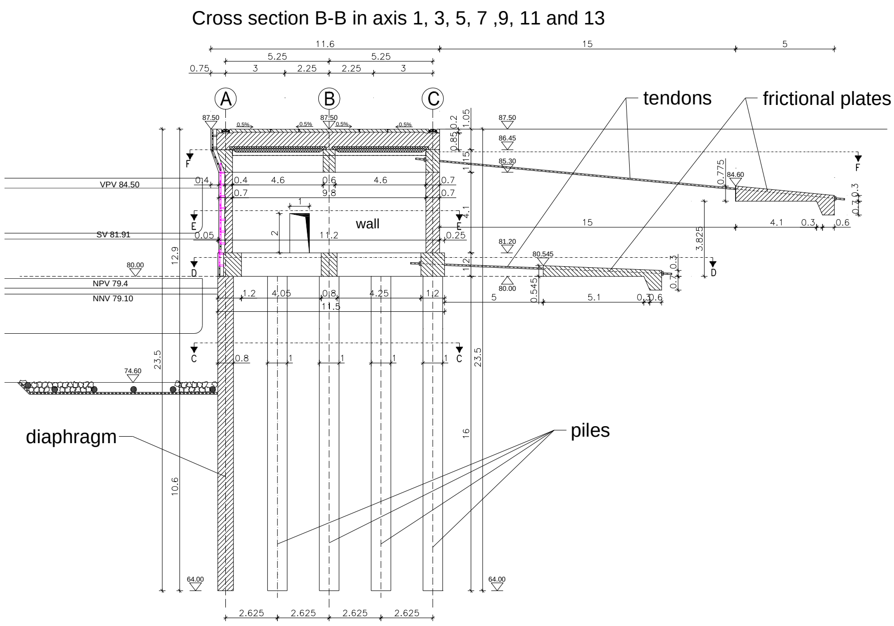

Fig. 2. The cross section B-B of reinforced concrete quay structure at the position of transversal wall

2.2 Concrete Structure

The structure above geotechnical piles and diaphragm will be constructed in situ. Only Π-plates, which serve as a lost formwork for a load-bearing plate, will be prefabricated. Figures 1, 2 and 3 show cross sections of the structure with element's dimensions. There would be altogether 13612.3 m^3 of concrete and 2025105 kg of reinforcement built in the structure. The concrete type shall be C30/37, and steel reinforcement B500B. Concrete cover will be at least 4 cm for elements that are not in contact with soil. For those in direct, unprotected contact with soil, cover will be 7 cm. First, the geotechnical diaphragm will be constructed in the axis (A), along the entire length of river port. Its thickness is 80 cm and depth of 16.0 m that is from level +64.00 m (in soil) to level +80.00 m. At every 5.0 m (longitudinally), from the axis (1) (edge of the structure), the reinforced concrete piles with diameter 100 cm will be drilled. In the axes (1), (3), (5), (7), (9), (11) and (13), 4 piles will be spaced 2.625 m apart, while in the axes (2), (4), (6), (8), (10) and (12), 2 piles will be spaced of 5.25 m apart. Piles will be drilled 16.0 m deep, from level +80.00 m to level +64.00 m. After finishing the reinforced concrete supporting geotechnical structure elements (diaphragm and piles) the main reinforced quay structure will be constructed. First, the lower grid structure, which is consisted of girders that connect the piles and diaphragm, will be constructed. These girders make certain network (grid) with axes distances 5.0×5.25 m. The dimensions of girders in the axes (A) and (C) are b/h = 120/120 cm, while all other girders are of dimensions b/h = 100/120 cm.

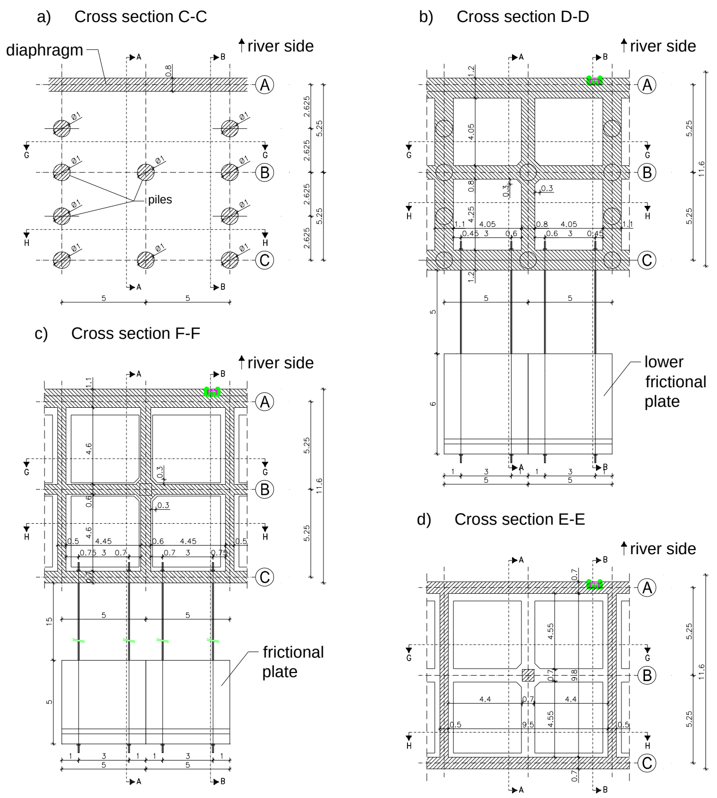

Fig. 3. The cross section of reinforced concrete quay structure: a) positions of diaphragm and piles, b) positions of girders above diaphragm and piles, c) position of girders below upper deck, d) longitudinal and transverse walls and middle column

After the construction of lower grid girders, the two 70 cm thick reinforced concrete walls will be constructed. These walls are placed along a river side (above the diaphragm and girder) in the axis (A) and in the coastal side (above the piles and girder) in the axis (C) between level +81.20 m and +85.30 m. In the wall by the coastal side at level +81.60 m and at level +85.90 m, prestressed tendons along with frictional plates will be applied in order to support the soil pressure on the back wall. In each field, there will be four tendons connected to the back wall and frictional plate (see figures 1, 2, and 3). Tendons have the capacity of 400 kN. In the transverse direction, above the transverse girders in the lower grid axes (1), (3), (5), (7), (9), (11) and (13), the 50 cm tick reinforced concrete walls will be constructed. These walls will be placed every 10.0 m apart between level +81.20 m and level +85.30 m. They have openings 100×200 cm between the axes (A) and (B). In the middle of a field which is defined by transverse and longitudinal walls (at the joint of transverse and longitudinal lower grid middle girders in axis (B)) the 70×70 cm reinforced concrete columns, from level +81.20 m to level +85.30 m will be constructed. After finishing of columns and walls, the construction of upper girders grid will start. These girders are longitudinal in axes (A), (B) and (C), and transversal in every axis (1) to (13) above transverse walls and above columns.

Prefabricated Π-plates, which serve as a lost formwork for a load-bearing plate is set so that in one field, rely on the wall in axis (A) and the beam in axis (B) in the second field on the beam in axis (B) and on the wall in the axis (C). Π-plates are also connected with bearing plate. Bearing plate will be cast up to level +87.30 m with the abrasive layer of concrete which thickness varies between 15 and 20 cm.

3 Design of Quay Structure

The reinforced concrete quay structure was analyzed in 2-D and 3-D models. The 2-D and 3-D models of analysis that took into account all combinations of loads, settlement of soil and the temperature effects have been used for calculation of forces and stresses in concrete quay structure. 102 different load combination in 2-D and 102 in 3-D model were calculated. The main kinds of different loading and loading conditions that act on the reinforced concrete structure of port stage are sustained, variable, additional and accidental loading.

3.1 General Sustained Loading

General sustained loading of quay structure are: structure self-weight, hydrostatic pressure (that depends on water level), soil pressure, uniform live load (2 rows of container, with loading of 35.0 kN/m^2), railway crane and/or mobile crane, railway loading (two tracks), road loading on quay, snow, ice.

3.2 General Variable Loading

General variable loading of quay structure are: additional soil pressure from uniform live load on embankment, temperature, braking and accelerating of railway crane, impact of railway crane on buffer, angular deviation of railway crane, braking and accelerating of mobile crane, braking and accelerating of railway trains, impact of railway trains on buffer, angular deviation of railway, braking and accelerating of road vehicle, operating wind (v=20 m/s) on railway crane in use (perpendicular to quay length or along the quay), operating wind (v=20 m/s) on mobile crane in use (perpendicular to quay length or along the quay), operating wind (v=20 m/s) on railway (perpendicular to quay length), ship impact by landing perpendicular to quay length: 100 kN, ship impact by landing along to quay length: 0.5×100=50 kN, rope pull force perpendicular to quay: 100 kN, rope pull force along the quay: 4×100 = 400 kN, vertical force due to friction of ship toward quay, settlement of supports.

3.3 Additional Loading

Additional loading on a quay structure are additional soil pressure from mobile crane on embankment, additional soil pressure from road load on embankment, concrete shrinkage and temperature.

3.4 Accidental Loading

Accidental loading of quay structure are: hurricane (40 to 50 m/s) on railway crane not in use (perpendicular to quay length or along to quay length), hurricane (40 to 50 m/s) on mobile crane not in use (perpendicular to quay length or along to quay length), ship impact by landing (v_n= 0.2 to 0.3 m/s), hydrostatic pressure, soil pressure, seismic loading.

3.5 Calculations and Results

Of all loading that act on top surface of structure at level +87,5 m the mobile crane loading seems to be the most important, because of its weight and size. Besides, the mobile crane can move in all possible unfavorable places for structural elements. Mobile crane has four axles on wheels and four extensible legs. Distances from center to center of legs are 11.50 m and 10.00 m. The size of each leg is 1.20 x 1.80 m. Its total weight in work is 2500 kN. Two mobile cranes are foreseen to be in use for loading and unloading in port. The maximum load on one leg could be as much as 1434 kN which means the pressure of 287 kN/m^2 on reinforced concrete slab below each leg. This kind of load is way above any other load and therefore is relevant for design. The examples of the results calculations of reinforcement for upper slab are shown in figure 4. The figure 5 shows reinforcement in upper girders. Figure 6 shows reinforcement in front wall. In the figures 4 and 5 only 35.0 m of structure is shown, while in figure 6 covers 20.0 m of structure in order to show the most stressed elements. There was no difference in analysis of one curved and three straight parts of the quay.

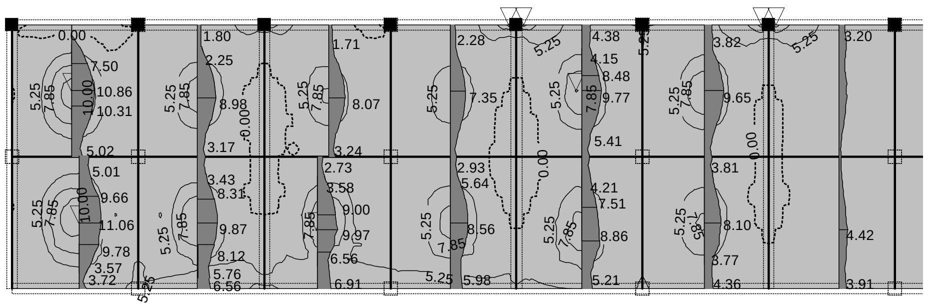

Fig. 4. Required reinforcement in upper slab in longitudinal direction

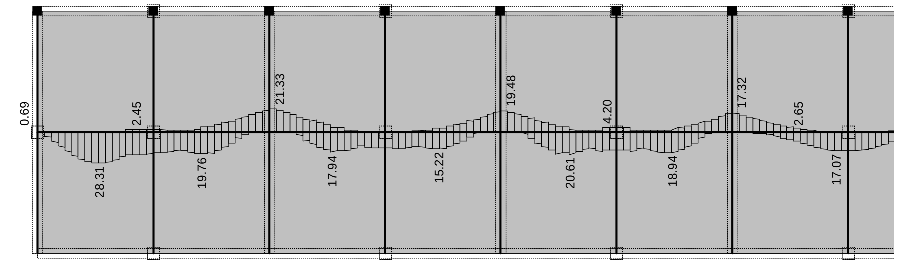

Fig. 5. Required reinforcement in longitudinal upper girders

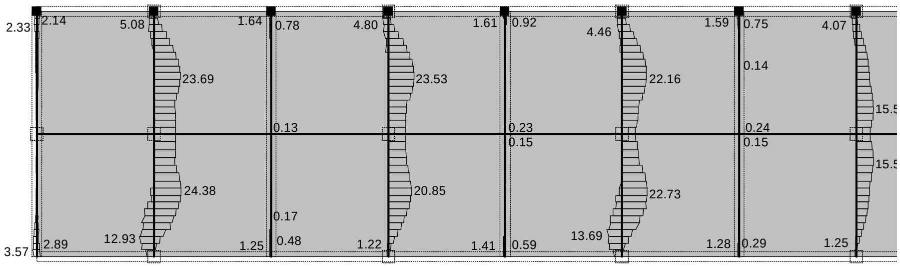

Fig. 6. Required reinforcement in transversal upper girders

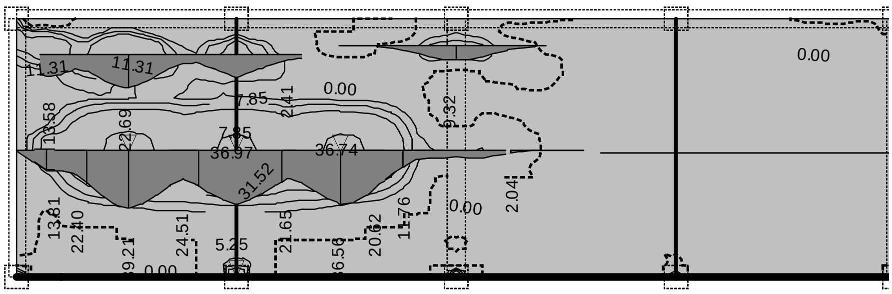

Fig. 7. Required vertical back side reinforcement in front wall – axis (A)

4 Conclusions

The 4×60m reinforced concrete quay on the river Drava at town Osijek has been designed and analyzed. This quay structure will be constructed of in-situ made reinforced concrete. Due to durability reasons the structure elements will be of simple shape and robust. The materials that shall be used in construction the quay structure are concrete and steel reinforcement. Concrete will be of class C30/37 and reinforcement of grade B500B (ribbed) bars.

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A100**

Experimental and Numerical Approach to Surge Tank Improvements

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Abstract. World demand for energy is expected to grow in the future. According to hydraulic systems will be optimized in order to increase the energy production efficiency. In the light of the general energy shortage pumped storage has come to be the environmentally most compatible method of balancing sudden drops in demand or unexpected production increase from wind power stations. The contribution introduces the numerical and experimental methods used for observation of form losses in the part of surge tank - hydraulic pipe junction with nozzle. The measurements of form losses will be done at different Reynolds number. In the continuation the measurements at more and higher Reynolds number were used for extrapolation of form losses to the nature dimensions. On the basis of the extrapolation with physical model the simulation with numerical tools (CFD) was performed. In this way further adaptation of change geometries of hydraulic system is possible.

Keywords: Surge tank, numerical model, experimental approach, form losses, operating regime

1 Introduction

In the optimization and construction of surge tanks new solutions are introduced and before their implementation, detailed analyses should be done.

At the Institute for water structures and water resources management, at Graz University of Technology, several research projects were done for different water pumped storage systems (Kops II, Limberg II).

Present contribution introduces the numerical and experimental process used for flow losses estimation of the nozzle. This nozzle is positioned close to lower chamber in the connection pipe between lower and upper chamber of the surge tank (Fig. 1)

The basic idea of research work was to implement the modern, computer based, simulation tools (CFD) and to combine them to findings of the model tests performed in the laboratory in the first phase. In the continuation, the conclusions were used for extrapolation of results to the nature dimensions with higher Reynolds numbers at the nozzle cross-section in order to assure the safe operation regimes.

2 The Surge Tank Construction at the Limberg II

Pump storage power plant Limberg II, which is at the construction now, will use 380 m of height between existing accumulations Mooserboden and Wasserfallboden. New plant will operate in parallel to pump storage Kaprun-Oberstufe. The expected power for both machines is 480 MW and this increase the total plant power from 353 MW up to the 833 MW at the turbine operating regime and from 130 MW up to 610 MW at the pump operating regime respectively.

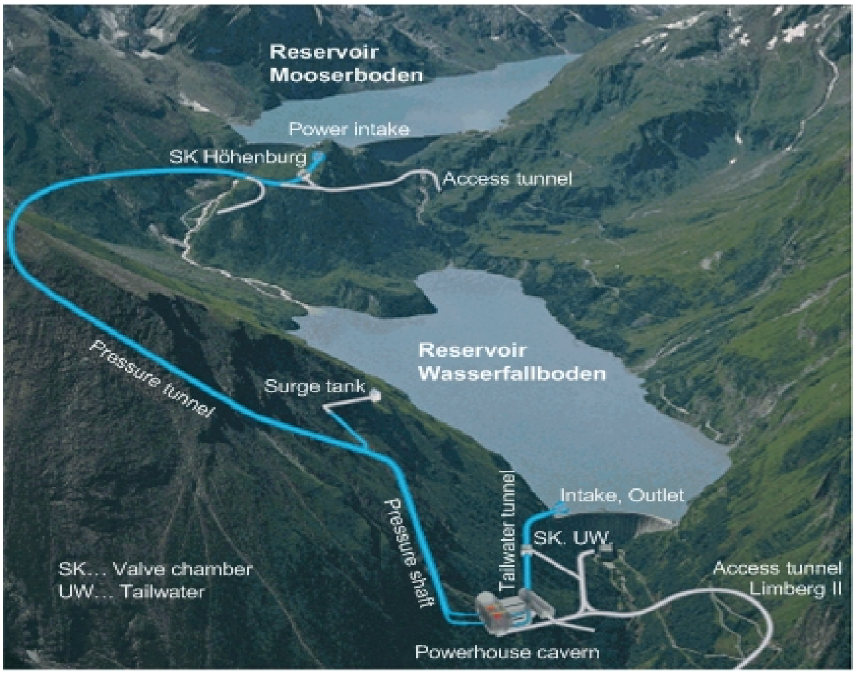

Fig. 1. Situation in the landscape

The surge tank consist of two chamber (upper and lower chamber) and a pipe junction with nozzle (Fig. 2) The advantage of introduced junction system is in flow passing chamber integrated in the water tunnel.

The one-armed upper chamber was already defined regarding its position and height, which resulted in its length of ca. 200 m. An additional parameter was the configuration regarding location and direction of the diagonal pressure pipe in the continuation of the armored pressure tunnel. Another problem is the surge waves caused by the increasing rapidity of the mode changes between the pumping and generation modes. This new situation is being met by improvements in surge-tank strategy.

In order to enable aeration of the chamber (to prevent cavitations) for the operational purposes, an additional ventilation construction with in-between access from chamber closure to upstream pipe was planned.

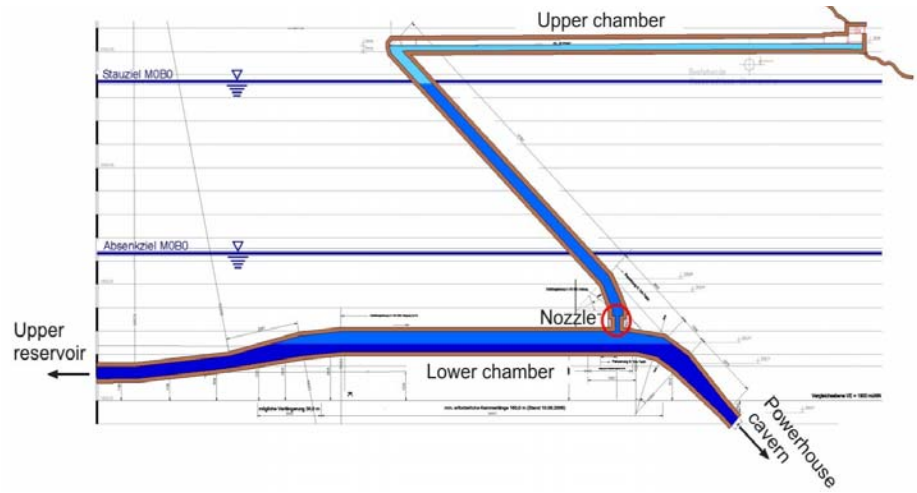

Fig. 2. Surge tank cross section

3 Form Losses Estimation

The pressure losses that occur in hydraulic systems and pipelines due to bends, elbows, joints, valves, etc. are often defined as form (vorticity) losses. In many cases these losses are larger than the losses due to pipe friction. For all form losses in turbulent flow, the pressure loss varies as the square of the velocity:

$$p_{loss} = \rho \cdot \zeta \cdot \frac{v^2}{2} \tag{1}$$

Thus a convenient method of expressing the form losses in flow is by means of a loss coefficient (ζ). Values of the loss coefficient (ζ) for typical situations and fittings could be found in literature, while for the special flow segments detailed analyses should be done.

Present chapter presents two methods for form loss coefficient estimation at the pipe junction nozzle for described surge tank.

3.1 Experimental Approach

The pipe junction with nozzle was analyzed on physical model with scale 1:21,8 shown in Fig. 3.

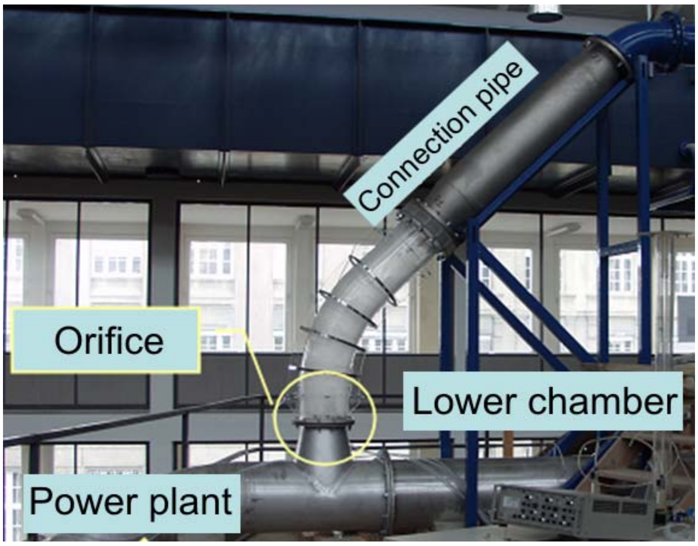

Fig. 3. Pipe junction nozzle model

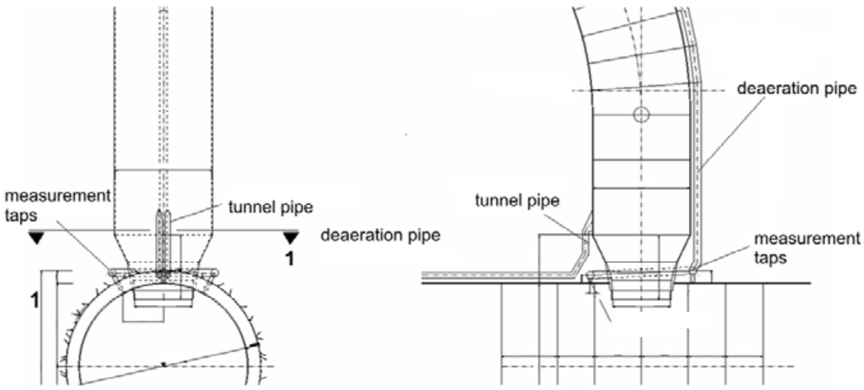

Fig. 4. Pipe junction nozzle detail

The nozzle, its position and flow losses were checked through a hybrid approach; initially through 3D–numerical simulation and later through a physical model experiment in scale 1:21,8.

3.2 Numerical Approach

Owing to fast computer capabilities increase and according to intensive numerical methods development, computational optimization of hydraulic systems became possible in the past. In last years CFD also matured as useful tool in the search for optimal solutions regarding flow behavior at different operating regimes.

This chapter presents the approach to numerical simulation of water flow through pipe junction, at different scales – from model size (s=1) up to the prototype (nature) dimension (s=21,8). Analyses scales range indicates the major CFD advantage comparing to expensive model construction since different scales and Reynolds numbers could be analyzed simply and fast.

Computational mesh and boundary conditions. For presented simulation, unstructured tetrahedral mesh was used. At the junction inlet, static pressure was prescribed and opening boundary condition with mass flow was defined at the pipe junction exit boundary.

Mathematical model. The conventional Reynolds averaged Navier Stokes equation system (RANS) in form of continuity equation, momentum equations and k-e turbulence model was applied for flow simulation. The Ansys–CFX software was used.

Results. The computational fluid dynamics results for the pump operating regime are shown in Figs. 5–8. Research on the physical model confirmed qualitative results of a numeric calculation**.**

Model **Prototype**

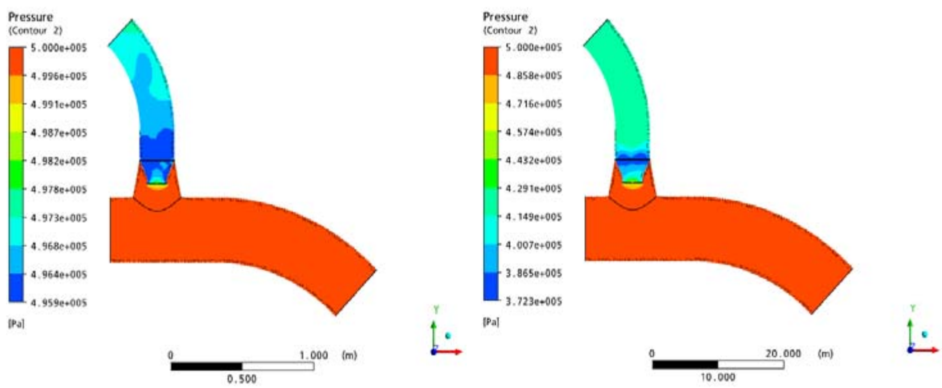

Fig. 5. Pressure distribution for pump operating regime

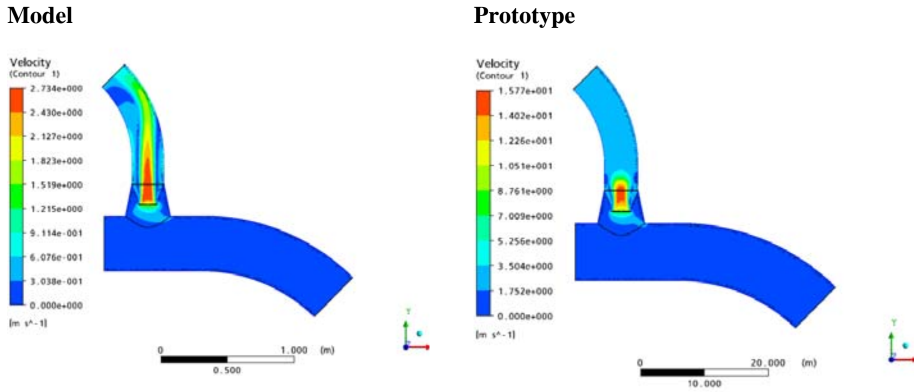

Fig. 6. Velocity distribution for pump operating regime

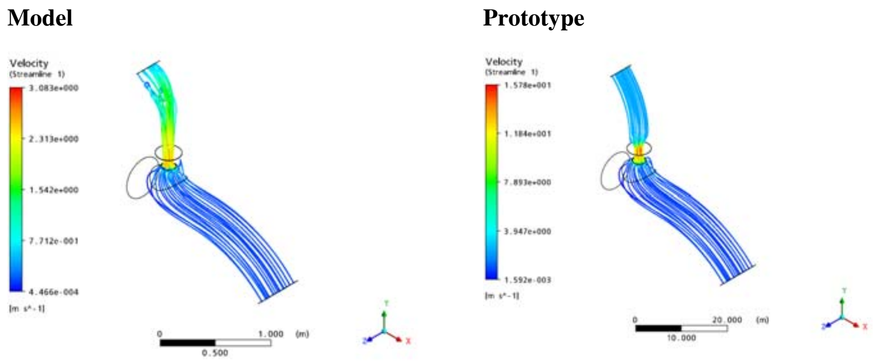

Fig. 7. Streamline plot for pump operating regime

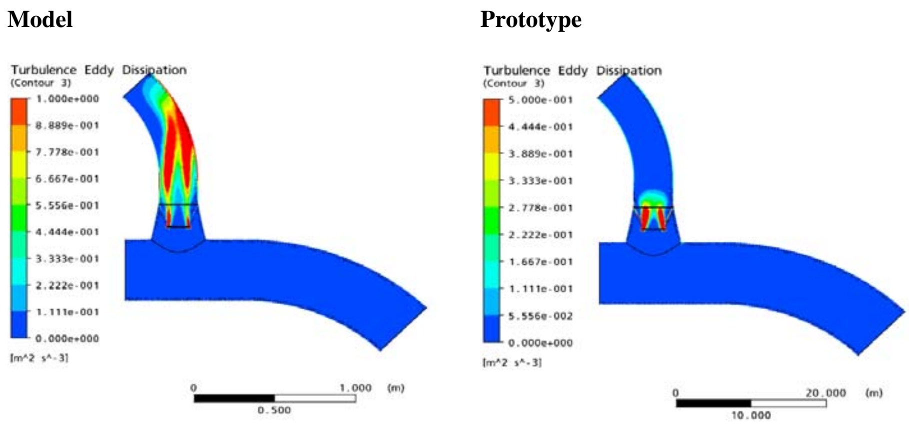

Fig. 8. Turbulence eddy dissipation plot for pump operating regime

Model **Prototype**

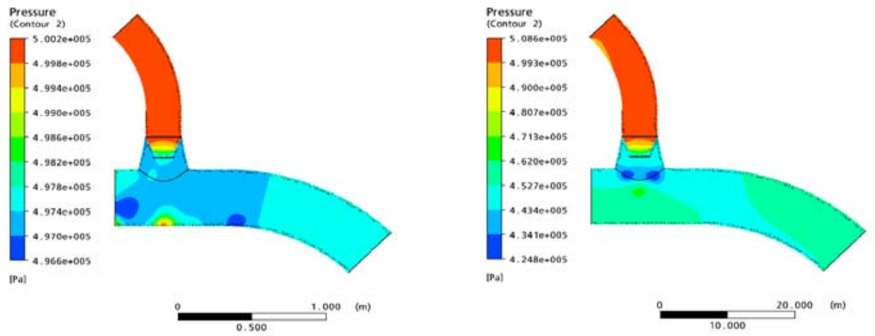

Fig. 9. Pressure distribution for turbine operating regime

Model **Prototype**

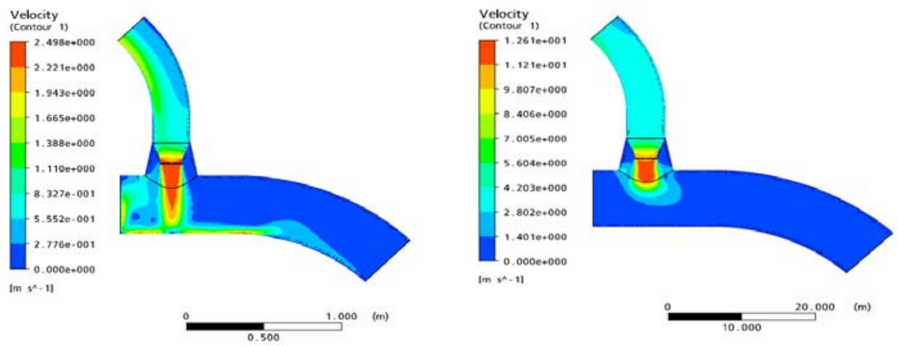

Fig. 10. Velocity distribution for turbine operating regime

Model **Prototype**

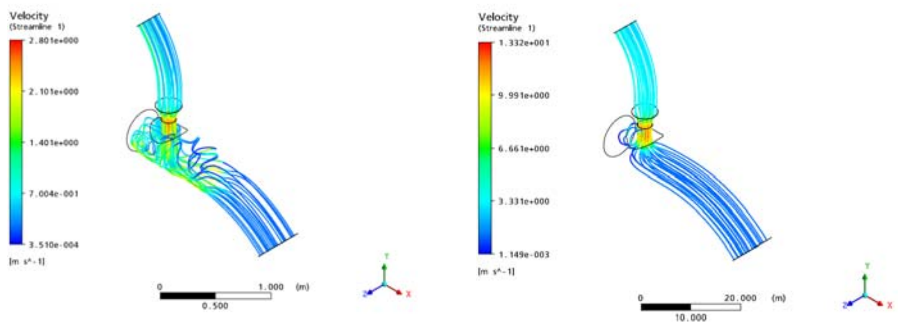

Fig. 11. Streamline plot for turbine operating regime

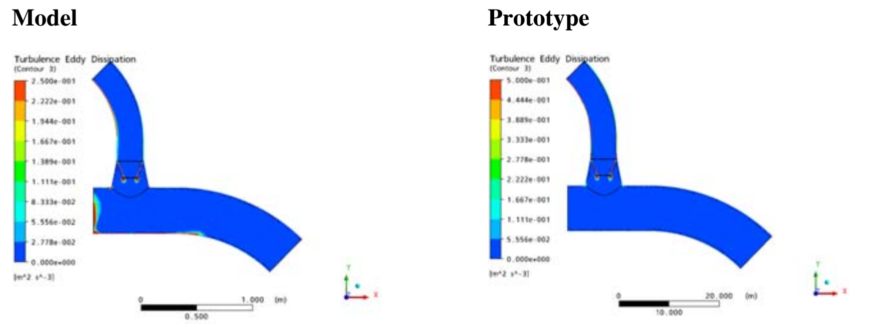

Fig. 12. Turbulence eddy dissipation plot for turbine operating regime

The results comparison for prototype and model scale in the turbine operating regime confirms the findings from the pump operating regime (Figs. 9–12). The flow pattern distinction is clearly evident at streamline plots shown in Fig. 11.

4 Form Losses Coefficient Comparison

According to the relative good agreement between results at scale s=21,8; CFD results for different scales was transposed to dependency flow losses - Reynolds number values. The comparison for variety of operating regimes is shown below.

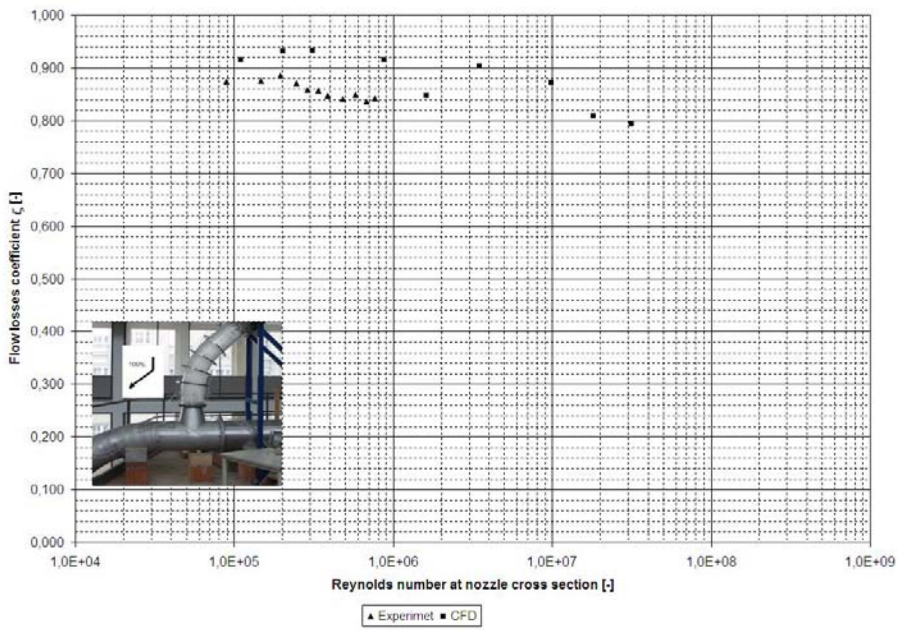

Fig. 13. Results comparison for turbine operating regime

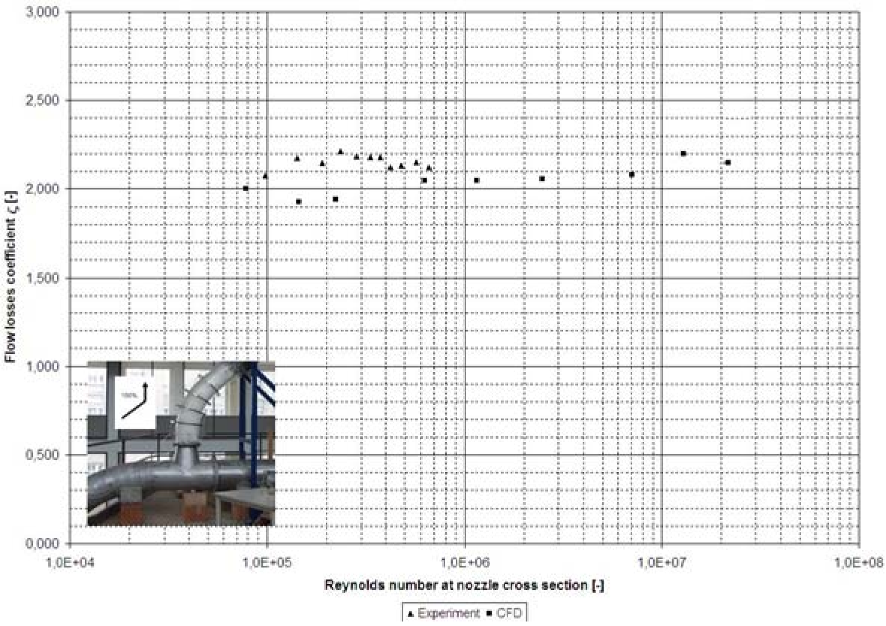

Fig. 14. Results comparison for pump operating regime

It is evident from Figs. 13 and 14, that flow losses coefficient follows the experimental results at the area of lower Reynolds number values. According to this, CFD predictions of coefficients at Reynolds number values higher than 10e6 should be assumed as correct.

5 Conclusions

The present work demonstrates the possibilities and difficulties of using CFD in the process of hydraulic machinery optimization and form losses coefficient determination.

The numerical and experimental results for different geometry scales presented in manuscript show reasonable agreement.

Both results show similar form losses coefficient distribution along different Reynolds number values.

Results show that RANS numerical model with conventional closure model – transport equations for turbulent kinetic energy and turbulence kinetic energy dissipation, should be used for determination of phenomena at the integral point of view.

The computational fluid mechanics results confirm the experimental values with average results deviation up to 15%.

It can be written, that used conventional RANS model presents suitable tool for flow losses coefficient calculation, for prediction of flow phenomena and as fast method for geometry optimization despite to the mismatch comparing to physics (empirical models).

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A101**

Influence of Pumped-Storage Power Plant on Physical and Ecological State of Lake Żarnowiec

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Abstract In 1983 the largest pumped-storage power plant (716 MW) in Poland was put into operation. It uses Lake Żarnowiec as the lower reservoir. At the design stage of the power plant and after 10 and 20 years of its operation complex investigations were carried out. In the paper results of these studies are presented together with the description of the lake and its catchment. Investigations of the lake for design purposes included water quality, water balance, currents, water temperature, chemical state of the lake water, fishery state and hydrobiology. Investigations after ten years of the operation of the pumped-storage power plant concerned the same spheres. These investigations indicate that important changes in thermal regime currents and shoreline state are observed. Water chemistry changed mainly due to the development and the use of lake catchment. No substantial changes in the hydrobiological state of the lake were observed.

Keywords: Water resources, Lake Żarnowiec, pumped-storage power plant, water quality, ecological changes

1 Introduction

In the seventies it was planed to construct pumped-storage power plant and nuclear power plant on the shores of the Lake Żarnowiec. The operation of both projects would result in considerable hydrodynamic, thermal and environmental changes in the lake, therefore comprehensive studies of the lake were undertaken. They included hydrology, hydrodynamics, thermal regime, hydrochemistry, hydrobiology, ichthyology, and meteorology of the lake region. These studies started in 1973 and were continued till 1990.

In May 1983 the pumped-storage power plant was put into operation, and in 1990 it has been decided to abandon the construction of nuclear power plant.

In 1993, after 10 years of the operation of pumped-storage power plant, it has been decided to carry out another comprehensive study which had to indicate how the operation of the power plant influenced the ecological state of the lake, and what changes of the lake are caused by the power plant and what is due to changed use of the land of the catchment.

This study included: changes in the shoreline, lake hydrodynamics and thermal regime, water quality assessment of the lake and its susceptibility to degradation, environmental and trophic conditions of the lake, hydrobiological conditions (phyto- and zooplankton, zoobenthos, primary productivity), and fishery state of the lake (resources and communities). Studies, in their predominant part, were based on the direct measurements in the lake as well as on the laboratory analysis of the samples taken from the lake. Studies were carried out by interdisciplinary team from Technical University of Gdańsk (Faculty of Environmental Engineering), Institute of Hydroengineering of the Polish Academy of Sciences in Gdańsk, and the Institute of Inland Fishery in Olsztyn.

In 2004 some additional measurements and studies were carried out for the purposes of the dikes of the lake and along its outlet - Lower Piaśnica River.

In the paper results of these studies are presented together with the assessment of pumped-storage power plant on the physical and ecological state of the lake.

2 Lake Żarnowiec

Lake Żarnowiec, of glacial origin, is situated in the northern part of Gdańsk Province. It is one of the largest and most beautiful lakes in this region and it plays important role in creating the landscape. The lake has very good water quality and is used intensively for recreation and fishery. Therefore preservation of water quality and ecological values is of prime importance. The lake belongs to the catchment of Piaśnica River. Upper Piaśnica River flows into the lake and flows out of the lake through the control weir as the Lower Piaśnica. Catchment amounts to 310 km^2 and is divided into 4 parts: Upper Piaśnica River 16.7 km long with the catchment of 88 km^2, Bychowska Struga River 20.1 km long with the catchment of 122 km^2, Lower Piaśnica 5 km long with the catchment of 61 km^2, and the direct lake catchment of 39 km^2. Lower Piaśnica discharges into Baltic Sea [1].

In natural conditions at average water elevation 1.34 m above sea level the lake had the surface area of 14 km^2 and a total volume of 118 million m^3. The length of the lake is 7.6 km and the maximum width 2.6 km (Fig. 1). The maximum depth is 19.4 m and the average depth is 8.4 m. Minimum observed water surface elevation was 0.92 m and maximum 1.78 m. Daily water level variations were in the range 1 to 2 cm, and in extreme situations they reached 5 to 7 cm. The bottom of the lake consists of sand and gravel, and in some areas is covered with organic sediments [7].

Surface inflow to the lake comes from the direct catchment and 2 small tributaries (Upper Piaśnica and Bychowska Struga River). The average surface inflow to the lake is small and amounts to 1.7 m^3/s and the groundwater inflow was estimated as

0.6 m^3/s. This inflow is very uniform during the year because of the existence of several small lakes and marches.

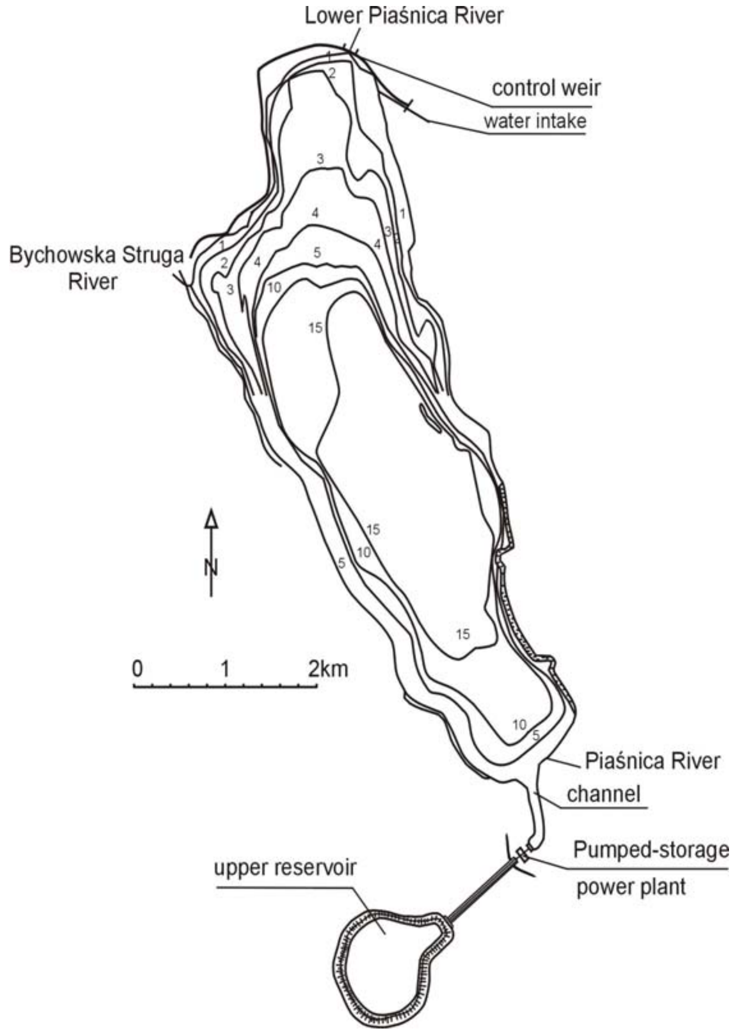

Fig. 1. Lake Żarnowiec and the layout of pumped-storage power plant

Since 1983 the Lake Żarnowiec forms lower reservoir of the pumped-storage power plant. The water surface elevations in the lake (operation of pumped-storage power plant) vary daily about 1.0 m. During the year water elevations range between 0.70 and 2.00 m. Minimum water elevation appears during morning hours (when upper reservoir is full) and maximum - late in the evening when upper reservoir is empty. In the past outflow from the lake was a natural one. Now with significant daily variation of water surface elevation the weir which controls the outflow from the lake has been constructed. The minimum biological outflow was determined as 0.3 m^3/s [5].

During vegetation season water is withdrawn from the lake by means of a pump station for irrigation purposes (Fig. 1).

3 Management of the Lake Catchment

Lake catchment consists of agricultural land, meadows, forests, and small amount of wasteland. In the catchment of Piaśnica River prevail forests while in the catchment of Bychowska Struga River agricultural land. Lower Piaśnica (Piaśnica Dolna) catchment consists primarily of meadows which are under special protection. The hydraulic engineering management of two main inflows to the lake is rather poor and to a large degree neglected.

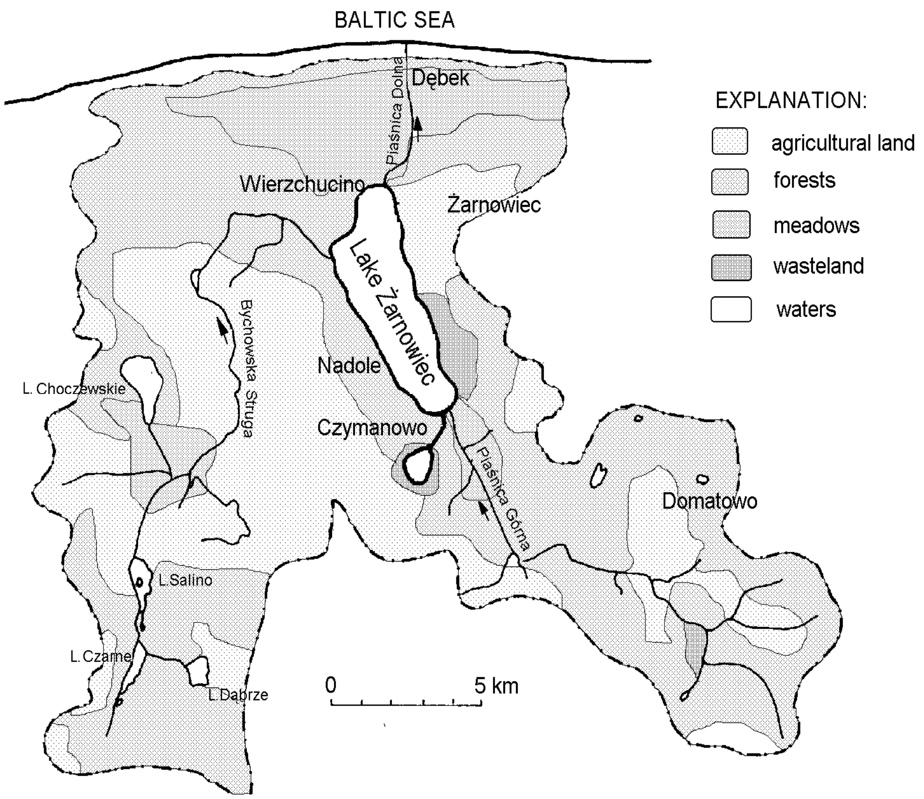

Fig. 2. Management of Lake Żarnowiec catchment [3]

During recent years important changes in the use and management of the catchment were observed. New sewage treatment plant at the western shore of the lake was constructed. Purified water is discharged into the lake by a bottom outlet situated about 100 m from the shore. Along the banks of the lake several recreation centers and individual summer houses have been constructed. Along the course of Bychowska Struga breeding farms were developed and the agriculture is using more chemical fertilizers than before. These new facilities cause increased inflow of organic and inorganic pollutants to the lake. In the upstream parts of both rivers there are trout breeding ponds which indicate good water quality. Deterioration of water quality in the downstream part of Bychowska Struga was especially observed [3].

4 Pumped-Storage Power Plant Żarnowiec

4.1 Pumped-Storage Energy System in Poland

The rhythm of human activities causes that the demand for electric energy varies during the daytime, week and the season. The energy system based on the thermal and nuclear power plants has difficulties to adapt its power production to theses changes of demands. The only practical solution is the storage of large quantities of energy during low demand periods and its return during peak hours. This is accomplished by the use of pumped-storage power plants.

Poland has no geographical and hydrologic conditions, which favour the development of hydraulic power plants and especially pumped-storage plants. In Poland, approximately 1.5% of the electric energy is produced by conventional hydraulic power plants. Pumped-storage power plants produce about 2% of electric energy during peak demand. However, the production of electric energy in pumped-storage power plants requires earlier production of electric energy for pumping. In national energy system three pumped-storage power plants are in operation with total capacity of 1418 MW. The other three are the conventional hydraulic power plants equipped with pumps or reversible units (302 MW). During off-peak hours they can pump water from the tail-water to the headwater [2].

4.2 Design of the Energy System Żarnowiec

Deficit of electrical energy in Poland in the 1970s stimulated the design of a pumped-storage and nuclear plant. Both power plants were located about 70 km from Gdańsk in the vicinity of Lake Żarnowiec, which was to be used as the lower reservoir of the pumped-storage plant (4 x 170 MW) and as a cooling water reservoir for the nuclear plant (4x440 MW). Nuclear plant was designed to provide electric power for pumping in pumped-storage plant during off-peak hours. The planning and engineering of the Żarnowiec pumped-storage and nuclear projects was initiated in the 1970s. In 1983 the pumped-storage plant was put into operation, but in 1990 the construction of nuclear power plant was stopped as the result of public protests caused by Chernobyl catastrophe and difficulties in further financing of the project from the state budget. Few months after it has been decided to abandon the construction of this power plant [2].

At present, because of economic crisis and foreseen energy shortage in Poland state authorities returned to the design of the nuclear power plant on the Lake Żarnowiec.

4.3 Basic Data of Pumped-Storage Power Plant Żarnowiec

During the turbine cycle maximum power of pumped-storage power plant Żarnowiec is 716 MW with discharge to the lake 700 m^3/s. During pump cycle maximum power taken from the power grid is 800 MW and the discharge to the upper reservoir amounts to 600 m^3/s. The project consists of the following parts:

- upper reservoir (artificial),
- four penstocks,
- power plant with 4 reversible Francis type units,
- open channel connecting the power plant with the lake,
- lower reservoir (lake Żarnowiec with control weir at the outflow from the lake).

Upper reservoir is an artificial structure situated on the moraine hills. It has the surface area of 135 ha and 15.9 million m^3 of the total volume. Operational volume amounts to 13.8 million m^3 and has energy potential of 3.9 GWh. Maximum daily variation of water surface elevation in the reservoir is 16 m. Filling of the upper reservoir proceeds usually during night time from 23 to 6 h and eventually from 13 to 16 h. Emptying of reservoir (turbine cycle) is during two periods: in the morning (7-12 h) and in the evening (17 to 22 h). Maximum water surface elevation is 126 m above sea level. The crest of the reservoir embankment is on the elevation 127.7 m.

Four pressure penstocks have the length of 1100 m and varying diameters from 710 cm (near the upper reservoir) to 540 cm near the power plant.

Hydraulic power plant is equipped with 4 reversible Francis units of 6 m in diameter. The total efficiency of the pumped storage system is 73%.

Outflow - inflow open channel is 835 m long and connects the power plant with the lake. It has trapezoidal cross-section with 100 m width at the bottom. Maximum depth near the power plant is 13 m and maximum velocity does not exceed 1 m/s [5].

During 30 years of exploitation the energy system of the power plant was modernized with the aim to increase the power plant reliability. Now there are continued modernization works connected with the introducing of computer methods of the monitoring, controlling and the diagnostic.

5 Studies of Lake Żarnowiec

It was recognized that the operation of both energy projects would result in considerable hydrodynamic, thermal and environmental changes in the lake and therefore comprehensive studies of the lake were undertaken. They included hydrology, hydrodynamics, thermal regime, hydrochemistry, hydrobiology, ichthyology, and meteorology of the lake region. These studies started in 1973 and were carried out regularly till 1990.

5.1 Studies of Lake Żarnowiec Carried Out During 1973 - 1990

The first comprehensive studies were carried out during 1973-1975 with the main purpose of engineering character. In subsequent years studies were conducted within a limited program till the decision of closing the construction of nuclear power plant. They included the following problems: hydrology and water balance of the lake, thermal regime of the lake and meteorological conditions near the lake, hydrodynamic conditions of the lake, hydrochemistry and hydrobiology of the lake, ichthyology and fishery state of the lake, and consequences of the catastrophe of the upper reservoir filled with water.

5.2 Studies of the Lake Carried Out in 1994

After 10 years of the operation of pumped storage power plant, in 1993, it has been decided to carry out another comprehensive study which had to indicate:

- how the operation of the power plant influenced the ecological state of the lake;
- what changes of the lake are caused by the power plant and what are due to varying use of the land of the catchment;
- what we have to do to keep present lake ecological characteristics taking into account possible changes in lake hydrodynamics and changes in the catchment.

This study included: changes in the shoreline, lake hydrodynamics and thermal regime, water quality assessment of the lake and its susceptibility to degradation, environmental and tropic conditions of the lake, hydrobiological conditions (phyto- and zooplankton, zoobenthos, primary productivity), and fishery state of the lake (resources and communities). Studies, in their predominant part, were based on the direct measurements in the lake as well as on the laboratory analysis of the samples taken from the lake.

The studies performed in 1994 indicated that visible hydrochemical and hydrobiological changes in the lake cannot be attributed to the operation of the power plant, but to the changes which have been currently observed in lake catchment management. The most important factor influences the ecological state of the water resources are agricultural non-point source pollutions. By the nature they are spatial character, and therefore it seems appropriate to use a GIS to analyze and model the data related to this problem. This type of approach to the design on water and environmental management has not been used in Poland and this was one of the first trials in this respect. The Lake Żarnowiec catchment was taken as the case study, but the methodology will be suitable for use with other small river basins. The study uses DHI Software – Mike 11 and Mike SHE.

This study has been undertaken in 1996 within the grant of the Polish Committee for Scientific Research. The purpose of the study was the development of practical methods for the analysis of the impact of the changes in the management of the small river catchment on the water resources and water quality. This research work consisted the continuation of the studies that were presented above. Therefore the application of proposed methods will be made to the Lake Żarnowiec catchment.

The Lake Żarnowiec catchment was very useful for the pilot study. It is typical meso-scale catchment (<300 km^2) so it is large enough for a check of general data availability at different scales and for further regional differentiation. Moreover long-term measurements of many factors are available. This data will be used for model verification.

The study included:

- the estimation of the quality and quantity of the catchment data that is sufficient for calculations and for modeling purposes, and will be used in the study;
- the assembly of the catchment characteristics (land use, vegetation cover, hydrology, soil parameters, quality of lake and rivers water etc.) and construction of the GIS data structure;
- calculations and GIS overlay analysis on the basis of the existing data;
- estimation of the model parameters (data transformation);
- prognostic calculations and its analysis;
- elaboration of methodology of the problem-solving;
- analysis of the possibility of implementation of the proposed methodology to the other small river catchments [3].

5.3 Studies of the Lake in 2004-2007

In 2004 designs connected with the reconstruction of the right-hand flood dyke of the Lower Piaśnica were initiated together with the Environmental Impact Assessment (EIA). Within EIA the following problems were taken into account:

- actualization of hydrological data concerning the outflow from the lake (on the basis of data supplied by the pumped-storage power-plant);
- elements of the natural environment included by the range of impact of designed project;
- assessment of the influence of the changes of hydrological regime on the state of ecosystems in the valley of the Lower Piaśnica;
- assessment of the actions, which aim at the mitigation, limitation or natural compensation of the negative impacts on the environment;
- proposition of the monitoring of the impact of the project on the environment [8].

6 Influence of the Power Plant on the Physical and Chemical Conditions of the Lake

6.1 Range and Methodology of Chemical Investigations

The interdisciplinary investigations carried out in 1994 on Lake Żarnowiec included, among others, chemical tests of the lake water and the surface water courses within the catchment area of this reservoir. The chemical tests were aimed at the determination of the effect of the Żarnowiec power plant and the pollutants from the catchment area upon the quality of the lake water. The qualitative assessment of the pelagic zone water was carried out by using water from the southern station and the northern station. The samples were taken from the surface water layer and the bottom layer [7].

The water quality of the eastern and western lake littoral zone has been estimated on the basis of test results of samples taken from regions with anticipated pollution. The main surface water courses were determined using the results of the samples taken from the mid-stream of the Upper Piaśnica, the Lower Piaśnica and the Bychowska Struga River. Also some random qualitative analyses of the wastewater discharged to the lake by the biological treatment plant of Nadole were carried out.

Since Lake Żarnowiec is numbered among reservoirs with no stratification [4], a chemical assessment of the pelagic and littoral zone waters was made taking advantage of the investigation results of following major indices related to lake water quality: concentration of dissolved oxygen in the bottom layer (in summer), content of organic substances (BZT_5, $ChZT_{Cr}$) in the surface layer (in summer), content of phosphates in the surface layer (in spring), mean concentration of total phosphorus (in spring and summer) in the surface layer, concentration of mineral nitrogen in the surface layer (in summer), mean concentration of total nitrogen (in spring and summer) in the surface layer. The adapted scope and frequency of the analytical work were compatible with the investigation methodology and classification of lake waters applied in Poland. The investigations also involved the following supplementary parameters: pH, colour, total alkalinity, calcium, magnesium, sodium, potassium, chlorides, and sulfates and some heavy metals. The above mentioned parameters were subject to testing in the surface water layer in spring and summer. The range of the qualitative tests of the main surface water-courses situated in the catchment area of the lake referred to: pH, electrolytic conductivity, dissolved oxygen, sodium, potassium, total iron, manganese, chlorides, sulfates, ammonia nitrogen, nitrite nitrogen, nitrate nitrogen, total nitrogen, phosphates, total phosphorus, $ChZT_{Cr}$, $ChZT_{Mn}$, BZT_5, dissolved substances and suspended solid matter.

In the assessment of the sanitary state of water in the lake streams, advantage has been taken of the results of some bacteriological investigations which were conducted simultaneously by other researchers [7]. The susceptibility of Lake Żarnowiec to degradation has been determined by making use of the morphometric, hydrographic, and catchment area characteristics, including among others, mean depth of the lake, ratio of the lake volume to the length of the shoreline of the lake basin, percentage of water stratification, quotient of active bottom area and volume of the upper surface, percentage of annual water exchange, the Schindler coefficient and method of management of the immediate catchment area.

6.2 Results of the Assessment of Water Quality of the Lake

The field investigations have indicated some evident variations in the quality of water in the middle part (pelagic zone) and along the lake shore (littoral zone).The chemical experiments on the pelagic zone water have not shown any significant differences in the major magnitudes of water quality.

The analytical investigations have proved that the pelagic zone waters are subject to only some insignificant variations in spring-and-summer. The mean concentrations of cations and anions in these waters were as follows: Ca-59.0 mg/dm^3, Mg-6.1 mg/dm^3, Na-12.4 mg/dm^3, K-1.7 mg/dm^3, Cl-23.5 mg/dm^3, SO_4-35.8 mg/dm^3. The water alkalinity was characterized by a mean value 2.6 mval/dm^3. Taking into account the contents of the main ions in the pelagic zone water, Lake Żarnowiec can be classified as a bicarbonate-calcium lake of mean salinity. Concentration of heavy metals in pelagic waters can be estimated as insignificant (Cr^{+6}-0.005 mg/dm^3, Ag-<0.001 mg/dm^3, Cu-0.024 mg/dm^3, Pb-0.014 mg/dm^3, Ni-0.004 mg/dm^3, Cd-0.001 mg/dm^3, Zn-0.070 mg/dm^3).

Chemical analyses of basic) and additional quality rating of water in the eastern and western littoral have indicated evident variations in its quality.

Comparing the investigation results it is possible to conclude that the littoral waters were much more polluted than the water of the Lake Żarnowiec pelagic zone. Moreover, the waters of the eastern littoral exhibited a definitely more advantageous quality then the water of the western shore zone. This is associated with the fact that the village of Nadole with its sewage treatment plant, numerous recreation centers and camping sites are located on the western side of the shore.

Making use of the permissible magnitudes of pollution concentrations for respective quality classes as a criterion and taking advantage of the classification method employed in Poland with respect to lake waters, the water of the eastern littoral can be included in the second class of quality. The water of the western littoral might have been recognized as third class quality, but due to extensive dead fish occurrence noted in 1994, its class quality was regarded as beyond any class.

Taking a look of the variability diagrams and the mean values of the additional parameters it is possible to observe in summer an evident increase in pH and a decline in total alkalinity and the calcium ion content. The analyzed changes in the chemical composition of water were caused by intensive biological processes taking place in this area. As a result of the CO_2 consumption in the photosynthesis of the hydrogen carbonates decomposed to carbonates whose presence contributed next to the appearance of alkalinity and the pH rise. A greater intensity of the processes under analysis was noted in the western littoral, especially, in the vicinity of the outlet sewer trunk of sewage treatment plant at Nadole. At times along big lengths of the western shore there were noted large quantities of suspended matter tearing off from the bottom during wavy conditions of water. Such a situation was accompanied by a complete deoxidize of water.

It should be noted that the biological and chemical processes occurring in the shallow littoral of Lake Żarnowiec in 1994 (particularly in the eastern part), were characteristic for eutrophicated waters. The processes were intensified to a certain extent by variations in the water mirror resulting from the operation of the power station in close proximity. It is likely that the variations in the water mirror have accelerated the biochemical circulation of the biogenic elements, particularly phosphorus.

The investigations of the water quality of the main surface water courses situated in the Lake Żarnowiec catchment were also carried out. Comparing the water quality at the outlet of the lake (Lower Piaśnica River) with the pelagic zone water it is possible to note an evident increase in the concentration of biogenic compounds. This phenomenon is caused by mixing relatively pure water of the pelagic zone with significantly polluted waters of the shallow littoral, and the water of the Bychowska Struga River. The mouth of this river is located not far from the outlet of the Lower Piaśnica from the lake. When analyzing the influence of the major water courses flowing into Lake Żarnowiec upon its quality it is worth pointing out that both the Upper Piaśnica and the Bychowska Struga are characterized by definitely higher concentration of mineral and organic components than the pelagic zone waters. Loads of organic and biogenic compounds brought by these inlet streams as well as a low sanitary condition of the water are a particular hazard to the lake. Also the wastewater discharged by the biological treatment plant of Nadole pose a serious threat to the water quality of Lake Żarnowiec.

In view of the fact that Lake Żarnowiec belongs to water reservoirs susceptible to degradation, the water quality of this reservoir can be subject to gradual deterioration, unless pollutants flowing from catchment to the lake are limited. Due to a potential eutrophication of Lake Żarnowiec, some measures should be taken to improve, first of all, the water quality of the Upper Piaśnica River and the Bychowska Struga and to raise the effectiveness of the removal of pollution from the Nadole sewage treatment plant.

7 Conclusions

Operation of the pumped-storage power plant has caused changes of ecological state and water quality of the lake. But, in general, it is to conclude, that pumped-storage power plant results in the stabilization of lake trophy during the year.

Lake Żarnowiec belongs to water reservoirs susceptible to degradation. Therefore the water quality of this reservoir can be subject to gradual deterioration. Due to potential eutrophication of Lake Żarnowiec, some measures should be taken to improve, first of all, the water quality of the Rivers Upper Piaśnica and Bychowska Struga.

Taking into consideration the lake water classification used in and the criteria to determine the quality classes, the waters of the Lake Żarnowiec in pelagic zone, in general, have been qualified to the first class of water quality. The water of shore zone reveals significantly worse quality (it is connected with water level variations caused by the operation of the power plant which intensify the circulation of the nutrient elements). But the results of numerical analysis have shown that the operation of the power plant caused the decrease of the seasonal changes of physicochemical parameters and elimination of the differences between litoral and pelagial zones.

The biological (accumulation of phytoplankton and chlorophyll *a* in pelagial of the lake) and chemical processes occurring in the Lake Żarnowiec in 1994 (particularly in the eastern part), were characteristic for eutrophicated waters. Total nitrogen content was rather low, characteristic for mesotrophy. Levels of phosphorus were similar in particular years and were typical for eutrophic lakes, and occurence of particular groups during the study period (1973-1994) suggests the stabilization of the phytoplankton changes caused by the operation of the power plant. Trophy of pelagic zones can be defined as transitory between meso- and eutrophy.

Construction of the right-hand flood dyke of the Lower Piaśnica will not affect the change of lake chemical status. However, it will result in the change of hydrological regime of the river. Increase of discharges in the Lower Piaśnica may result in unfavorable changes for the natural plants. It may create danger for the population of protected *Utricularia vulgaris*, *Neottia nidus-avis* as well as for *Molinietum caeruleae* [8].

Moreover, the change of hydrological regime of the outflow from the lake may result in the change of chemical and ecological state of the reservoir. In studies carried out during 2004-2007 such problem was not taken into account. Before the start of the project it is indispensable to perform EIA on the state of the lake waters.

Moreover, in connection with the idea of the construction of nuclear power-plant near Lake Żarnowiec actions concerning the monitoring of the state of lake waters should be continued according to the environmental objectives of the EU Water Framework Directive.

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A103**

Static and Dynamic Analysis of Spillway Tunnel Final Concrete Lining at Knezevo Dam

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Abstract. This paper presents static and dynamic analysis of spillway tunnel concrete lining at Knezevo dam. Two characteristic sections along the tunnel are analyzed: first before the injecting curtain and second after the injecting curtain. Critical loading condition for dimensioning of spillway final lining is when the tunnel is under pressure. For the first section two scenarios are analysed. In the **first scenario** assumption is made that the primary lining (shotcrete, anchors, steel ribs) are not existing and that all loads are received from the final concrete lining. In the **second scenario** it is assumed that the excavation will be fully stabilized with installing the primary lining. This means that loads from the surrounding rock material in dry condition are received by the outer tube. At this stage after installing the final concrete lining, it will be stressed only from self weight of the concrete. For the section after the injecting curtain assumption is made (on safety side) that rock material around the tunnel is in dry condition and that this loads are received by the outer tube. Final concrete lining is loaded with water pressure from inside defined with the piezo line, earthquake influence and temperature changes with same intensities as for section before injecting curtain.

Keywords: Static and dynamic analysis, spillway tunnel, numerical analysis, FEM

1 Introduction

Spillway tunnel is designed with slope of 7.7% and developed length L=169.4m. Inside diameter of the tunnel is d=4.4m, and the thickness of the final concrete lining is d_c=40cm.

In numerical analysis of the final concrete lining overburden of rock material with high of 40 meters (Hn=40m) is taken into account. Two characteristic sections along the tunnel are analyzed: first (section 1-1) before the injecting curtain and second (section 2-2) after the injecting curtain.

Critical loading condition for dimensioning of spillway final lining is when the tunnel is under pressure. It is assumed that from inner side of the spillway tunnel water pressure is acting with intensity defined with the piezo line which starts from the point in the shaft axis where annular flow becomes jet flow, and ends in the crown at the end of the tunnel (Fig. 1).

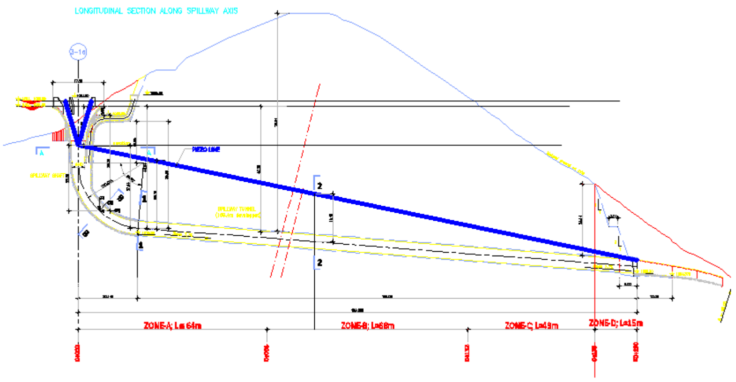

Fig. 1. Piezzo Line along the Spilway Tunnel

For the first section, normal water level (N.W.L. 1061.50) is taken into account, and two different scenarios are considered:

- I-st scenario: All loads from outside the tunnel are received by the final concrete lining (assumption that primary lining is not existing), and

- II-nd scenario: loads from self weight of the overburden in dry conditions are received by the primary lining (shotcrete, anchors and still ribs) and the final lining is receiving loads from saturated rock material.

In both scenarios water pressure from inside of tunnel is acting, with intensity defined with the piezo line (P_{wi}= 230 KN/m^2).

For the second section (after the injecting curtain), assumption is made that rock material around the tunnel lining is in dry conditions and this loads are received by the primary lining, and the final lining is receiving loads from water pressure inside the tunnel (P_{wi}=160 KN/m^2).

2 Numerical Analysis

2.1 Numerical Model

For numerical analysis of the spillway final concrete lining Finite Element Method (FEM) is used implemented in **Phase 2** software developed from ROCSCIENCE Inc. from Canada (www.rocscience.com).

Numerical model for the continuum includes 40m overburden above tunnel opening, and 35m of rock material on left, right and under the tunnel opening as shown on the following figure.

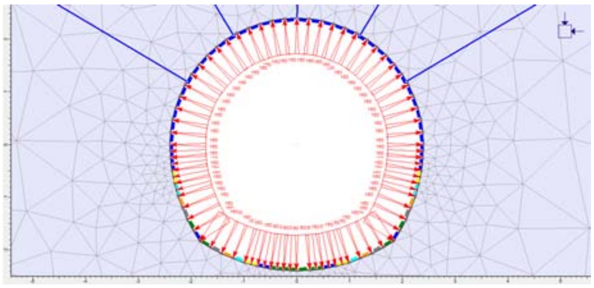

Fig.2. Zoomed view of the numerical model for section after the injecting curtain with water pressure inside the tunnel with intensity (P_{wi}=160 KN/m^2)

2.2 Rock Material Input Parameters

All calculations for the final lining of the spillway tunnel are made assuming that Rock material along the tunnel is Class – IV (Poor rock after Bieniawski). For tunnel elbow it is assumed that the rock material is Class – III (fair rock after Bieniawski). Adopted values for Strength deformation parameters of the rock material and final concrete lining are shown in the following table.

Table 1. Adopted parameters in numerical analysis of spillway tunnel and elbow section

Material	Type	Failure Criterion	γ (Kn/m^3)	C (KPa)	ϕ^0	Em (MPa)	ν
Rock Class III	Elasto-plastic	Hoek&Brown	26.7	275	49	3100	0.25
Rock Class IV	Elasto-plastic	Hoek&Brown	26	230	38	1025	0.28
Concrete	elastic	-	25	-	-	30000	0.2

2.3 Loads on Final Concrete Lining

Section before injecting curtain (Section 1-1). For the section before injecting curtain (Section 1-1) two scenarios are analyzed.

In the **first scenario** assumption is made that the primary lining (shotcrete, anchors, steel ribs) are not existing and that all loads are received from the final concrete lining. Loads which are acting on the final concrete lining are as follow:

- Self weight of final concrete lining with thickness d_c=40cm;
- Self weight of overburden in dry condition => $26 \cdot 40$=1400 kN/m^2;
- Self weight of overburden in saturated condition =>$27 \cdot 40$=1080 kN/m^2;

- Water pressure inside the spillway tunnel =>10·23=230 kN/m^2;
- Earthquake influence with horizontal acceleration 0.3g, and vertical acceleration 0.15g, acting in down direction;
- Negative temperature change (cooling) of 15 degrees Celsius.

Spillway tunnel is one composite structure consist of **outer tube** (rock material, shotcrete, anchors, and ribs) and **inner tube** (in situ cast concrete).

In the **second scenario** it is assumed that the excavation will be fully stabilized with installing the primary lining (shotcrete, anchors, ribs), which means that all deformations of the surrounding rock material are finished. This also means that loads from the surrounding rock material in dry condition ($P_{V,dry}$=1040 KN/m^2) are received by the outer tube. At this stage after installing the final concrete lining, it will be stressed only from self weight of the concrete.

After filling the accumulation with water to the normal water level, rock pressure in saturated conditions from outside the spillway tunnel will be ($P_{V,sat}$=1080 KN/m^2). These loads in saturated conditions are slightly increased compared with dry conditions, for value ΔP_V=40 kN/m^2.

Taking into account the previous assumption that the tunnel excavation is fully stabilized with the primary lining, and that the final concrete lining is set after finishing of all rock deformations, in dry conditions final concrete lining is loaded only with self weight of the concrete. Because the loads from the overburden in dry conditions are already received from the outer tube, in saturated conditions loads which will influence the final concrete lining will be only the difference ΔP_V=40 KN/m^2. This assumption is on safety side because in case of water flow in the spillway tunnel, outside loads decreases the water pressure from inside the tunnel, which decrease tension axial forces in the final concrete lining.

Section after injecting curtain (Section 2-2). For this section assumption is made (on safety side) that rock material around the tunnel is in dry condition and that this loads are received by the outer tube. Final concrete lining is loaded with water pressure from inside defined with the piezo line (P_{WI}=160 KN/m^2), and earthquake influence and temperature changes with same intensities as for section before injecting curtain.

3 Results from Numerical Analysis

3.1 Spillway Tunnel Section before Injecting Curtain and Final Lining Only

In this section cross-sectional forces in the final concrete lining of the spillway tunnel will be shown assuming that the primary lining does not exist, and that all loads are received by the final lining **(Scenario 1).**

Stage Installed Final Lining, Rock Pressure from Self Weight of Overburden in Saturated Conditions from outside, and Water Pressure from Inside the Tunnel. On the figures below cross section forces (axial forces(N), bending moments (M), and shear forces (Q)) are shown in various sections of the concrete lining of the spillway tunnel from self weight of overburden in **Saturated conditions** (Hn=40m) from outside, and **water pressure from inside** the tunnel with intensity Pwi=230 KN/m^2 . Stresses for the most unfavourable sections (tunnel crown, invert etc.) are calculated and shown in the table.

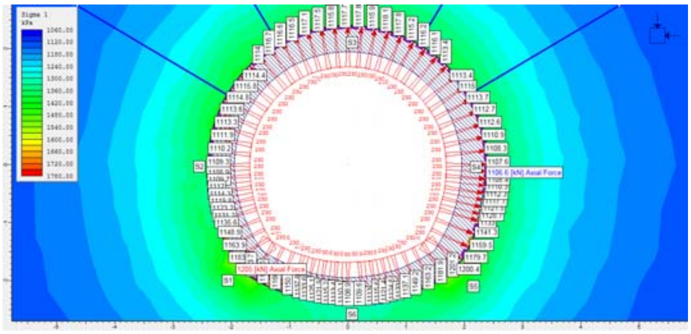

Fig. 3. Axial forces in characteristic sections of the final concrete lining from self weight of overburden in saturated conditions and water pressure from inside the final lining

Table 2. Normal stresses in characteristic sections of the final concrete lining from Self Weight of concrete lining, self weight of overburden (Rock Pressure), water pressure from outside, and water pressure from inside the spillway tunnel

Section	d(m)	A(m2)	W(m3)	N(KN)	M(KNm)	σ1(KN/m2)	σ2(KN/m2)	Coment
1	0.74	0.74	0.091267	1205	-135	149	3108	OK
2	0.4	0.4	0.026667	1109	12	3223	2323	OK
3	0.4	0.4	0.026667	1118	-1	2758	2833	OK
4	0.4	0.4	0.026667	1109	12	3223	2323	OK
5	0.74	0.74	0.091267	1205	-135	149	3108	OK
6	0.4	0.4	0.026667	1109	30	3898	1648	OK

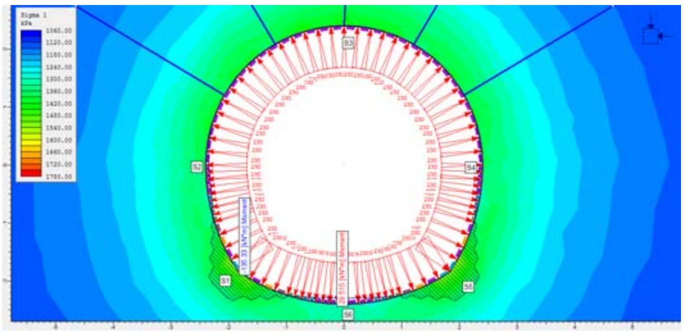

Fig. 4. Extreme Bending Moments in the final concrete lining from self weight of overburden in saturated conditions and water pressure from inside the final lining

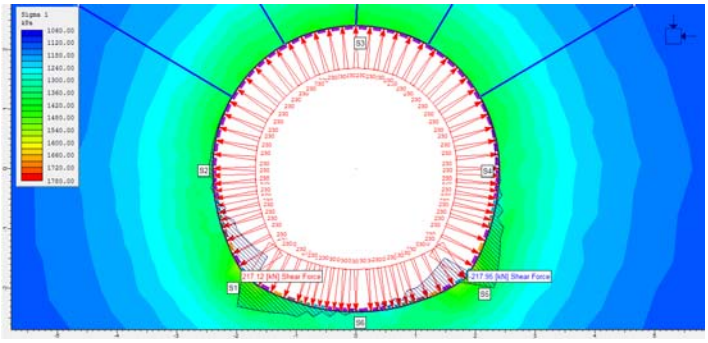

Fig. 5. Extreme Shear Forces in the final concrete lining from self weight of overburden in saturated conditions and water pressure from inside the final lining

From table above it can be seen that in case when all loads from the surrounding rock material are received from the final concrete lining (assumption that primary lining is not existing), all sections of the concrete lining are in compression, and all stresses are less than the allowable normal stresses due to bending for concrete grade C-30 MPA.

3.2 Spillway Tunnel, Section before Injecting Curtain, Primary and Final Lining

In this section cross-sectional forces in the final concrete lining of the spillway tunnel will be shown assuming that all loads from overburden in dry condition are received by the **primary lining**, and that in saturated conditions loads which will influence the **final concrete lining** will be only the difference between the loads in saturated and dry condition (ΔP_V=40 KN/m^2), because loads from the overburden in dry condition are already received by the primary lining. **(second scenario).**

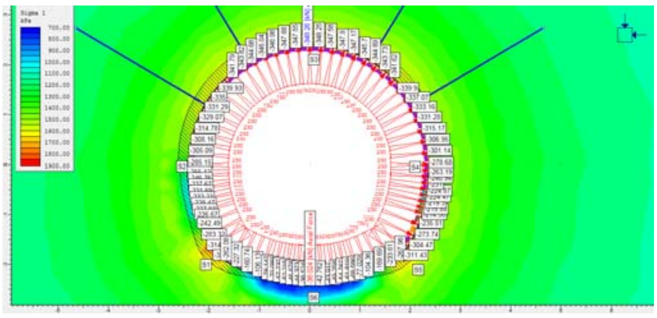

Fig 6. Axial forces in characteristic sections of the final concrete lining in saturated conditions from outside and water pressure from inside

Stage Installed Primary and Final Lining, Saturated Conditions from Outside and Water Pressure From Inside. On the figures below cross section forces (axial forces(N), bending moments (M), and shear forces (Q)) are shown in various sections of the concrete lining from **self weight** of overburden in **Saturated conditions** with height of H_n=40m, and **water pressure inside** the tunnel (P_{wi}=230 KN/m^2). Stresses for the most unfavourable sections (tunnel crown, invert etc.) are calculated and shown in the table.

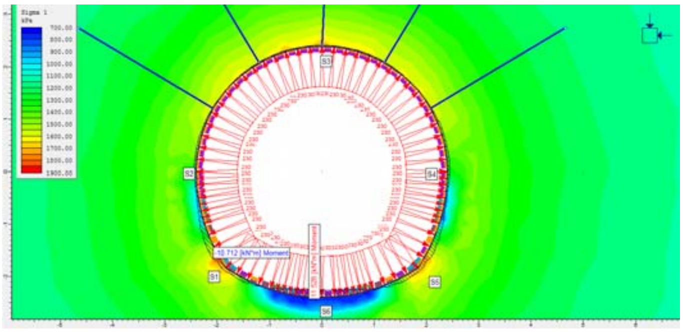

Fig.7. Bending moments in characteristic sections of the final concrete lining in saturated conditions from outside and water pressure from inside

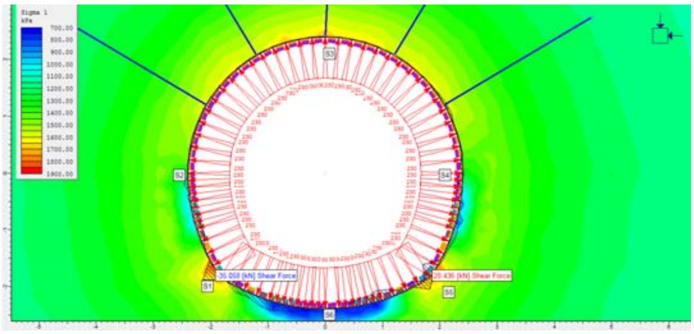

Fig.8. Extreme Shear forces in the final concrete lining in saturated conditions from outside and water pressure from inside

All sections of the final concrete lining are in tension, which indicate necessity of reinforcement.

Maximum ultimate shear stress (τ_{xy}) in concrete lining at sections 1&5 appears with intensity:

$$\tau_{u\,max} = \frac{1{,}6 \cdot Q_{max}}{b \cdot d} = \frac{1{,}6 \cdot 35}{1{,}0 \cdot 0{,}74} = 76 kN/m^2 \qquad (1)$$

According to Macedonian standards for concrete grade MB30 τ_r = 1100 KN/m^2

$$\tau_{u\,max} = 76 kN/m^2 < \tau_r = 1100 kN/m^2 \rightarrow OK \qquad (2)$$

Table 3. Normal stresses in spillway concrete lining from self weight of concrete, water pressure from outside and water pressure from inside (SW+WP_out+WP_in)

Section	d(m)	A(m2)	W(m3)	N(KN)	M(KNm)	σ1(KN/m2)	σ2(KN/m2)	Coment
1	0.74	0.74	0.091266667	-315	-11	-546	-305	necessary reinforcement
2	0.4	0.4	0.026666667	-285	-4	-863	-563	necessary reinforcement
3	0.4	0.4	0.026666667	-348	2	-795	-945	necessary reinforcement
4	0.4	0.4	0.026666667	-285	-4	-863	-563	necessary reinforcement
5	0.74	0.74	0.091266667	-315	-11	-546	-305	necessary reinforcement
6	0.4	0.4	0.026666667	-38	12	355	-545	necessary reinforcement

3.3 Section after the Injecting Curtain, Primary and Final Lining

For this section assumption is made (on safety side) that rock material around the tunnel is in dry condition and that this loads are received by the outer tube (rock material and primary lining). Final concrete lining is loaded with water pressure from inside defined with the piezo line (P_{WI}=160 KN/m^2), earthquake influence and temperature changes with same intensities as for section before injecting curtain.

Table 4. Normal stresses in characteristic sections of spillway tunnel in dry conditions from outside and water pressure from inside the tunnel

Section	d(m)	A(m2)	W(m3)	N(KN)	M(KNm)	σ1(KN/m2)	σ2(KN/m2)	Coment
1	0.74	0.74	0.091266667	-342	39	-35	-889	necessary reinforcement
2	0.4	0.4	0.026666667	-287	-3	-830	-605	necessary reinforcement
3	0.4	0.4	0.026666667	-285	0	-713	-713	necessary reinforcement
4	0.4	0.4	0.026666667	-287	-3	-830	-605	necessary reinforcement
5	0.74	0.74	0.091266667	-342	39	-35	-889	necessary reinforcement
6	0.4	0.4	0.026666667	-298	-9	-1083	-408	necessary reinforcement

Stage installed primary and final lining and water pressure from inside. On the figures below cross section forces (axial forces(N), bending moments (M), and shear forces (Q)) are shown in various sections of the concrete lining from self weight of overburden in dry conditions with height of Hn=40m, and water pressure inside the tunnel (Pwi=160 KN/m^2).

In table 4 stresses in various sections of the concrete lining from **self weight** of overburden in **dry conditions** with height of H_n=40m, and **water pressure inside** the tunnel (P_{wi}=160 KN/m^2) are shown.

All sections of the final concrete lining are in tension, which indicate necessity of reinforcement.

Maximum ultimate shear stress (τ_{xy}) in concrete lining at sections 1&5 appears with intensity:

$$\tau_{u\max} = \frac{1,6 \cdot Q_{\max}}{b \cdot d} = \frac{1,6 \cdot 66}{1,0 \cdot 0,74} = 143\mathrm{kN/m^2} \tag{3}$$

According to Macedonian standards for concrete grade MB30 τ_r = 1100 KN/m^2

$$\tau_{u\max} = 143\mathrm{kN/m^2} < \tau_r = 1100\mathrm{kN/m^2} \rightarrow \mathrm{OK} \tag{4}$$

4 Final Concrete Lining Reinforcement Calculations

Calculations of ultimate forces **(M_{ul}, N_{ul})** are made in accordance with Macedonian regulations for reinforced concrete **(PBAB)**, and calculations of necessary reinforcement are made with computer software **PanelPro** (RadImpex Belgrade).

Ultimate forces are calculated by multiplying different loads with partial safety coefficients all in accordance with Macedonian standards (**PBAB).**

Following load combinations are considered:

$$\mathrm{COMBO1} = 1.3\mathrm{SW} + 1.5\mathrm{WP\_OUT} + 1.3\mathrm{DT} \tag{5}$$

$$\mathrm{COMBO2} = 1.3\mathrm{SW} + 1.5\mathrm{WP\_OUT} + 1.5\mathrm{WP\_IN} + 1.3\mathrm{DT} \tag{6}$$

$$\mathrm{COMBO3} = 1.3\mathrm{SW} + 1.3\mathrm{WP\_OUT} + 1.3\mathrm{EQ} + 1.3\mathrm{DT} \tag{7}$$

Values for cross-section forces from different loads can be obtained with subtracting the values from two subsequent (following) stages.

All Calculations for necessary reinforcement in the final concrete lining are made with reinforcement type RA -400/500 and concrete grade MB30. Minimum concrete cover should be a=5-7cm.

Adopted Reinforcement for Spillway Tunnel. Cross sectional reinforcement: Ø16mm/20cm from internal and external side of the lining. Longitudinal reinforcement: Ø16mm/20cm from internal and Ø14mm/20cm from external side of the lining.

5 Conclusions

Results from the numerical calculations for the final concrete lining of spillway tunnel at Zletovica Dam shows that most unfavorable stress condition appears when water pressure from inside the tunnel influence the concrete lining. Also these calculations shows that more unfavorable stress condition in the final concrete lining appears when primary lining (shotcrete, steel ribs, anchors) are taken into account in the numerical calculations, compared when this lining is neglected in the model. The explanation for this is the fact that when the primary lining is not taken into the numerical model, loads from the surrounding rock material in saturated condition generate compression stresses in the final concrete lining which reduce the tension stresses produced by the internal water pressure. In case when primary lining is taken into account with the numerical model, all loads from the surrounding rock material in dry condition are received by this lining. In saturated conditions loads which are influencing the final lining from outside the tunnel are the difference between the rock load in saturated and dry condition, which are with lower intensity from the loads (water pressure) inside the tunnel, so the whole tunnel section is tensioned and there is necessity of reinforcement.

References

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2. Phase II, ROCSCIENCE Inc. from Canada (www.rocscience.com).

International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A109**

Flood Disasters' Inventory in Turkey in 2009

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Abstract. Turkey is a country both on Europe and Asia and located between 36^{o}-42^{o} N latitudes and 26^{o}-45^{o} E longitudes with covering an area of 780 580 km^2 including the lakes. The arithmetical average of the annual precipitation is 642.6 mm and it corresponds to 501.6 km^3 of total water volume. The figures for the surface water potential of Turkey can be stated as runoff volume 185.59 km^3, runoff coefficient 37%. Due to geographical location, geology, and topography, Turkey undergoes mainly three different types of natural disasters related to gravity flows; floods, landslides, and snow avalanches. Flooding is second important natural hazard after earthquakes with 39 flood events causing 25 deaths per year, on average. A flood inventory of the period extending from 1955 to 2009 having 2089 events was prepared using a simple computer program for PC use and based Excel for easy access to different characteristics of each flood. By categorization of the available data in hand, spatial and time distributions of past flood events were determined. In recent years, a number of devastating flood events has occurred in various river basins of Turkey. In many cases, floods caused deaths and extensive damages to both public and private properties. Almost after each flood, a large proportion of the damage has been paid by state, in addition to losing significant revenues due to the consequences of economic disruption. The flood disaster at Trabzon on 2nd August 2005, was one of the major flood events recently experienced and it caused 9 deaths and led to financial losses worth \$ 25 10^6. Another big flood disaster occurred in the Southeastern Anatolia on November 2, 2006, and this flood caused 39 deaths, damaged more than 3800 houses, flooded 365000 decares of agricultural land, with a total damage of \$ 300*10^6. In order to prevent the floods and minimize the adverse effects to property, both structural and non-structural solutions are applied in Turkey. In this paper a flood inventory covering the period extending from 1955 to 2009, factors causing the flood, and the solutions to decrease the flood damages are presented.

Keywords: Flood disaster, flood inventory, flood damage, structural and non-structural solutions

1 Introduction

In designing the water control structures on rivers, economy is the most important parameter. This does not mean that to design a dam to control a flood which may occur any time in a year will be considered economical. Therefore it is necessary to design the engineering structures on the rivers to cope with a flood of a certain magnitude that means a flood of a certain return period.

At present due to economical restrictions it is not possible to design and build unnecessarily large and expensive water structures. For this purpose reliable estimate of the magnitude of the project flood becomes very important. This creates the necessity of having an flood inventory covering the whole Turkey. It is believed that such a document will also clarify some of the questions when new settlements will be planned at the flood prone areas and help the decision makers at the very early stage of the investment.

In this connection the floods of the period extending from 1955 to 2009, were analyzed from engineering and economic perspectives, by creating a data base having different parameters to define a single flood event. In order to reach the flood data compiled from the archives, a simple computer program for PC use and based Excel was prepared for easy access to different characteristics of each flood. By analyzing the data about the past flood events in Turkey, a series of maps and graphics were prepared. In Fig. 1 the areal distribution of the floods is presented. The archives of the Turkish State Hydraulics Works (DSI), the Turkish State Meteorological Service(DMI), the General Directorate of Disaster Affairs (AFET), local TV's and newspapers on different natural disasters such as earthquake, land slide, rockfall, snow avalanches and inundations were utilized and out of 9000 disasters, 2089 flood cases were selected (Table 1). With the data in hand it was possible to study the spatial (Fig. 1) and seasonal distribution of the flood events in Turkey (Table 2).

2 Factors Affecting Flooding

Besides parameters originating from the types of precipitation, there are other important factors, such as the prevailing hydrometeorologic conditions prior to flooding, topography, vegetation, and land use in the flood-prone area.

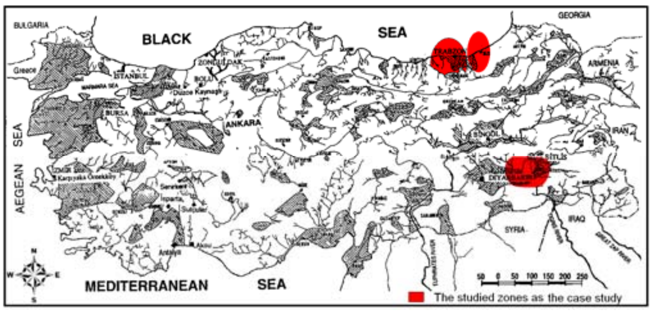

Fig. 1. Areal distribution of flood events in Turkey [1]

In the snow melt-produced floods of central Anatolia, it is not enough to have heavy snow accumulation on the ground; the previous autumn's meteorological conditions also have an important role in producing flooding.

Table 1. Distribution of floods and damages caused by calendar year

Year	Number of flood events	Death Toll	Flooded Area (ha)	Flood Damage (US $)
1955	27	-	86 675	3 122 733
1956	22	90	178 668	11 324 144
1957	22	13	49 336	7 617 225
1958	49	190	61 266	20 276 255
1959	50	17	21 954	11 564 586
1960	98	1	78 123	12 370 351
1961	42	24	45 860	15 510 580
1962	117	15	94 014	16 423 257
1963	122	25	191 983	35 255 747
1964	69	32	53 149	11 379 669
1965	63	3	96 358	24 833 845
1966	90	25	137 971	39 872 782
1967	81	4	52 466	8 935 221
1968	170	24	170 029	86 153 068
1969	30	6	125 104	20 066 068
1970	8	3	18 306	876 242
1971	31	6	4 400	7 831 998
1972	109	33	21 076	17 462 526
1973	19	22	44 188	11 090 778
1974	35	73	2 534	5 366 167
1975	51	8	7 242	18 141 515
1976	28	1	324	5 488 901
1977	26	11	426	4 651 923
1978	22	-	13 759	1 466 603
1979	22	61	2 950	13 027 595
1980	36	9	48 473	12 449 342
1981	16	1	1 980	13 740 520
1982	10	-	2 812	1 307 639
1983	12	6	1 588	21 154 159
1984	12	-	28 457	2 602 760
1985	7	-	300	211 636
1986	8	-	-	-
1987	7	-	915	1 275 708
1988	24	17	3 910	10 030 206
1989	11	1	9 550	3 252 309
1990	22	56	5 508	207 616 671
1991	17	52	11 384	6 319 905
1992	13	1	690	12 644 290
1993	2	-	60	56 095
1994	8	4	1 655	1 283 090
1995	27	164	201 077	202 000 000
1996	8	-	11 026	1 410 010
1997	19	-	4 817	60 000
1998	59	79	42 345	41 831 770
1999	21	5	9 889	2 235 967
2000	25	9	20 367	138 281 040
2001	56	37	95 545	405 326 632
2002	48	47	4 494	104 840 590
2003	22	14	56 111	325 377 200
2004	39	15	23 458	307 586 950
2005	39	35	53 920	502 706 880
2006	44	62	76 377	353 682 242
2007	47	44	1 350	38 416 360
2008	27	15	10	18 284 150
TOTAL	**2089**	**1360**	**2 276 229**	**3 146 093 900**

Table 2. Distribution of flood events based on watersheds(*), months and seasons for the period 1955-2008

Number of	WINTER			SPRING			SUMMER			AUTUMN			ANNUAL
watershed	Dec	Jan	Feb	Mar	Apr	May	June	July	Aug	Sep	Oct	Nov	TOTAL
1	14	9	16	5	3	1	4	4	3	3	25	3	90
2	18	6	1	10	1	0	6	12	16	13	30	4	117
3	55	38	9	7	3	7	12	9	4	25	15	31	215
4	26	4	4	1	0	0	2	1	0	0	7	2	47
5	13	5	18	7	0	3	5	1	0	1	1	1	55
6	7	2	6	2	0	3	2	1	2	1	1	3	30
7	46	40	9	11	2	5	6	0	4	12	0	3	138
8	8	7	0	1	0	0	0	0	1	5	2	2	26
9	30	24	5	2	0	0	8	2	5	2	4	10	92
10	0	0	0	0	0	0	1	2	0	1	0	0	4
11	2	0	0	0	1	0	9	1	1	0	0	1	15
12	9	2	6	17	20	17	28	27	17	12	8	2	165
13	1	1	3	1	1	14	29	25	21	4	1	2	103
14	5	5	2	8	13	9	12	28	22	3	0	4	111
15	6	6	1	18	12	25	60	28	9	18	0	0	183
16	8	3	1	4	10	9	13	4	4	3	1	0	60
17	36	14	0	5	0	0	0	1	1	0	0	6	63
18	9	7	2	13	2	5	6	0	2	1	1	0	48
19	12	12	12	6	20	22	3	0	0	11	2	3	103
20	3	2	0	17	8	3	1	6	0	1	1	0	42
21	1	13	2	17	43	20	10	8	11	3	2	12	142
22	0	1	0	1	12	5	42	51	25	8	2	2	149
23	0	0	0	1	0	0	0	2	0	0	0	0	3
24	0	1	0	2	11	4	15	12	11	1	0	0	57
25	0	0	0	0	12	2	3	5	6	1	1	1	31
Monthly totals	309	202	97	156	174	154	277	230	165	129	104	92	2089
Seasonal total	608			484			672			325			2089

(*) given in Figure 2

If the previous autumn had very little rainfall and a warm period before the permanent snow cover, a major portion of the snow melt water will be absorbed by the dry soil mantle, and surface flow will start only after the upper soil zone is completely saturated. But after a rainy and cool autumn almost all the water in the snow pack will be transformed to surface runoff and subsequently to floods. Of course other meteorological parameters such as the relative humidity, and wind velocity and geomorphologic parameters such as surface size, slope, geological structure, elevation above mean sea level and existing vegetation have important roles in the formation and magnitude of floods. There is one more important factor to be considered for developing countries such as Turkey; the reduction in flood plain capacity. This causes a decrease in the available safe wetted area and an increase in the devastating effect of floods. Unauthorized use of flood plains by the construction of local barriers and settlements, construction of inadequate bridges and culverts, and creating new agricultural plots are examples. Continuous forest cutting during the last

40 years to obtain new plots for agricultural purposes, especially on the steep slopes of the northern Anatolian mountain range, and the clear cutting of shrub-size oaks in the Southeastern Anatolia for winter use in stoves have increased the possibility of landslides with flash floods in these parts of the country. Erosion of valuable surface soil and transportation of sediments first to rivers then to the sea is very common.

3 Floods Inventory in Turkey

According to some statistics used in the flood inventory for the period 1955-2009; the total number of floods analyzed for this period is 2089. There are 25 drainage basins in Turkey. The Susurluk Basin has the highest number of floods, 215 in number. While the maximum number of floods (170) occurred in 1968, there were no flood damages in 1986. As shown also in Table 1, the most devastating year was 1995. In terms of seasonal distributions, 672 floods took place in summer; and as for monthly distributions, 309 floods took place in December (Table.2). The areal distribution of floods occurred during the period 1955-2009 shows that Black Sea Region, Eastern Anatolia and Mediterranean Sea Region respectively had higher flood risk than the others. It is recorded information that the flood causing the maximum number of deaths in Turkey occurred on September 11, 1958 in Ankara, Central Anatolia. The number of 169 people living on the banks of Hatip Creek died of this event. After that flood event, Hatip Creek has been regulated by DSI, and now this river flows through Ankara under ground in a culvert. As a result, during the period that flood inventory covers, 39 floods have occurred and 25 people have lost their lives during the floods on average per year. Besides this, the monetary cost of the economic losses per year on average has been over than \$ $58 \cdot 10^6$.

Although the Eastern Black Sea region has the highest long term mean annual precipitation (Fig. 3), the Susurluk Basin in the south of Marmara Sea experienced the highest number of floods and these floods in the Susurluk Basin during 54 years period caused 17 deaths and \$ $65 \cdot 10^6$ damage.

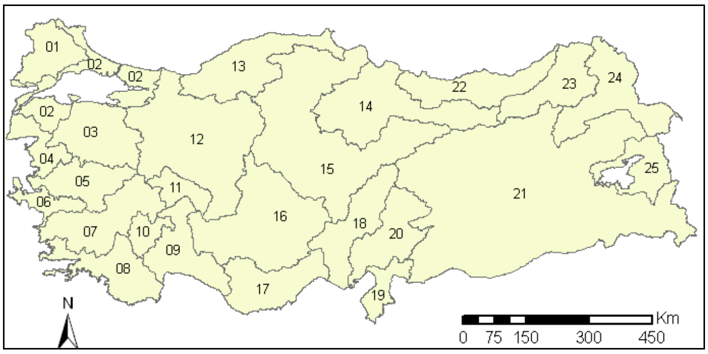

Fig. 2. Locations and numbers of watersheds in Turkey

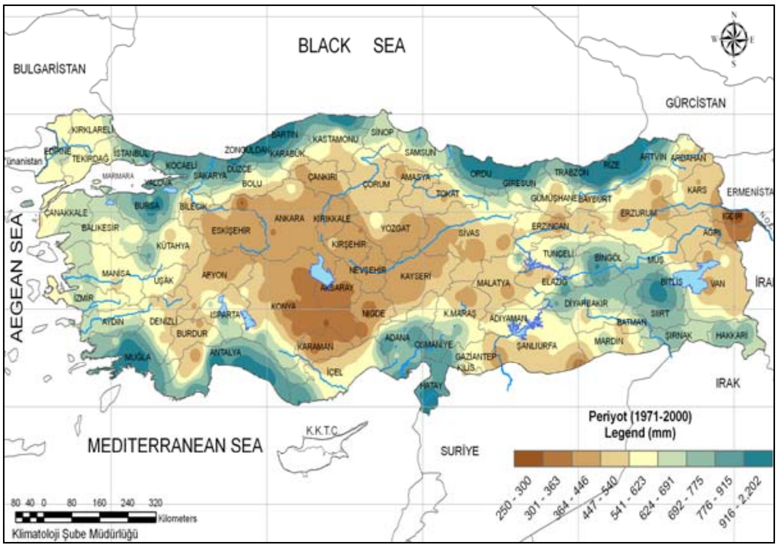

Fig. 3. The long term mean annual precipitation map of Turkey [2]

In Central Anatolia Kızılırmak Basin which has dry continental climate experiences snowmelt induced floods with lower peaks but longer flood periods and the 72 people died during 54 years.

The Eastern Black Sea region has the less number of floods, but more destructive. Therefore 235 people have died in this region since 1955. Property damage totaled $ 359.185.738.

In this study the floods and their damages for the last 54 years are studied and three extreme cases are presented as case studies. The first case is about Rize (Merkez, Guneysu, and Cayeli) Flood which occurred on July 23, 2002 and this flood caused 27 deaths. The second case is Trabzon Flood on August 02, 2005 and 9 people lost their life. The last case is South Eastern Anatolia Flood on November 02, 2006 which caused 39 deaths.

4 Case Studies

The North Anatolian Mountains generally parallel the Black Sea coast. In the west, the mountains tend to be low, with elevations rarely exceeding 1500 meters, but they rise in an easterly direction to heights greater than 3000 meters south of Rize. Rivers in the Black Sea region flow through narrow valleys. Therefore, big flood disasters can happen without very high discharge.

In the Eastern Black Sea region, Rize Flood on July 23, 2002 caused 27 people death, 18 of them died under one building collapse. In this region, three-day (July 23, 24 and 25) rainfall totals were 195.7 kg in Pazar (Rize), 116.8 kg in Rize, 63.1 kg in Hopa (Artvin), 35.9 kg in Ordu. The measured rainfall amount was 107.5 kg in Pazar

Town of Rize during one-night. This value is equal the amount of one-month total rainfall [3]. The detailed information of the rainfall before Rize flood of July 23, 2002, is given in Table 3.

Table 3. The characteristics of precipitations of the Rize Flood [2]

Station Name : Rize						
Date	Start	Finish	Duration (min)	Amount (mm)	Intensity (mm/min)	Specific Flow Produced (L/sec/ha)
23/7	6:12	6:17	5	12.8	2.56	426.7
23/7	6:09	6:19	10	17.1	1.71	285
23/7	6:09	6:24	15	21.3	1.42	236.7
23/7	5:54	6:24	30	28.3	0.943	157.2
23/7	5:49	6:49	60	35.4	0.59	98.3
23/7	5:02	7:02	120	36.1	0.301	50.1
23/7	4:02	7:02	180	36.1	0.201	33.4
23/7	3:02	7:02	240	36.1	0.15	25.1

In Table 3, duration, amount, intensity, and specific flow produced by this rainfall are presented with respect to the time of start and finish of the rain storm. The measured discharge at Guneysu Creek was 450 m^3/sec with a recurrence interval Tr=500 years (Table 4).

Table 4. Measured discharge value of Creeks in Rize [Modified from 4]

Date	Loss of life	Inundated Area (decares)	Monetary Loss ($)	Debi (max) Creek	Q(m^3/sec)	T(yıl)
24.07.2002	27	-	7789000	Senoz	57	5
				Salarha	175	10
				Guneysu	450	500
				Asiklar	100	70
				Sairler	130	100

Trabzon Flood on August 2, 2005, caused 9 deaths. The intense rainfall occurred at Of and Caykara towns, and around of Trabzon (Figs. 4 and 5). The detailed information of the rainfall about Trabzon Flood on August 2, 2005 is given in Table 5.

The flood damaged the infrastructure, roads, cultivated land and buildings, with a total damage of $ $25*10^6$. The discharge values of August 2, 2005 flood event are given in Table 5. The flow records of Cevizli Gauging Station (22-06) were not included in Table 3 due to flood damage on gauge, but when the creek reached to the sea, the discharge was estimated 400 m^3/sec with recurrence interval was estimated as Tr=30 years. Solakli discharge value given in Table 6 is the maximum measured value during the 25-year observation period.

Table 5. The characteristics of precipitations of the Trabzon Flood[2]

Station Name : Trabzon						
Date	Start	Finish	Duration (min)	Amount (mm)	Intensity (mm/min)	Specific Flow Produced (L/sec/ha)
2/8	7:04	7:09	5	8.4	1.68	280
2/8	7:04	7:14	10	9	0.9	150
2/8	7:04	7:19	15	15.1	1.007	167.8
2/8	6:54	7:24	30	19.2	0.64	106.7
2/8	6:49	7:49	60	30.9	0.515	85.8
2/8	5:59	7:59	120	43.1	0.359	59.9
2/8	5:59	8:59	180	45	0.25	41.7
2/8	5:59	9:59	240	45.9	0.191	31.9
2/8	5:59	10:59	300	46	0.153	25.6
2/8	5:59	11:59	360	46	0.128	21.3
2/8	5:59	13:59	480	46	0.096	16

Table 6. Measured discharge value of Creeks in Trabzon on August 2, 2005[5]

Creek Name	Gauging Station Number	Day	Hour	Discharge (m^3/sec)
Solakli Creek	22-52	02.08.2005	09.30	200
Manahoz Creek	22-53	02.08.2005	09.30	190

Fig. 4. Caykara Kumlu Village[5]

Fig. 5. Caykara road (after the flood)[6]

Fırat and Dicle rivers' system experiences mainly snowmelt floods. Actually this is the largest drainage basin of Turkey and about 5 times larger than Eastern Black Sea region. In this basin, snowmelt induced floods occur mainly in April, May and even in June. The flood on November 2, 2006 at Southeastern Anatolia, chosen as the case study was due to the heavy rainfall in and around Batman (Table 7). In a similar way, duration, amount, intensity, and specific flow produced by this rainfall are presented during the rain storm in Table 7. In this flood 39 people lost their lives, 3800 houses damaged and 365000 decares of agricultural lands flooded (Figs. 6 and 7). The disaster has the biggest financial damage as the flood hazard up to the present in Turkey with a total loss of $ 300·10^6 it was the biggest flood damage in Turkey.

Table 7. The characteristics of precipitations of the Batman Flood [2]

Station Name: Batman						
Date	Start	Finish	Duration (min)	Amount (mm)	Intensity (mm/min)	Specific Flow Produced (L/sec/ha)
1/11	19:37	21:37	120	26.6	0.222	36.9
1/11-2/11	17:54	5:54	720	54.8	0.076	12.7

Fig. 6. Batman city center [7]

Fig. 7. Bitlis city (after the flood) [7]

5 Structural And Non-Structural Flood Protection Measures

The structural flood protection measures in Turkey are classified as follows:

1. Construction of water impoundment structures (dams, artificial lakes, etc.) mainly in upstream parts of drainage areas;
2. Construction of transverse structures such as weirs, drop structures, chutes, etc. to dissipate the kinetic energy of the supercritical flow of mountain rivers, most of which are torrential in character;
3. Construction of longitudinal structures to define the flow path, and to protect river banks and areas on the banks of rivers;
4. Stream bed modification such as change in the existing river bed and setting up new diversion structures. Dikes and groins are economical, but they are temporary solutions;
5. Reforestation and improvement of land use techniques, such as terracing, strip cropping and enrichment of the existing vegetation, will decrease the velocity of the sheet flow into the river channel and minimize erosion and the amount of sediment transported into rivers and to the sea.

The non-structural flood protection measures can be summarized as:

1. Education of the local people about the reasons of flooding, and the probable magnitude of damage;

2. Convincing the people of using the flood plains for recreational purpose during dry seasons;
3. Setting up first-aid teams made up of local people to help the victims in remote locations immediately where state help may come very late due to damaged roads is necessary in remote but flood-prone areas.

6 Conclusions

An inventory of 2089 floods throughout Turkey was prepared using available data for the period 1955 to 2009. The costliest flood damage took place in 2006, a total of \$ $300 \cdot 10^6$. When the number of flood events is classified according to maximum discharge, a distinct peak appears in 1968, and there is a decreasing tendency toward the end of 1994. Floods in western Turkey and in the coastal zones mainly are produced by heavy rainfall in combination with geomorphologic features; however in the central and eastern parts of Anatolia snow accumulation has an important role. To minimize the adverse effects of floods on communities and other types of flood damage, a comprehensive flood management program is required for each newly proposed development zone. It is also important to have good coordination among both partners; the state, local people. In the Black Sea region people settle in very narrow flood plains because the mountains generally parallel the Black Sea coast and the settlement plains are very limited due to the narrow valley. Because of the economic situation of Turkey, and the approach of the country to tackle with the natural disasters is relocation of the disaster-hit people, it is hardly possible to answer the demand of the people at once because at present The Turkish State owes a large number of houses to the disaster-hit people. In the Eastern Black Sea region the land for resettlement is very limited but in the Southeastern Anatolia, the resettlement of the flood-hit people may be possible. The local people expect the state to take into consideration their traditional way of living when providing new settlements.

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International Symposium on Water Management and Hydraulic Engineering

Ohrid/Macedonia, 1-5 September 2009

Paper: **A110**

Hydrodynamic Pressure on Rigid Vertical Dams with Two Segmented Bed During Earthquakes

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Abstract. In this paper the hydrodynamic pressures acting on the surface of a rigid dam during earthquakes are examined analytically. The equations for the earthquake forces on a rigid dam are solved exactly by two-dimensional potential-flow theory; that its reservoir bottom is two segmented each with constant slopes. Analytical solution is obtained for the cases where the upstream dam face is vertical, inclined bottom of the reservoir has a constant slope and thus the reservoir has a trapezoidal shape. This solution is compared with experimental data and numerical solutions with good agreement.

Keywords*:* Hydrodynamic pressure, vertical upstream dam face, two segmented bed, analytical solution, rigid dam, conformal mapping.

1 Introduction

Earthquake-induced hydrodynamic pressures on upstream face of a dam are important factors in design consideration. Assuming that the fluid is incompressible, Westergaard (1933) was first to derive an expression for the hydrodynamic pressure acting on a rigid dam with a vertical upstream face as a result of horizontal harmonic ground motion [15]. In the last fifty years, many researches have extended Westergaard's classical work to include more physical parameters such as the compressibility of the fluid in the reservoir (e.g. Chopra [2]; Rashed & Iwan [13]), the flexibility of the dam (e.g. Finn & Varoglu [7]; Mei, Foda & Tong [12]), and reservoir bottom absorption (e.g. Fenves & Chopra 1983, 1985). Although there are many numerical models (e.g. Hall & Chopra [8]; Rashed [14]; Liu & Cheng [10]; Yeh & Ho [16]) that can be used for dam-reservoir-interaction problems with complex two and three dimensional geometries, analytical solutions are rare and available only for a reservoir with a simple geometry.

Using a two-dimensional potential-flow theory, Chwang (1978) presented an analytical solution for the hydrodynamic pressure on an accelerating dam [3]. The duration of the dam acceleration is short enough that the compressibility of the fluid can be ignored. He supposed that the dam has an inclined upstream face with constant slope and the reservoir bottom is horizontal. Liu (1986) distributed Chwang's solution with this suppose that the reservoir bottom is inclined with constant slope (reservoir has a triangle shape) [11].

In this paper we assumed that the total of the upstream dam face, reservoir bottom and the free surface of water have formed a trapezoidal shape with vertical upstream dam face and horizontal bed at first (Its comforts dam construction at that place) and inclined bed with constant slope to infinity (river natural slope) (see Fig. 1).

The objective of this paper is presenting an analytical solution for the earthquake-acceleration induced hydrodynamic pressure on a rigid dam with two segmented reservoir bottom for simulating natural and actual conditions.

2 Theoretical Formulation

Consider a rigid dam with a vertical face (Γ_d in Fig. 1). The origin of the coordinates is located at the base of the dam and the free surface is represented by $y = h$. As shown in Fig. 1, the bottoms of the reservoir are denoted as Γ_{b1} , Γ_{b2} that the horizontal part length is l. The dam and reservoir system is assumed to undergo a constant acceleration a_0 in the direction making an angle γ with the x-axis. Assuming that the fluid in the reservoir is incompressible and inviscid, the hydrodynamic pressure P satisfies the Laplace equation

$$\nabla^2 \mathrm{P} = 0 \tag{1}$$

If the duration of the acceleration is sufficiently short the free surface perturbation shall be small and negligible. The only free surface boundary condition is to require that the dynamic pressure vanish. Thus

$$P = 0 \quad (\mathrm{y} = \mathrm{h}) \tag{2}$$

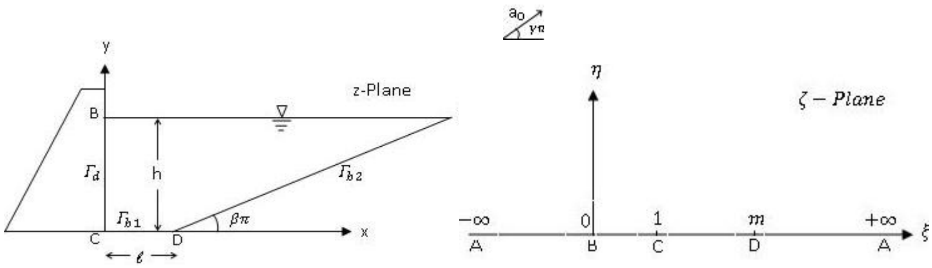

Fig. 1. A simplified dam-reservoir system. The reservoir occupied by the water in the z-Plane is mapped onto the upper half of the ζ -Plane.

Along the upstream dam face and the reservoir bottom the normal derivations of the dynamic pressure are prescribed:

$$\frac{\partial P}{\partial n_d} = -\rho a.n_d \quad \text{on } \Gamma_{d;} \tag{3}$$

$$\frac{\partial P}{\partial n_{b1}} = -\rho a.n_{b1} \quad \text{on } \Gamma_{b1;} \tag{4}$$

$$\frac{\partial P}{\partial n_{b2}} = -\rho a.n_{b2} \quad \text{on } \Gamma_{b2;} \tag{5}$$

where: $a = (a_0 \cos\gamma, a_0 \sin\gamma)$, and n_d and n_b are the unit outward normals along the dam surface and the reservoir respectively. In the case where the water depth is a constant, the reservoir becomes a semi-infinity long channel. The hydrodynamic pressure vanishes as $x \to \infty$.

3 Analysis

As depicted in Fig. 1, the upstream dam face is assumed to be a vertical line,

$$x = 0 \quad \text{on } \Gamma_{d;} \tag{6}$$

The reservoir bottoms are horizontal and inclined lines and are described as:

$$y = 0 \quad \text{on } \Gamma_{b1;} \tag{7}$$

$$y = \tan\beta\pi(x - l) \quad \text{on } \Gamma_{b2;} \tag{8}$$

where $0 \le \beta \le \frac{1}{2}$. When $\beta = 0$, the reservoir depth becomes constant and the reservoir domain extends to infinity ($x \to \infty$).

The boundary conditions along the solid surfaces can be reduced from (3) to (8) to the following:

$$\frac{\partial P}{\partial n_d} = -\rho a_0 \cos\gamma\pi \quad \text{on } \Gamma_d, \tag{9}$$

$$\frac{\partial P}{\partial n_d} = -\rho a_0 \sin\gamma\pi \quad \text{on } \Gamma_{b1}, \tag{10}$$

$$\frac{\partial P}{\partial n_d} = -\rho a_0 \sin(\beta - \gamma)\pi \quad \text{on } \Gamma_{b2}. \tag{11}$$

Introducing the complex-conjugate function q with respect to P, we can construct an analytical function:

$$W(z) = p + iq, \quad z = x + iy. \tag{12}$$

Using the Cauchy-Riemann relation $\partial P/\partial n = \partial q/\partial s$, the boundary conditions (9) to (11) can be re-written as:

$$q = -\rho a_0 s \cos \gamma\pi \quad \text{on } \Gamma_d, \tag{13}$$

$$q = -\rho a_0 s' \sin \gamma\pi \quad \text{on } \Gamma_{b1}, \tag{14}$$

$$q = -\rho a_0 s'' \sin(\beta - \gamma)\pi \quad \text{on } \Gamma_{b2}, \tag{15}$$

Where s measures distance from the origin of the coordinates system (point B in Fig. 1) to any point on the upstream face of the dam and s' and s'' represent the distance along the reservoir bottom measured from the origin in horizontal and inclined parts, respectively.
Using the conformal mapping:

$$z = K \int_1^{\zeta} t^{-1/2} (t-1)^{-1/2} (t-m)^{-\beta}\, dt; \tag{16}$$

Given by the Schwarz-Christoffel theory, we can transform the reservoir fluid region in the physical plane into the upper half of the ζ -plane $(\zeta = \xi + i\eta)$ (see Fig. 1). Points B, C and D are mapped into ζ =0, 1 and m respectively $(m > 1)$. Point A is mapped into points at infinity in the ζ -plane along the negative and positive axes.

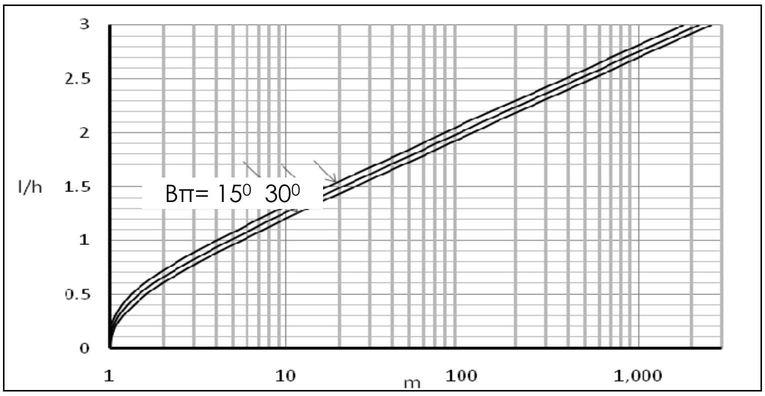

Fig. 2. The variation of m with horizontal dimensionless distance l/h for several inclination angles $\beta\pi$

The complex constant K in (16) is determined by:

$$K = \frac{-hi}{\int_0^1 t^{-1/2}(t-1)^{-1/2}(t-m)^{-\beta}\,dt}, \tag{17}$$

where the parameter m in (16) and (17) depends on length (l), reservoir height (h) and slope of reservoir bottom (β). The variation of m with horizontal distance l/h for several bottom slopes $\beta\pi$, is obtained by numerical integration and shown in Fig. 2. From the equations (2), (12), (13), (14) and (15) we get the following relations along the real axis of the ζ -Plane:

$$\operatorname{Re} W(\zeta) = 0 \qquad (-\infty < \xi < 0), \tag{18}$$

$$\operatorname{Im} W(\zeta) = \begin{cases} -\rho a_0 s(\xi)\cos\gamma\pi & (0 < \xi < 1), \quad (19) \\ -\rho a_0 s'(\xi)\sin\gamma\pi & (1 < \xi < m), \quad (20) \\ -\rho a_0 s''(\xi)\sin(\beta-\gamma)\pi & (m < \xi < \infty), \quad (21) \end{cases}$$

where s, s' and s'' are given in (16) as:

$$s = Ki\int_\xi^1 t^{-1/2}(t-1)^{-1/2}(t-m)^{-\beta}\,dt \qquad (0 < \xi < 1), \tag{22}$$

$$s' = K\int_1^\xi t^{-1/2}(t-1)^{-1/2}(t-m)^{-\beta}\,dt \qquad (1 < \xi < m), \tag{23}$$

$$s'' = ke^{-i\pi\beta}\int_m^\xi t^{-1/2}(t-1)^{-1/2}(t-m)^{-\beta}\,dt + l \qquad (m < \xi < \infty). \tag{24}$$

Equations (18), (19), (20) and (21) are mixed-type boundary conditions for W(ζ). However if we introduce an auxiliary function:

$$H(\zeta) = \zeta^{-1/2}W(\zeta), \tag{25}$$

where the positive branch is taken for the square-root function, the boundary conditions (18) to (21) change to:

$$\operatorname{Im} H(\zeta) = \begin{cases} 0 & (-\infty < \xi < 0), \quad (26) \\ -\rho a_0 \xi^{-1/2} s(\xi)\cos\gamma\pi & (0 < \xi < 1), \quad (27) \\ -\rho a_0 \xi^{-1/2} s'(\xi)\sin\gamma\pi & (1 < \xi < m), \quad (28) \\ -\rho a_0 \xi^{-1/2} s''(\xi)\sin(\beta-\gamma)\pi & (m < \xi < \infty). \quad (29) \end{cases}$$

Using the Riemann-Hilbert theory (e.g. Carrier, Krook & Pearson 1966 [1]), we can express the analytic function $H(\zeta)$ in the upper-half ζ-plane as

$$H(\zeta) = \frac{1}{\pi}\int_{-\infty}^{\infty} \frac{\operatorname{Im} H(\zeta) d\xi}{\xi - \zeta}. \tag{30}$$

Substitutions of (26), (27), (28) and (29) into (30) yield:

$$\begin{aligned} W(\zeta) = \frac{-\rho a_0}{\pi}\zeta^{1/2}\Bigg\{ &\cos\gamma\pi \int_0^1 \frac{s(\xi)d\xi}{\xi^{1/2}(\xi-\zeta)} \\ &+ \sin\gamma\pi \int_1^m \frac{s'(\xi)d\xi}{\xi^{1/2}(\xi-\zeta)} + \sin((\beta-\gamma)\pi)\int_m^{\infty} \frac{s''(\xi)d\xi}{\xi^{1/2}(\xi-\zeta)} \Bigg\}. \end{aligned} \tag{31}$$

The hydrodynamic pressure on the upstream dam face is the real part of $W(\zeta)$ for $0 < \xi < 1$. Using (22), (23) and (24) in (31) and integrating by parts, we obtain:

$$\begin{aligned} P(\xi) = \frac{K\rho a_0}{\pi}\Bigg\{ & i\cos\gamma\pi \int_0^1 t^{-1/2}(t-1)^{-1/2}(t-m)^{-\beta} \ln\left|\frac{t^{1/2}+\zeta^{1/2}}{t^{1/2}-\zeta^{1/2}}\right| dt \\ & + \sin\gamma\pi \int_1^m t^{-1/2}(t-1)^{-1/2}(t-m)^{-\beta}\left(\ln\left|\frac{m^{1/2}+\zeta^{1/2}}{m^{1/2}-\zeta^{1/2}}\right| - \ln\left|\frac{t^{1/2}+\zeta^{1/2}}{t^{1/2}-\zeta^{1/2}}\right|\right) dt \\ & - \sin((\beta-\gamma)\pi)e^{-i\beta\pi}\int_m^{\infty} t^{-1/2}(t-1)^{-1/2}(t-m)^{-\beta} \ln\left|\frac{t^{1/2}+\zeta^{1/2}}{t^{1/2}-\zeta^{1/2}}\right| dt \Bigg\} \\ & - \frac{\rho a_0}{\pi} l \sin((\beta-\gamma)\pi)\ln\left|\frac{m^{1/2}+\zeta^{1/2}}{m^{1/2}-\zeta^{1/2}}\right| \qquad (0 < \xi < 1), \end{aligned} \tag{32}$$

where $\int$ denotes the Cauchy principal value.

We remark that the analytical solutions presented by Chwang [3, 4] are the special case of (32) with $\beta = \gamma = l = 0$.

4 Conclusions

Hydrodynamic pressures are only calculated for horizontal ground acceleration (i.e. $\gamma = 0^0$). The vertical ground acceleration simply modifies the gravitational acceleration. Therefore the corresponding hydrodynamic pressure values linearly in the vertical direction independently of the geometries of reservoir and dam [4, 11].

The hydrodynamic pressure distributions on the upstream dam face are plotted vs. the vertical distance y/h for several horizontal distances l/h in Figs. 3-6. In each figure, the hydrodynamic pressure is given for several inclination angles $\beta\pi$.

For $l/h = 0$, hydrodynamic pressure agrees with Chwang's [3] and Liu's [11] solutions when upstream face of the dam is vertical $(\alpha\pi = 90^0)$. Moreover, when the water depth is constant $(\beta\pi = 0^0)$ the present solution reduces to Chwang's results [3].

From the Figs. 3-6, it is clear that the hydrodynamic pressure decrease as $\beta\pi$ increases. Therefore the pressure distribution for $\beta = 0$ (regardless of the bed horizontal distance l/h) is the maximum envelope of all pressure distributions and it occurs at the base of the dam.

Also from the Figs. 4-6, it is clear that the hydrodynamic pressure increases as l/h increases except $l/h = 0$ that it greater than $l/h = 0.5$ and this shows that in $l/h < 1$ value of the hydrodynamic pressure is lesser than $l/h = 0$.

Because the upstream face of the dam is vertical, the maximum pressure occurs at base of the dam in all figures. Moreover it is recognized from the Figs. 4-6, that values of l/h are effective in hydrodynamic pressure. But when $l/h > 3$ (bed horizontal distance three times greater than maximum water depth) we can approximately suppose that the bed inclination doesn't affect the hydrodynamic pressure induced on vertical upstream dam face.

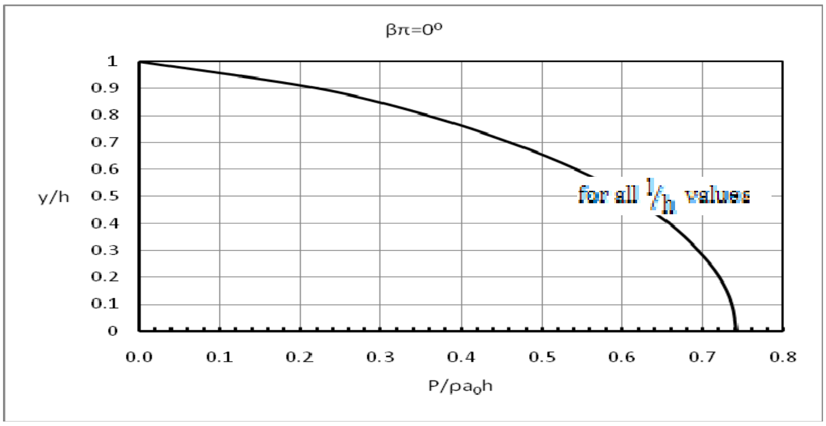

Fig. 3. The pressure distributions on the upstream dam face when the water depth is constant

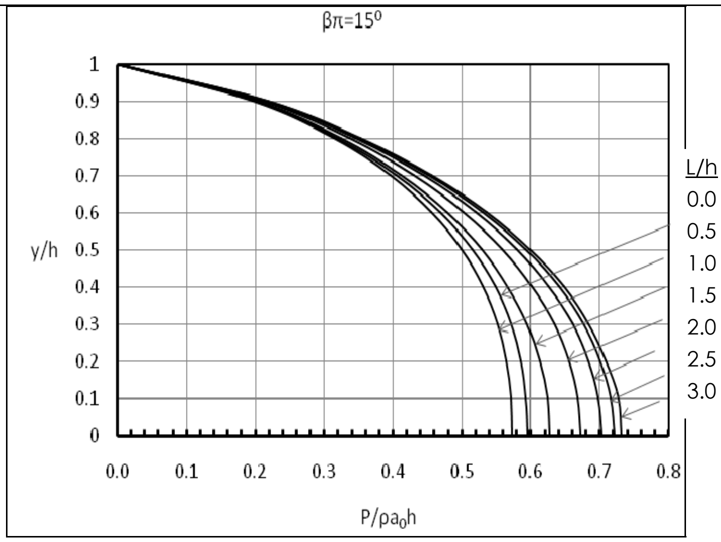

Fig. 4. The pressure distributions on the upstream dam face for different l/h

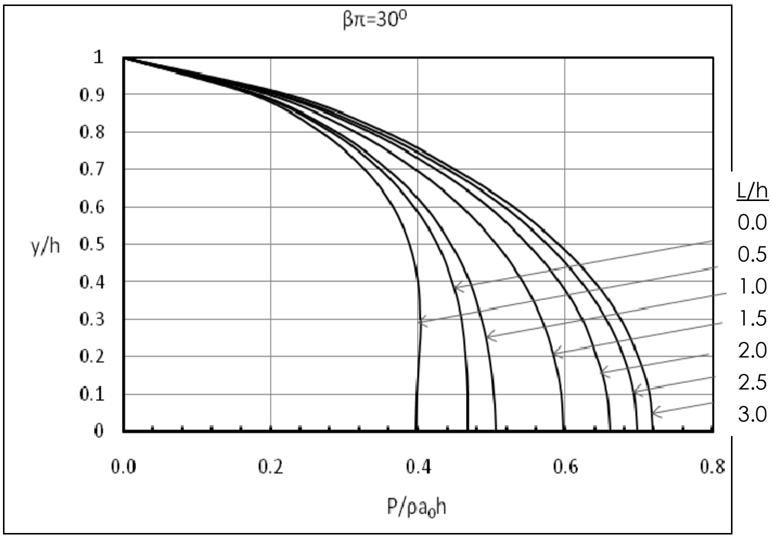

Fig. 5. The pressure distributions on the upstream dam face for different l/h

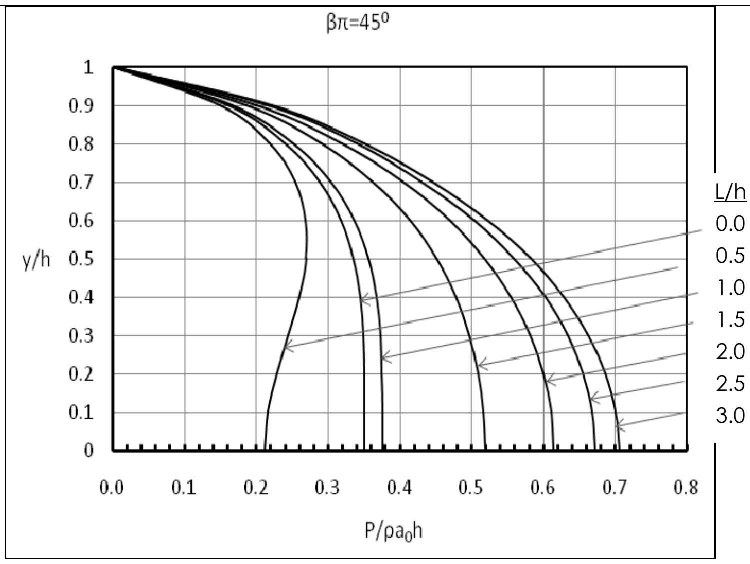

Fig. 6. The pressure distributions on the upstream dam face for different l/h

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Topic 2

Sanitary Engineering and Sustainable Water Use

International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A04**

Sewer Flow Monitoring

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Abstract. Measuring of discharge in the sewage system of town in fact are measures the flow in particular canals of system, i.e. in suitable points of network. Based on such measurements was able reconstruct the individually charge of parts of systems. Such suitable points find mostly on collection supply channels or pipes from some area and downstream of couplings of larger canals, as well as on outlet canals, and on network components where has changed the section of canal and the where possible approach shaft. How it is about the liquid, the more or less burdened suspension and floating substances, often very aggressive, measuring instruments as well as the monitoring must is adapted such conditions. Generally, instruments have been conceived so that measure level and average velocity of water in the profile in programmed time intervals, then based on geometry of profile calculate the flow or discharge of water passed main sewer. But exists and another methods of monitoring depending on hydraulic conditions of channel network.

Key words: Flow measurement, hydraulics condition, methods of sewer flow monitoring

1 Introduction

This paper wants to give a brief outline about hydraulics as well as the most common flow measurement techniques and what should be taken into account. Flow measurement in full-filled pressure pipes normally is a routine task that is easily performed. The difficulties begin in:

- Increasing accuracy requirements
- Large pipes/channels with irregular profiles, low flow levels and flow velocities
- Partial filling with backwater

The paper will present and explain basic approaches measuring the discharge in sewage or sewer canals. It will be describe necessary requests and explanations of concept of particular measurement method like:

- Measurement Principles tracer and with propeller
- Weirs and Flumes
- Magnetic Inductive Flow Measurement
- Radar method
- Ultrasonic Pulse-Doppler method

Comparative analysis of advantage and disadvantage of particular measurement method will be presented.

2 Requirements for Flow Measurements in Channel Systems

Flow Measurements in sewer systems must be suitable for specific flow hydraulics and geometric boundary conditions. Channels often are dimensioned to convey off high water volumes at heavy rainfall, while the water volume by night or in dry weather periods may be very low.

Measurement devices for such applications must yield usable measurement values under conditions of high water volumes, backwater, low water levels and low flow velocities. Complex channel profiles, stream turbulences caused by feed openings, manholes and arcades must be taken into account. This task is highly demanding for the used measurement technique.

3 Background-History

The water and waste management practices of the olden times required great skills of the engineers for its performances. The old Romans had already built such plants and recognized that: The Flow **Q** cannot be measured by only measuring the fluid height.

Fig. 1. Romans aqueduct

The simplest measurement method for investigating flow consists of measuring the fluid height in a channel with defined geometry.

Flow Q is a function of Q/h, the slope J and the roughness coefficient k. The roughness coefficient is determined experimentally and is dependent on the material types and the age of the material. For example, a concrete channel will have a different roughness from the original due to coating of the channel by grease and slime. This will cause a smoother surface and thus a different measurement value.

4 Free Flows - Manning Principle

Robert Manning (1816–1897) was born in Normandy, but he lived in Ireland. He wrote many papers on hydraulics. During living period, Manning devoted considerable effort to the development of a simple, dimensionally homogeneous formula for open-channel flow. His paper "On the Flow of Water in Open Channels and Pipes" (1891) became the primary reference for his work and the source of Manning's monomial equation:

$$v = \frac{K_n}{n} R^{\frac{2}{3}} J^{\frac{1}{2}} \qquad (1)$$

where:

v	cross-sectional average velocity
R	hydraulic radius
J	energy slope
K_n	1 for SI units and 1,486 for English units
N	Manning resistance coefficient

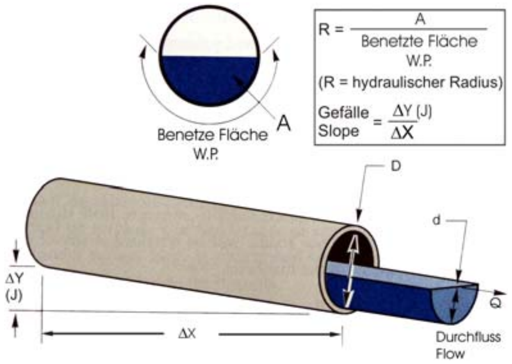


Fig. 2. Partially filled pipes flow

Similar conditions are given for pipes with the exception that, unlike in channels, Q might be smaller in full filled pipes than in partially filled pipes.
When calculating full filled pipes generally the flow velocity v (m/s) or the possible flow Q (l/s) is investigated at a certain slope (J) and a known diameter (DN). Besides the slope (1:x or ‰) and other dissipation coefficients the values Q and v are influenced especially by the resistance coefficient according to Prandtl-Colebrook.

The trade-specific, computer-based calculation systems as well as calculation by tables cannot handle dissipation coefficients of single components like e.g. manholes. Thus, in practical applications we calculate with the operational roughness value k_b.

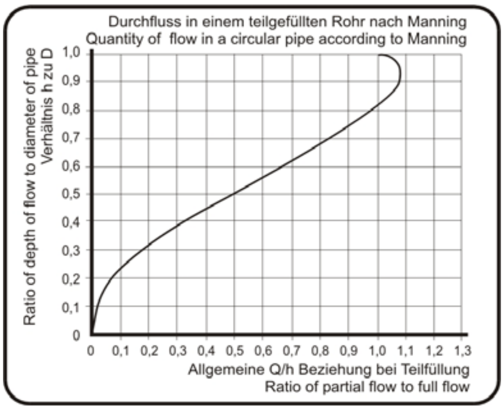


Fig. 3. Graph for correction flow in circular partially filled pipe (according to Manning)

5 Explanation of Concepts

The modern electronic methods determining flow Q (m^3/s) measure the average flow velocity **v** (m/s) depending on the wetted cross section A (m^2).

$$\mathbf{Q} = \mathbf{v}_{\text{average}} \mathbf{A} \ (m^3/s) \tag{2}$$

Average Flow Velocity. The velocity in flowing water isn't evenly distributed. Therefore the average flow velocity is used for calculation of Q. This is the product of the cross-sectional area A and the average flow velocity.
Flow can be determined by the product of the average flow velocity and the wetted cross-sectional area of the flow. For cross-sectional area, one must know the exact geometrical data of the channel. For partially-full channel, the level must be determined additionally. If there are going to be deposits or silting in the channel, then this must be taken into account too.

Flow Profiles. Sufficiently low flow velocities give rise to laminar flow. This is represented as stratified flow in a physical condition. The single water layers glide over each other without any mixing. By means of the frictional strength (roughness of the walls, viscosity of the medium), the flow velocity at the walls is 0. For full-filed pipes, for example, the maximum flow velocity is in the middle of the pipe.
Transition flows are intermixing of the laminar and turbulent flows. These forms of hydraulic flows are unstable and swinging. It cannot develop a defined, stable flow profile. The flow profiles cannot be assessed.

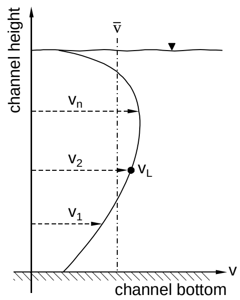


Fig. 4. Flow velocity profile

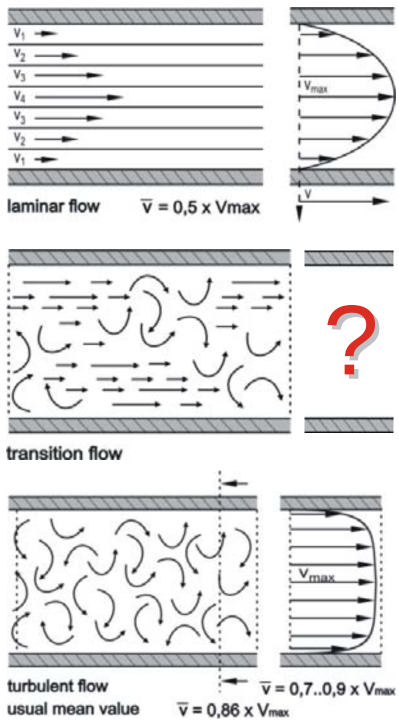


Fig. 5. Flow velocity profile

Increase in flow velocities lead to turbulent flows. In turbulent flows, one finds an intermixing of the single water layers. The wall roughness has little influence, and the velocity profile is uniformly shaped.

A characteristic value to describe streaming conditions is the dimensionless Reynolds value “Re” (Re= v·d /cinematic viscosity). The critical Reynolds value for liquids is around 2320.

Beyond this value, the streaming condition may switch from laminar to turbulent.

The ideal shapes of the different flow profiles from flow measurement equipment.

General problems of the velocity measurements in liquids

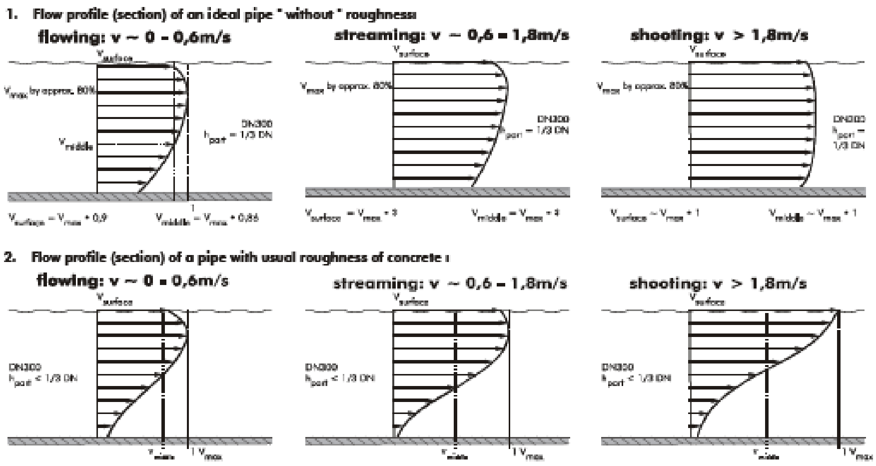


Fig. 6. Different flow velocity profiles

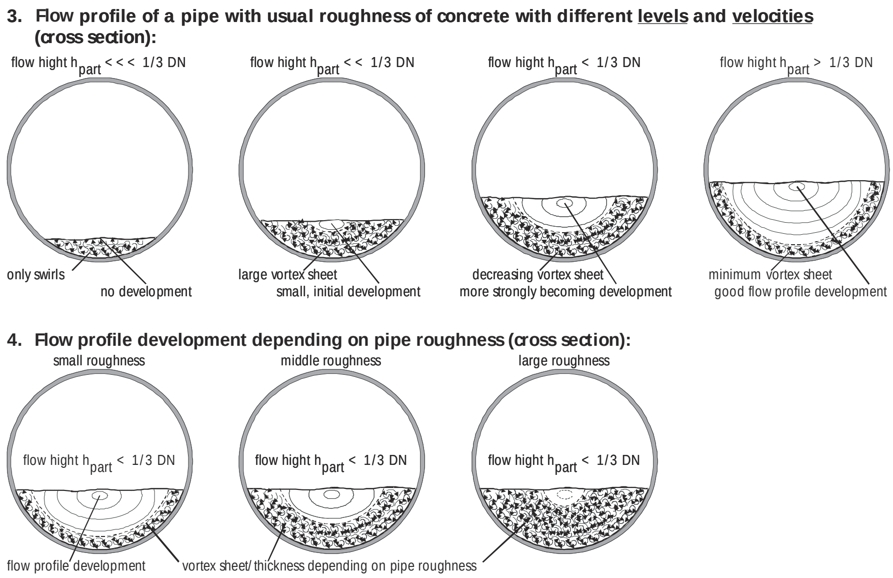


Fig. 7. Different development of the flow profiles

6 Tracer Dilution Methods

Basically we distinguish between splash input and continuous input. One of the methods is the dilution method. Here a dissolved marker medium (dyes, salt, radioactive tracers) in a defined amount and concentration is brought in. Above and below the point of insertion from a measured magnitude (light absorption, electric conductivity, activity) to the resulting background or overall concentration of the tracer is concluded. From that known magnitudes the flow is calculated.

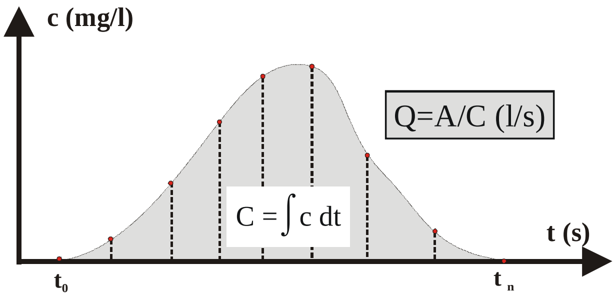


Fig. 8. Method and device for measurement

Splash Input: A small amount of salt is brought into the water above the point of measurement. At this point the salt concentration is constantly detected as the salt cloud passes the sensor. As soon as the measurement device detects the end of the cloud the user is asked to finish the measurement. The flow rate is displayed immediately. The flow rate is calculated with the following formula: Q=A/C (l/s), where Q is the flow rate in (l/s); A is the known salt quantity; C is corresponding measured concentration of the salt cloud.
Advantages: no measurement of channel geometry.
Disadvantages: a complete mixing is necessary.

7 Velocity Measurements with Propeller

Flow velocity measurement can be approximated with floats. The velocity of the water can be determined with the time of the float over a defined distance. Often used for the measurement of the velocity is the hydrometric propeller. From the rotations of the propellers at different points and in different heights of the cross section of the flow, the average velocity can be determined from the rating curve. The point measurements have to cover sufficiently the cross section of the flow. The flow is again determined with the average current velocities of the cross section. Flow measurements are used to calculate the volume of the flow at the time of the measurement. The relationship between water level (W) and flow (Q) can be determined by measuring various times. With this various values a W/Q rating curve can be developed.

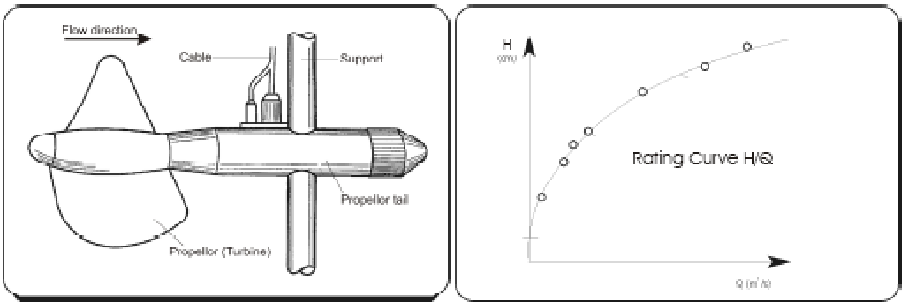


Fig. 9. Device for measurement and rating curve

Advantages: Very simply determining the flowing.
Disadvantages: Measure with the hydrometric propeller are possible only in relatively clean water, without of the bigger suspensions.

8 Weirs

Weirs are hydraulic object for the measurement of discharge in open canals. It is possible use a different types and forms of a weir, depending of the hydraulic characteristic of measuring point. In everyone is the necessary measure the level of the water H (cm) on weir.

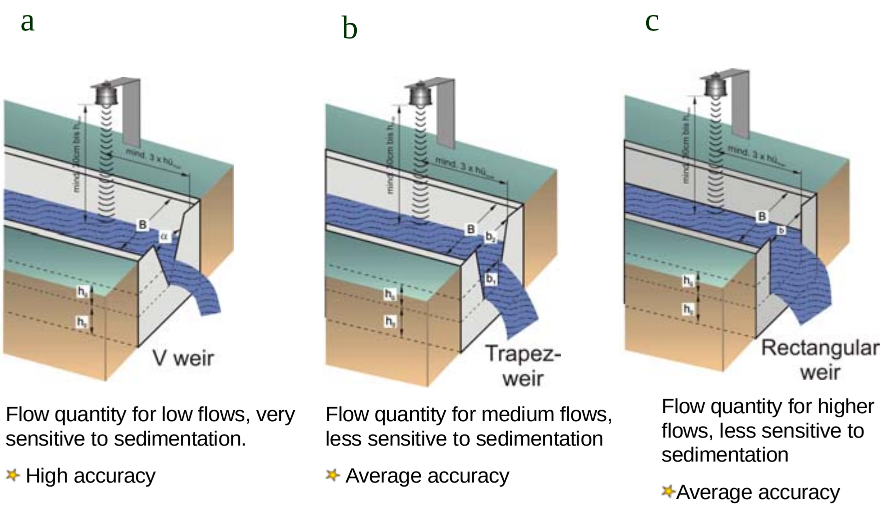
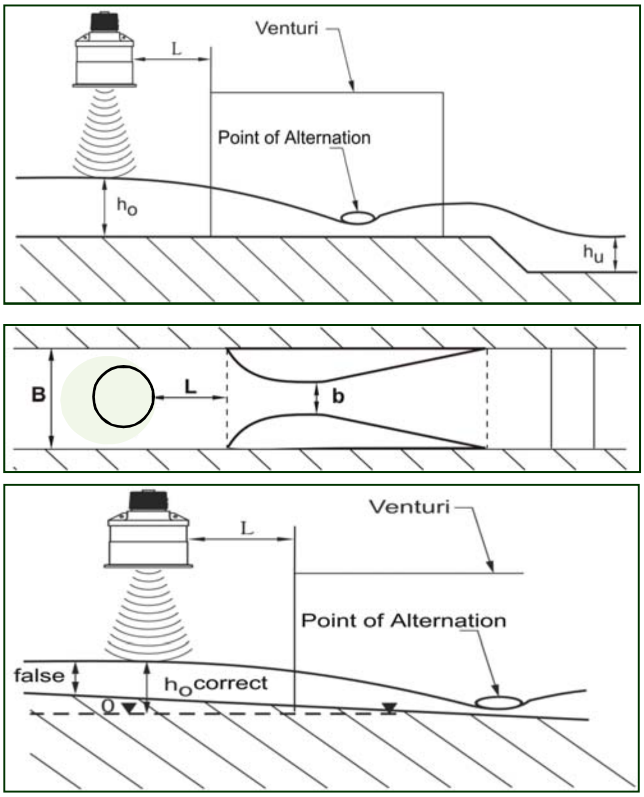


Fig. 10. Different weir forms

9 Flumes

Flumes are flow measurement devices that are specially formed for channels and have a defined lateral narrowing. A flow change of the current takes place in this narrowing from free flow to fast flow. Relative flume measurements are done on stationary, pre-fabricated forms, but also concrete flumes can be constructed in the channel. The geometric shapes vary depending on the measurement method (e.g. Parshall Flume, Palmer-Bowlus Flume, and many other forms manufactured).



B = Channel width L1 = Distance from transducer

H_o = High water level H_u = Sub-water level

Fig. 11. Principles of measurements

More detailed information for the installation conditions, stilling well section, measurement range, preferred accuracy etc. must be known.

Basic Operation: An accumulation of the medium to be measured occurs due to the restriction in front of the flume. A defined transitional flow takes place at the throat of the flume and the medium shoots through (hydraulic jump). Due to this defined hydraulic condition, it is possible to ignore the roughness and slope of the flume. The impounded height in front of the flume is directly related to the flow measurement in the flume.

The following must be noted when using a flume for measurements: The least measurable quantity of raw sewage is about 5 l/s. The ratio Q_{max} to Q_{min} lies at 10:1; for special flumes, it is maximum 20:1. It is important that the measurements are made without any back-flow condition. This means that higher water level h_o must not be lower than the sub-water level h. The measurement transducer must be 1.5-2 $h_{o\ max}$ above the start of the installed throat (L). The flow quantity should be in the maximum possible range, and the water level raised (3-4)h_o. The zero point measurement of the transducer is always the zero point of the flume, not the zero point of the measurement place.
Advantages: measurement relative exact, easy to control, suitable for each medium
Disadvantages: no measurement under backwater condition, low measurement dynamics.

10 Magnetic Inductive Flow Measurement (MID)

The magnetic inductive flow measurement is based on Faraday‘s Law of Induction. A homogeneous magnetic field is built up in the pipe. The flowing liquid induces an electric current coupled with the magnetic field in the pipe, which acts like an electric wire (e.g. copper wire). This current measured by the electrode, is directly proportional to the average flow velocity.

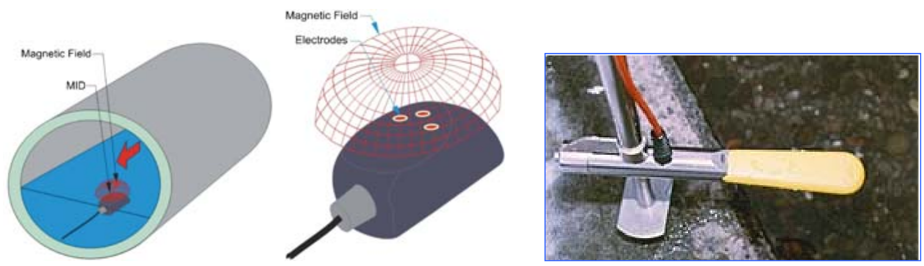


Fig. 12. Principles and device of measurement

Partially full MID´s operate on the similar principle as the full filled MID. In contrast to fulfill MID, the induced current over several electrode pairs differs with differing heights. The magnetic field structure becomes the actual flow height evaluated. The characteristics of the magnetic fields at various heights are stored in the measurement transmitter. By means of correction methods depending on the actual flow height, a signal is produced that is proportional to the flow.

The Magnetic Inductive Single sensor, generally shaped like a „Channel Mouse“, can be used in wastewater channels and pipes. Similar to the traditional MID, the current is induced on the electrodes of the sensor. The level is measured by a pressure transducer at the bottom of the channel, or by an external level measurement device. The instrument can be used in both, temporary and permanent applications.

*Advantages***:** No disruptive fittings, measures on surfaces; requires no reflective particles or gas bubbles in water, easy mounting.

Disadvantages: No measurement under 10% of filling, not very accurate, measurement errors with decreasing flow heights,_measures only conductive fluids >10mS, isolating coatings on electrodes introduces errors, high power requirement, expensive with increasing diameter of a pipe.

11 Radar Flow Measurements

Alexander Watson-Watt (1892-1973) was the Scottish physicist who developed the radar locating of aircraft in England. He was born in Brechin, Angus, Scotland, educated at St Andrews University in Scotland, and taught at Dundee University. Radar was patented in April, 1935. A new measurement principle is the application of radar waves. These are preferable in the industry for level measurements. The radar waves through the antenna are transmitted to the surface of the medium and reflected.

For flow measurement, a similar principle is used. It is similar to the evaluation of frequency shift by the ultrasonic Doppler method. At a defined angle the radar waves are transmitted and reflected. The flow height is measured by a separate level sensor.

Advantages: Non-contact measurement, no fittings in the medium, low mounting costs, low mobilization, no particles required.

Disadvantages: A minimum dielectric constant of the medium is required, measures point velocity on the fluid surface, and flow profile is calculated or empirically determined. On a smooth flow surface the signal will not be reflected, waves will be refracted and not reflected back.

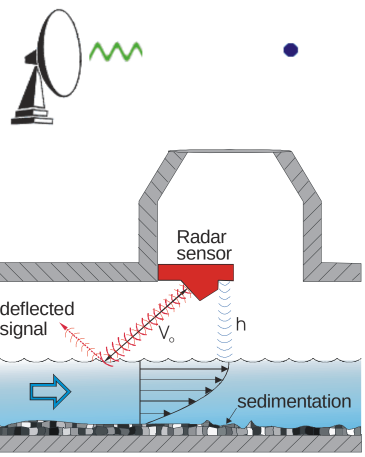


Fig. 13. Radar sensor

12 Ultrasonic Doppler Method

Doppler radar is named after Christian Andreas Doppler. Doppler was an Austrian physicist who first described in 1842, how the observed frequency of light and sound waves was affected by the relative motion of the source and the detector. This phenomenon became known as the Doppler effect. HydroVision uses the Doppler effect for flow measurement. Unlike the origin, transmitters and receivers are in fixed positions with a moving medium and reflecting particles. (CW Doppler principle)

The measurement principle is based on the fact that a bundled ultrasonic beam (f1) is continuously insolated into a liquid in a defined angle and a known frequency. A part of the ultrasound energy is reflected by the solids or gas bubbles contained in the liquid. Caused by the particles‘movement a frequency diversion (rf) occurs. This diversion is direct proportional to the particle velocity.

At a constant transmitting frequency, insolation angle and sound velocity the particle velocity Vp can be determined, where: f – transmitter frequency, C_0 – medium sound velocity, Vp – particle velocity, α - beam angle.

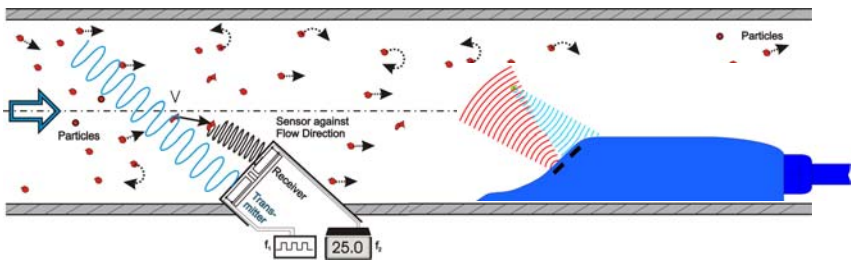


Fig. 14. Principles and device of measurement

At a constant transmitting frequency, insolation angle and sound velocity the particle velocity can be determined. A particular frequency spectrum results from the numerous reflecting particles and the flow profile.

This Doppler frequency spectrum must be evaluated with the help of analytical methods to get a characteristic velocity within the measurement distance. From that the average flow velocity **v** (m/s) in the wetted cross section can be calculated.

The Ultrasonic Pulse Doppler Principle is a new development that supersedes all the older Doppler principles. In contrast to CW Doppler principle, the transmitting frequency is continuously adjusted with the pulse Doppler and a shorter ultrasonic frequency bundle of defined length is transmitted.

Based on defined beam angle, the acoustic velocity and the medium temperature (compensated), the sensor transmits ultrasonic frequency bundles at time t_1, and receives the signal at time t_2.

This makes it possible to assign a defined measurement window for received signals. The frequency shift of the transmitted ultrasonic signal into a defined measurement window is the measurement of the flow velocity in that measurement window. Reflections of particles in other areas do not have any influence on the velocity measurement. The measurement window is defined on V_{max} (rotationally symmetrical flow profile). From this and a known weighting $V_{average}$ is determined. With the $V_{average}$ and the known diameter, the flow Q can be calculated.

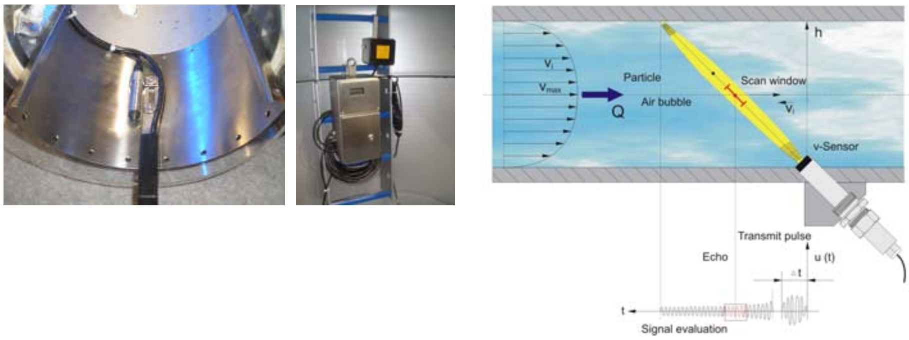


Fig. 15. Mounting of sensor Q-Eye in the sewer and principles of measurement

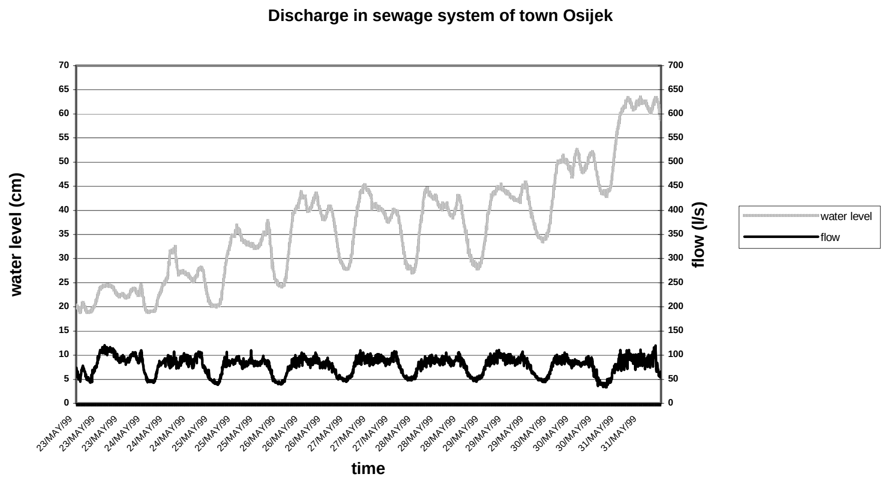


Fig. 16. Monitoring flow in very non stationary conditions (Sewage system of Osijek)

Device Q-Eye is a major improvement in open channel flow measurement. This compact system uses HydroVision's proven Acoustic Pulse-Spectral-Correlation Technology. The system is designed to measure both, the water level and the velocity in a single transducer assembly. Q-Eye directly reads mean velocity with superior accuracy, using up to 10 scan cells. Gone is the time wasted in figuring roughness factors and velocity corrections. A sub-merged ultrasonic level sensor is combined in the velocity sensor.

13 Conclusions

Measurement in the sewage system is extremely important for getting to know and the system management. Which of mentioned measurement method choose, depends of numerous local conditions of measuring point. In the non stationary conditions flow in sewage, has to necessary chosen the ultrasonic Doppler method.

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A05**

Environmental Impact Assessment of the Waste Water Treatment Plant for City of Skopje

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Abstract. This paper presents the results of the environmental impact assessment of the wastewater treatment plant for city of Skopje, with main focus on water, both groundwater and surface water, as part of the complex Study on Environmental Impact Assessment Study for the named waste water treatment plant. This presentation includes assessment methodology, project description and alternatives, baseline data related to water resources, impact assessment on water and proposed mitigation measures,. The assessment of the impacts has been performed for the construction and operational phase of the wastewater treatment plant. The construction phase was subdivided into several phases, while in the operational phase; the impacts are analyzed following the line for treatment of the wastewater and effluent production, the line for sludge production and drying process, sludge disposal and the line for gas utilization. Considering the fact that this wastewater treatment plant would be with the largest capacity in the Republic of Macedonia, and among the largest in the region, the presented results and outcomes could be of interest for wider scientific and engineering community in the country.

Keywords: Wastewater treatment plant, EIA, groundwater, surface water, pollution, sludge drying beds

1 Introduction

In July 2005, Macedonia requested support for the project "Wastewater treatment development in Skopje city", with expectation for public finance support of the Government of Japan. As a result of the positive answer of the Government of Japan, the Japan International Cooperation Agency (JICA) conducted a Preparatory Study and later "The Study for Wastewater Management in Skopje in the Republic of

Macedonia" that started in September 2007 and was completed by the end of 2008. This Feasibility Study includes construction and proposed type of technology of the Wastewater Treatment Plant (WWTP) for city of Skopje and installation of the main collectors on both river banks (left and right). The planed installations are required to comply with current EU and Macedonian legislation including the EU Urban Wastewater Treatment Directive (91/271/EEC). In parallel, the Environmental Impact Assessment (EIA) Study was conducted as part of the Feasibility Study [1].

2 Project Description

Design Parameters. According to the Feasibility Study the collectors are planned to be installed under roads planned by General Urban Plan - GUP (Target Year 2020). The total length of the left bank collector is 5,230 m, and it is foreseen to install concrete pipes of diameter 1,500 mm, 1,600 mm and 1,800 mm. The total length of the right bank collector is 4,030 m. Also, concrete pipes of diameter of 1,000 mm and 1,800 mm shall be installed. In order right bank collector to cross the River Vardar, it is planned to construct a siphon.

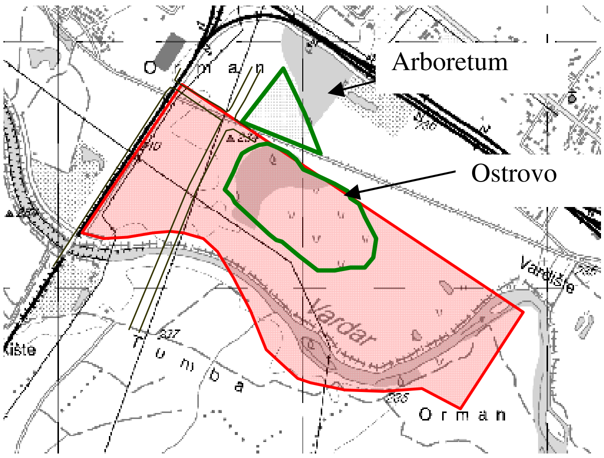

Fig. 1. Water Economy Facility Zone as proposed location of the WWTP in Trubarevo

Planned area for the WWTP is indicated as Water Economy Facility Zone in the GUP map and is designated as land for WWTP (Fig. 1). Planned location is near the settlement Trubarevo in the Municipality Gazi Baba, on distance of 1,2-1,9 km (depending on the used roads) from settlement Madzari which is at the south end of City of Skopje. The proposed location of the WWTP is at the lowest part of municipality, at the left Vardar river bank. In the vicinity of the location there are roads, railway line, while cargo station and warehouses (Public custom warehouse) is on the east side of the location. There are no settlements within the boundary of the proposed site. The area landscape characteristics include: meadows, un-fertile and fertile lands, water areas, etc.

The main features within this zone are the natural heritage Ostrovo and the Arboretum, an area out of the zone for the WWTP, which has the level of protection as natural monument.

The targeted year for the Central WWTP is 2020. The basic parameters used for designing the WWTP are given in the Table 1 and 2:

Table 1. Basic Conditions of the Central WWTP (1)

	Population (persons)	Per Capita Flow (l/pc/d)	Flow (m^3/d)		
			Average	Peak Factor	Hourly Maximum
Domestic Wastewater	513,570	200	102,720	2.0	205,440
Industrial Wastewater	-	-	32,300	-	32,300
Stormwater	-	-	31,000	-	-
Total	513,570	-	166,020 ≈166,000	-	237,740

Table 2. Basic Conditions of the Central WWTP (2)

	Population (persons)	Per Capita Load (g/capita/d)		Pollution Load (kg/d)		Water Quality (mg/l)	
		BOD	SS	BOD	SS	BOD	SS
Domestic Wastewater	513,570	60	45	30,814	23,111	300	225
Industrial Wastewater				5,399	9,175	167	284
Stormwater				3,410	12,400		
Total				39,623	44,686	239 ≈240	269 ≈270

Note: Population Equivalent based on pollution load is 603,900 persons, using 60 g/capita/day

Treatment Process. In the Feasibility Study, the selection of the treatment process was made upon the requirements of the EU Directives, constrains of the WWTP site and the project's investment and maintenance costs. After comprehensive analysis of several possible treatment processes (Conventional Activated Sludge Process, Extended Aeration Process, Oxidation Ditch Process and others), it was proposed to apply Conventional Activated Sludge Process (CASP) treatment process for the WWTP in Trubarevo**.**

The implementation of the proposed project for the WWTP is planned to be in two stages. The first stage is up to 2020 and includes conventional activated sludge process for BOD removal for the wastewater and secondary treatment for certain amount of storm water as first flush. At the second stage, after 2020, the treatment of the wastewater will be upgrade with nitrogen and phosphorous removal, while for the storm water, check on a need for detention reservoir is planned.

The Process flow of CASP and treatment of the sludge is shown in Fig. 2.
After thickening of the sludge in the gravity sludge thickener and digesting in anaerobic digestion tanks, the stabilized sludge is spread at drying beds. The selection of this method was done in comparison with the mechanical dewatering. The planned production of sludge is 72,4 m^3/day.

The selected solution for storage of the sludge is very much depending on the contents of the sludge, regarding the presence of dangerous substances from the industrial wastewater. Under the Law on Environment [2], the IPPC system has come into force and the prevention measures of water, air and soil will be the obligation of each industry before discharge. Each industry is required to implement the prevention measures by 2014 and the hazardous substances should be pre-treated before the wastewater is discharged into the collectors. By implementation and compliance of IPPC system, any dangerous substances will not be discharged into the sewers and the sludge can be disposed of at the existing landfill or used for the agriculture.

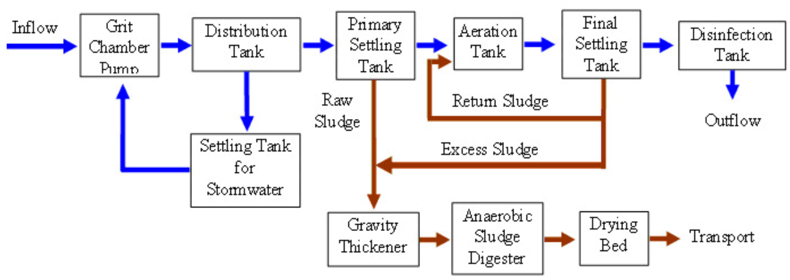

Fig. 2. Process Flow of CASP and Sludge Treatment

However, considering the situation that the deadline for complete introduction of IPPC system is extended to 2014 from 2007, and the duration of the process of issuing permits and achievement of compliance with the legal requirements, it is difficult to say that by the operation of WWTP the industrial wastewater which will be discharged into the sewers will be completely pre-treated and any dangerous substances will not be present in the sludge.

If there will be no dangerous matters in the sludge, it should be disposed at Drisla landfill. The solution for sludge with detected dangerous matters is to store it at special landfill for hazardous waste, which according to the plans of the MoEPP (Waste Management Strategy Action Plan) shall be operational in 2013. In the case that the special landfill for hazardous waste is not constructed yet, the sludge will be disposed at the temporary storage at the WWTP site.

Concerning utilization of sludge, it is planned that the dried sludge after composting can be utilized for agricultural purposes.

One of the most likely events that could jeopardize and endanger the operation of the WWTP is the flooding. Currently at the site there are embankments that cope with 1,000 year return period flood. However, there is a need for extension of the embankments between the railway on the western side of WWTP site and the existing embankment at the downstream of the river.

3 Environmental Impacts Assessment and Mitigation Measures

3.1 General

The first attempt to assess the environmental impacts was done within the "Initial Environmental Examination – IEE level study" [3]. Using the basic data from this Study, following the general recommendations for elaboration of the environmental impacts, using updated information and large amount of new data and taking into consideration all media and their interaction, detailed Environmental Impact Study was prepared [4]. In order to assess in more details possible impacts during construction, operation phase and post operation phase (closure) or some changes which are planed in the view of capacity or technology, of the access roads, main collectors, the siphon and the WWTP, following phases and activities have been taken in consideration:

a) Construction phase
- Construction of the access roads and main collectors (left and right river bank);
- Construction of the siphon structure across the River Vardar;
- Preparatory works at the location of the WWTP (tree cutting, humus removal and flattening of the location) and excavation works;
- Transport and disposal of surplus excavated material;
- Construction of the structures of the WWTP (civil works, use of heavy machinery and vehicles);
- Disposal of construction waste;
-Installation of the equipment;
-Construction of accommodation facilities for the workers (water supply, sewerage, waste disposal).

b) Operation phase
- Treatment technology/ operation of the equipment for sewerage treatment and effluent production;
- Operation of equipment for sludge production (digester, drying beds and biogas production);
- Sludge (with dangerous substances) disposal on temporary storage at WWTP site.

c) Scenario after 2020 - Upgrading of the capacity of the WWTP or developing additional treatments.

d) Closure phase
In the construction and operational phases, three main groups of environmental elements were analyzed, possible impacts were identified and mitigation measures were proposed. The following environmental elements were analyzed:

- *Natural Environment*: Topography and geology (including ground subsidence); Water quality: Groundwater, Surface water/River Vardar water quality (including bottom sediment); Hydrology of River Vardar; Biodiversity/ flora and fauna; Air quality (including meteorology); Landscape and visual effects; and Water use.
- *Social Environment*: Involuntary resettlement and Land acquisition; Livelihood and local economy; Institutions as local decision-making; Public infrastructure and services; Misdistribution of benefits and loss/damage; Local conflicts of interest; Archaeological and cultural heritage; Health and safety (including infectious diseases).
- *Public hazards*: Noise and vibration; Waste; Soil pollution and Offensive odour.

The impacts are assessed using qualitative assessment of the following parameters [5]:

Type:	Positive (+); Negative (-)
Magnitude:	A – large, B-medium, and C-low
Extent:	Local impact (at the site); Wider impact (in the surrounding area)
Duration:	Permanent impact; Temporary impact
Timing:	Immediate; Delayed
Reversibility:	Reversible; Irreversible

In this paper, the emphasis is given to the impacts and mitigation measures related to water resources in construction and operational phase.

3.2 Construction Phase

Water Quality/Groundwater

Impacts. The routes for the access roads are mainly on flat area where higher groundwater level could be expected [6]. Due to this, during excavations for some of the road accompany structures (rainfall water evacuation, crossings under the roads, culverts etc), possible medium negative impact can be expected in a form of disturbances of the groundwater table.

The same type of impact is expected during the construction of the main collector on both river banks. Additionally, medium negative impact is expected due to evacuation of the pumped groundwater from the construction trenches and its discharge downstream.

Construction of the siphon across the River Vardar could be one of the critical construction phases, due to the need for creation of river diversion structures. As the excavation and installation works shall be done on rather lower level than the river bed, large negative impact in a form of disturbances of the groundwater table can be expected. Additionally, low negative impact is expected due to discharged pumped groundwater downstream in the river.

During construction of the WWTP facilities, large negative impact is expected on the groundwater level, as the excavation works will significantly disturb the aquifer level. These disturbances of the groundwater level can provoke disruption of supply of the local wells which are used for domestic water supply or irrigation [7]. Voluminous excavation works will create large areas "without" materials (holes) that are imposed to possible pollution due to soil erosion and possible increased turbidity. This possibility is assessed as medium negative impact.

The ground water quality can be impacted by improper disposal of the construction waste on the construction site and surrounding.

Another type of medium negative impact is related to possible pollution of the groundwater due to leakages of fuels and oils from the heavy vehicles and machinery used for construction and due to applied chemicals during this phase.

The facilities for daily accommodation of the workers, supervision staff and other utility offices are equipped with systems for water supply and sewerage and adequate wastewater treatment. Improper operation of the sewerage system and wastewater treatment of such temporary facilities can have medium negative impact (due to duration and quantity of the impact) on the groundwater, as they can provoke pollution.

Mitigation Measures. Large number of the measures for mitigation of the negative impacts on the groundwater shall be defined and integrated in the Final designs. That includes measures for: avoiding or minimizing disturbance of the groundwater level; safe drainage and evacuation of the pumped groundwater, in order to avoid possible suffusion (to be defined in a separate part of the Final design); erosion control and soil conservation during excavation; definition of the characteristics of the heavy vehicles and machinery according to the required standards.

During construction, all measures foreseen in the Final design must be fully respected and applied.

At the construction site, in order to avoid pollution of the ground water, the following measures should be applied: construction waste shall be, regularly and timely transported from the construction site and disposed at the designated landfill for construction waste; refuelling or servicing of the vehicles and machinery shall be done only on impermeable ground; special measures to be design and foreseen to avoid potential spills leaks; washing of vehicles and equipment on the site to be restricted; chemicals and other liquid and solid dangerous materials must be managed properly (it covers: manipulation and storage); wastewater from the accommodation facilities shall be collected and adequately treated and solid waste shall be collected and disposed at Drisla landfill.

Surface Water/ River Vardar Water Quality (including the bottom sediment)

Impacts. During construction of the access roads, at some locations of the access road on the left bank, where the route goes very close to the river Vardar and due to the strong connection of the water from the river and surrounding groundwater, there is a possibility for pollution of the water in the river due to increased surface runoff and soil erosion during excavations. The impact is assessed as indirect impact.

During construction of the main collector (left bank), beside the already described possible pollution, there is possibility for surface water pollution indirectly through the originally polluted groundwater by the leakages of fuels and oils from the operation of heavy machinery and vehicles.

During the construction of the siphon, both cases of pollution are possible.

Construction of the WWTP facilities, additionally to already mentioned possible pollution, can pollute River Vardar after heavy rains and increased surface runoff on the construction site and from leakages and spills of fuels and oils from heavy machines and transport vehicles used for installation of the equipment for wastewater treatment. Possible pollution is expected due to discharges of untreated wastewater from the accommodation facilities into the river or ground water and /or improper solid waste management.

All these impacts are assessed as negative with medium magnitude, while those related to heavy rains and pollution during equipment installation, are of low magnitude.

Construction phase will not provoke any impact on the bottom sediment. Accordingly, there are no mitigation measures foreseen.

Mitigation Measures. Some of the mitigation measures for prevention of the pollution of the water in the river through the contact with the groundwater shall be integrated in the Final design and shall be fully applied during the construction phase. At the site, refuelling or servicing shall be done only on impermeable ground and oils shall be treated. Special measures shall be foreseen in order to avoid potential spills or leaks and adequate erosion control and soil conservation practices shall be applied. Wastewater from the accommodation facilities shall be treated and solid waste shall be collected and disposed at Drisla landfill, applying good waste management practices.

In general, all measures mentioned above for protection of the ground water, shall be applied.

Hydrology of the river Vardar

Impacts. During this phase, the hydrology (River Vardar) will be affected only with the construction of the siphon. In order to construct the siphon across the River Vardar, different river diversions structures and tail dams shall be constructed. With these structures, there will be large negative impact on the river flow direction, while the river discharges will stay unchanged.

Mitigation Measures. Mitigation measures in a form of solution for the river diversion with minimum disruption of the riverbed shall be defined in the Final design. During construction, all proposed protection measures related to the technology of the construction, shall be fully respected.

Water use

Impacts. During construction phase, there will be significant emission of dust. In order to reduce that emission, usual method is to spray water at the site. This will have negative impact of low magnitude on the water use at the site, specifically. Water at the construction site will be used also for drinking and sanitary purposes, mainly for the workers and other staff. This impact is assessed as negative of medium magnitude, as there will be large number of workers and other staff. Additionally, proposed activities can affect water drilling systems, downstream the locations of activities, as well private wells.

Mitigation Measures. In order to mitigate the above mentioned negative impact, it is recommended that water of low quality shall be used and the cisterns for spraying shall use the water efficiently. As the construction site is very close to the River

Vardar, water for spraying can be abstracted from the river or from wells at nearby locations.
In order to minimize the water use for drinking and sanitary purposes, it is recommended to use the water efficiently and to apply water saving techniques.

3.3 Operation Phase

Water quality/Groundwater

Impacts. During operation of the WWTP, there is a possibility for pollution of the groundwater due to leakages of the system for sewage treatment and effluent production, leakages of the system for sludge production, and due to refuelling of the vehicles and washing of the vehicles at the site. These impacts are assessed as negative with medium magnitude.
Also, during the drying process of the sludge on the drying beds, there is high possibility for pollution of the groundwater due to infiltration of drying beds leachate. As the drying beds are covering area of 18 ha, the possible negative impact is assessed as large affecting wider area, actually wider groundwater aquifer. Another large negative impact on the ground water can be caused by leakages and infiltration of the leachate from the temporary storage location of the sludge with dangerous substances.
On the other hand, operation of the WWTP will have large positive impact on the quality of the groundwater, as there will be no direct discharges of wastewater into the River Vardar. The pollution of the groundwater in Skopje area is mainly due to polluted water from Vardar through strong connection with the groundwater and leakages of the existing collectors of wastewater.

Mitigation Measures. Proposed measures must ensure mitigation of the large negative impacts on the groundwater. They shall include the following actions and recommendations: the system for the treatment of the wastewater and effluent production should ensure minimization of leakages of wastewater to groundwater (connections between pipes and tanks should be water-tight); refuelling of vehicles and equipment on the site shall be strictly controlled; washing of vehicles and equipment on the site shall be restricted; the system for the sludge production should ensure minimization of leakages of sludge to groundwater (connections between pipes and tanks should be water-tight); all requirements for construction of the sludge drying beds, especially for providing water impermeable basis, efficient drainage system for leachate and flood protection structures must be respected; to provide water impermeable basis and flood protection structures on the location for the temporary disposal of the sludge with dangerous substances and measurements of leachate should be taken.

Surface water/River Vardar Water Quality (including the bottom sediment)

Impacts. The major positive impact from the operation of the WWTP is the improved water quality of the River Vardar, on larger area (along the Skopje Valley and downstream of the city).
There is a possibility for indirect negative impact of medium magnitude due to the strong connection with the groundwater. The water of the river could be polluted from polluted groundwater due to leakages of the system for wastewater treatment and effluent production and system for sludge production as well as due to refuelling of vehicles and equipment and washing vehicles.
There are two more possible large negative impacts related to the sludge drying beds and temporary storage of the sludge with dangerous substances at the WWTP site. In both cases the main danger is the leakage and infiltration of the leachate into the groundwater and then into the River Vardar.
Operation of the WWTP will have large positive impact on the bottom sediment. As there will be no wastewater discharged into the River Vardar, there is no possibility for polluting the bottom sediment.

Mitigation Measures. The proposed mitigation measures are the same as those for protection of the groundwater during operation, presented above in the text.

Hydrology of the River Vardar

Impacts. Before the existence of the WWTP, the wastewater was discharged directly into the River Vardar, which increased the flows of the river. With collection of the wastewater by the main collectors, the quantity of the river flows shall be controlled and decreased up to the location of the WWTP. Accordingly, this impact is assessed as negative with low magnitude considering the volume of the river flow and wastewater on wider area of Skopje valley.
At the same time, downstream of the WWTP there will be increase of the river flows due to discharged treated wastewater, which is assessed as positive impact, with low magnitude.

Mitigation Measures. In order to mitigate the impact due to low flows in summer period in the river Vardar, additional quantities of water can be discharged in the river from upstream reservoirs Matka or Kozjak.

Waste

Impacts. Operation of the wastewater treatment system generates large quantities of sludge (72,4 m^3/day) that provokes large negative impact on all media (soil, groundwater, air, etc). The quantities of sludge with hazardous substances disposed at the temporary storage can also provoke large negative impact on all media. Improper treatment of sludge could lead to putrefaction and other related problems such as bad odour, health effects etc.

Mitigation Measures. Proposed mitigation measures comprise application of good waste management practices and disposal of the scrape and communal waste at Drisla landfill as discussed in the analysis of alternatives.

Especially important measure is the recommendation for analyzing alternatives for sludge treatment and reduction of the quantities of sludge and to propose optimal solution according to the local conditions. It is also of crucial importance to dispose the sludge with dangerous substances at the landfill for hazardous waste as a final disposal location.

4 Conclusions

This chapter presents the major negative and positive impacts from the construction and operation of the WWTP and the main collectors on the water resources.

4.1 Construction Phase

Negative Impacts:
- Construction of the siphon across the River Vardar could be one of the critical construction phases, due to the need for creation of river diversion structures. Large disturbances of the groundwater table can be expected during the excavation works on the WWTP facilities that could result in disruptions of supply of the local wells which are used for domestic water supply or irrigation.
- Different river diversions structures and tail dams will change the river bed morphology temporarily but the impact could rather high. With these structures, there will be impact on the river flow direction, while the river discharges will stay unchanged.

4.2 Operational Phase

Positive Impacts:
- In general, operation of the WWTP will have large positive impact on the quality of the groundwater, the bottom sediment and the most of all on the water quality of the River Vardar;
- The operation of the WWTP will have positive impact on the restoration and maintaining of the aquatic fauna in the River Vardar;
- There are large positive impacts on the Social Environment elements: improved water supply of the downstream populated areas due to good quality of the groundwater, and improved health of the downstream population as a result of the improved quality of drinking water.

Negative Impacts:
- During the drying process of the sludge on the drying beds, there is high possibility for pollution of the groundwater due to infiltration of drying beds leachate. As the drying beds are covering area of 18 ha, the possible negative impact is assessed as large affecting wider area, actually wider groundwater aquifer;
- There is high possibility for groundwater and soil pollution with substances due to leakages and infiltration of the leachate from the sludge with hazardous substances disposed on temporary storage at the WWTP site;

- Through the strong connection with groundwater, the water in the River Vardar could be polluted from infiltrated leachate from sludge drying beds;
- Generation of large quantity of sludge (72,4 m^3/day) provokes large negative impact on all media (soil, groundwater, air, etc). Improper treatment of sludge could lead to putrefaction and other related problems such as bad odour, health effects etc.;
- Generation of big quantity of sludge with hazardous substances can provoke large negative impact on all media (soil, groundwater, air, etc).

In order to mitigate the negative impacts, extensive mitigation measures are proposed, described in details in above text. It is expected that with efficient application of these measures, there will be no large negative impacts on the water resources during construction and operation phase.

Especially important measure is the recommendation for analyzing alternatives for sludge treatment and reduction of the quantities of sludge and to propose optimal solution according to the local conditions.

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A07**

A Comparative Study of Approaches for Hydraulic Width Determination of Sub-Catchments in Urban Stormwater Model

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Abstract. In the framework of analysis presented in the paper an influence of subcatchment hydraulic width determination approaches on runoff from urban catchment is investigated. Two approaches are compared. In the first one a hydraulic width is computed as a function of geometric dimensions of each particular subcatchment. In the second approach a hydraulic width is evaluated based on assumed of equal (for all subcatchments) length of runoff path from impervious part of its surface. A runoff path length is determined as a function of a distance between inlets to sewers. The analysis is performed for real urban catchment in the city of Poznan. A rainfall-runoff model based on SWMM5 package is applied for computation of runoff from particular catchment. Simulations are performed for historical rainfalls from the period 2006 – 2007. An assessment of approaches of hydraulic width evaluation is based on comparison of computed and measured runoff hydrographs. For both approaches a temporal pattern of computed hygrograms agrees with the shape of measured hygrograms. The first approach allows for computation of maximal values of runoff with an accuracy of 15%. The same accuracy was obtained in the second approach for the length of runoff path in range 50-75 m.

Keywords: Urban hydrology, runoff computation, sub-catchment width

1 Introduction

One of the most significant parameters characterizing runoff from urban catchments is a hydraulic width of its subcatchments. Two approaches to the width evaluation described in literature were extended by an original author's approach. The presented paper contains a comparative study of specified approaches performed for a real urban catchment in the city of Poznan. The study is based on theoretical analysis and comparison of hydrographs presenting measured and simulated variation from the considered catchment. Hydrographs were developed for three historical rainfalls. A set of special indicators was used for comparison of both hydrographs and assessing correctness of fit for the considered approaches.

2 An Overview of the Methods of Catchment Hydraulic Width Determination

2.1 Interpretation of Catchment Hydraulic Width

The first step in runoff computation from an urban catchment is usually aimed at its division into subcatchments according to the configuration of terrain and the layout of network. A runoff from such a subcatchment can be computed using different hydrological models, one of them being the model of kinematic wave described by well known set of equations:

$$\frac{dh}{dt} + \frac{Q}{A} = q \tag{1}$$

$$Q = B \cdot \frac{\sqrt{s}}{n} \cdot h^{\frac{5}{3}} \tag{2}$$

Substituting Eq. (1) for Eq. (2) one obtains:

$$\underbrace{\frac{dh}{dt}}_{\text{retention}} = \underbrace{q}_{\text{inf low}} - \underbrace{\frac{B \cdot \sqrt{s}}{A \cdot n} \cdot h^{\frac{5}{3}}}_{\text{outflow}} \tag{3}$$

Retention characteristics of subcatchments are represented in these equations by three parameters: hydraulic width B, surface slope s and surface roughness coefficient n of surface according to Manning. The increasing of hydraulic width B results in augmentation of outflow and reduction of surface retention. Considering an extreme situation when $B \to \infty$, by an unchanged area of subcatchment, its answer to a rain impulse is immediate and the shape of rainfall hydrographs and runoff hydrographs are the same. On the other hand, by $B \to 0$, runoff also streams to zero and subcatchment is transformed into retention basin without outflow. The expression B/A in Eq. (3) represents the reciprocal of average length of runoff path L_{av}. Assuming constant area of subcatchment, the hydraulic width B and the average length of runoff path can be treated in computations as equivalent parameters.

2.2 Evaluation of Hydraulic Width Based on Dimensions of the Particular Subcatchments – Option 1

For a subcatchment which is symmetric in respect of its main collector (Fig. 1a) it is assumed that a hydraulic width B is two times larger than the length of the main collector L_{col} of subcatchment [3]:

$$B = 2 \cdot L_{col} \tag{4}$$

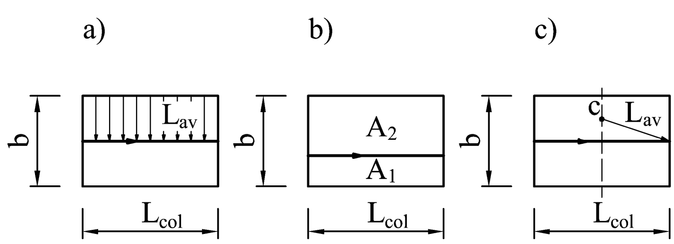

Fig.1. Graphical interpretation of subcatchment hydraulic width determination approaches: a/ Option 1- symmetric catchment, b/ Option 1- asymmetric catchment, c/ Option 2

Denoted as b the width of collector subcatchment (measured perpendicularly to this collector) an area of subcatchment A can be computed as a product of b and L_{col} :

$$A = b \cdot L_{col} \tag{5}$$

Hence an average length of runoff path L_{av} can be computed as a ratio of A and B and expressed as a function of b:

$$L_{av} = \frac{A}{B} = \frac{b \cdot L_{col}}{2 \cdot L_{col}} = \frac{b}{2} \tag{6}$$

For an asymmetric subcatchment or by its irregular shape (Fig. 1b) the hydraulic width is computed as a function of skewness coefficient S_k. The latter is defined as the ratio of subcatchment areas' difference on both sides of collector to total area of subcatchment:

$$S_K = \left| \frac{A_2 - A_1}{A_2 + A_1} \right| \tag{7}$$

Thus the hydraulic width is computed from the formula is:

$$B = (2 - S_K) \cdot L_{col} \tag{8}$$

The average length L_{av} of runoff path is obtained from the expression:

$$L_{av} = \frac{A}{B} = \frac{b \cdot L_{col}}{\left(2 - S_K\right) \cdot L_{col}} = \frac{b}{\left(2 - S_K\right)} \tag{9}$$

If both components of subcatchment area are equal i.e. $A_1 = A_2$ then according to Eq. (7) $S_k = 0$ and the hydraulic width can be computed from Eq. (4). With increasing differences between areas A_1 and A_2, the coefficient S_k is approaching 1, and the hydraulic width, according to Eq. (3), is approaching the length of collector.

2.3 Evaluation of Hydraulic Width Based on Location of Subcatchment Gravity Center –Option 2

In this approach it is assumed that the length of runoff path in a subcatchment L_{av} is a function of an impervious surface center of gravity and is evaluated from Eq. (8):

$$L_{av} = \sqrt{\left(\frac{L_{col}}{2}\right)^2 + \left(c \cdot \frac{b}{16}\right)^2} \tag{10}$$

For a symmetrical subcatchment, dimensions of which satisfy condition $b < L_{av}$, a length of runoff path computed by approach 1 is always smaller than by approach 2. It means that the ratio of respective subcatchment widths is reciprocal.

2.4 Evaluation of Hydraulic Width Based on the Assumption of Equal Length of Runoff Paths for all Subcatchments – Option 3

In urban catchments impervious surfaces composed mainly of roofs, streets, and pavements usually play a dominant role in transformation of rainfall into runoff. This process doesn't depend so much on area and dimensions of subcatchments but on the structure of a drainage system, more specifically on the distance among inlets to storm sewers. It justifies, in computation of hydraulic width, an assumption of equal runoff paths for all subcatchments, which respond to runoff paths from impervious surfaces:

$$B = \frac{A_{imperv}}{L_{av}} \tag{11}$$

It should be noticed that due to simplification of drainage network model caused by neglecting storm sewers of smaller diameters, the flow through these sewers is simulated as surface flow. It results in extension of runoff paths, which are longer than distances among inlets. Such a tendency can be generalized [13]. Taking into consideration the same simplification for all subcatchments, it can be assumed that an extension of runoff path will be similar for all subcatchments.

3 Case Study

3.1 Characteristics of the Piasnica Catchment

Analyzed in the case study catchment of Piasnica collector is situated in the city of Poznan in the area of Rataje (Fig. 2a). Its surface measures almost 700 ha. It is mainly covered by dwelling houses built in the second half of the last century (blocks of flats and some villas), two large industrial quarters (a brewery and a paper mill), a shopping center and a hospital. Runoff from catchment is collected by storm sewer network and transported to Cybina River at cross-section downstream from the Malta Lake.

3.2 Computer Model of the Catchment

Simulation of runoff from the Piasnica Catchment was performed using the stormwater model SWMM5. For computation of the surface runoff a model of kinematic wave was applied, while flow in drainage network was computed using a model of dynamic wave (the simplified version of the full Saint Venant equation). The data for construction of the Piasnica Catchment model was taken from the plans of drainage network in scale 1:500, longitudinal profiles of storm sewers, and additionally from topographic maps in scale 1:10.000 and 1:25.000.

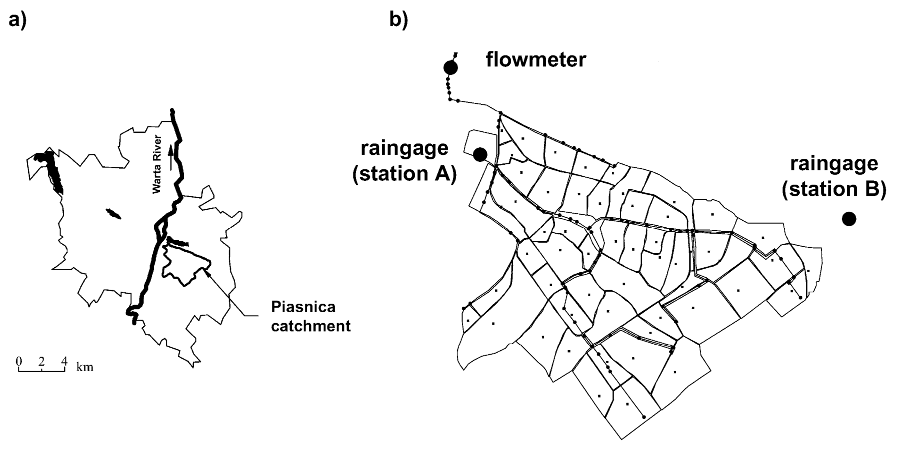

Fig. 2. a) Sitiuation of Piasnica catchment in area of town Poznan, b) location of measurement points

By modeling drainage network only larger sewers (with diameters of 500 mm and larger) were considered. The model of the flow through smaller storm sewers was replaced by a surface runoff model. This simplification resulted from the lack of complete drainage network documentation and was justified by its small influence on characteristics of outflow (depth and discharge) at the outlet from the exemplary catchment. Altogether 82 stretches of storm sewers and 55 subcatchments were taken into consideration by modeling of the catchment (Fig. 2b).

Its total area was evaluated as 700 ha; an average percentage of impervious surfaces were assessed as 29% but it was evaluated separately for each individual subcatchment. A slope of specific subcatchment was identified with an average slope of the terrain determined along its main collector. The roughness of storm sewers was computed on the basis of depth and discharge measurements – more specifically using measured hydrographs of these variables [12]. The values of remaining parameters used in modeling of an exemplary catchment were taken from literature [3, 4] and are presented in Table 1.

Table 1. The values of parameters used in modeling Piasnica catchment.

Parameter	Type of surface	
	impervious	pervious
Slope of catchment (-)	0.005	
Manning roughness coefficient of surface ($s/m^{1/3}$)	0.015	0.100
Surface retention (mm)	1.5	5.0
Maximum infiltration (mm/h)	-	50
Minimum infiltration (mm/h)	-	1.5
Infiltration constant (h^{-1})	-	4
Manning roughness coefficient of sewers ($s/m^{1/3}$)	0.018	

3.3 Measurements of Rainfall and Runoff

Measurements of rainfall for the study were performed each year since 2002 from April to October. Tipping bucket raingauges were located in two points (Fig. 2b):

- on the yard of the police headquarter of District Rataje (station A)
- in the garden of a private property in Kobylepole Area (station B)

Two types of rain gauges were used:

- ISCO 674 with resolution of 0.1 mm equivalent to filling one tipping bucket (station A),
- ANEMO SR 49 with resolution of 0.2 mm (station B).

The flow meter ISCO 2150 AV applied in the study enabled the measurements of depth (with a hydrostatic sensor) and the discharge (with ultrasonic sensor). It was installed in the year 2006 in the main collector (1.80 m diameter) about 100 m upstream from its outlet to the Cybina River, near the Malta reservoir dam [10].

The data from both flow meters, similarly as well as from rain gauges was registered in data-loggers and collected to laptop every four weeks. All the items of equipment specified above were supplied from batteries.

4 Scope of Analysis

4.1 Selection of a Hydraulic Width Evaluation Option

Two options of hydraulic width evaluation were compared in the study:

- Option 1 (based on specific dimensions of subcatchment) with the assumption of subcatchment symmetry with respect to its main collector,
- Option 3 (based on equal route of runoff for all subcatchments) for different values of this characteristic.

An analysis of subcatchment delimitation procedure reveals that they are usually traced along the largest neglected storm sewers (of diameters smaller than 500 mm). The width of these sewers subcatchments (denoted by b) doesn't exceed their length L_{col} i.e. $b < L_{col}$. Taking into account this relation it can be shown (by comparing Eq. (6) and (10) that the average length L_{av} of runoff path computed according to Option 2 is always longer than the one computed according to Option 1. The relations between hydraulic width of subcatchments are reciprocal (Eq. 6). Hence the hydraulic width of subcatchment computed according to Option 1 can be treated as the special case of the width computed according to Option 2. It justifies neglecting Option 2 in the further analysis.

In Option 3 of catchment width evaluation it was assumed that an average length of runoff path can be approximated by a distance between runoff inlets. According to literature [1, 2] this distance should not exceed 25 m. Taking into consideration remarks enclosed in section 2.4 one can expect that an average length of runoff path assumed in the model should be longer. Therefore in this study the runoff path of 25 m and its multiplications: 50, 75 and 100 m were considered.

4.2 Measurement Data

Three rainfall events selected from historical series were used in analysis. The maximum depth of water measured in the main collector of Piasnica catchment due to the above specified rainfalls was located in the range 50-80% of a collector height.

Table 2. The depth of rainfall used in the analysis

Rainfall	Date	Station A [mm]	Station B [mm]
A	25.06.2007	21.4	29.2
B	07.11.2007	18.2	22.0
C	12.04.2008	15.0	13.0

The initial verification of rainfall-runoff model of Piasnica catchment is based on comparison of computed and measured volume of runoff. By evaluation of the first value it was assumed that runoff takes place only from impervious surface of catchment constituting 29% of its total area i.e. 700 ha. Rainfall depth was evaluated in an approximate way as an average from the two rainfall station described above.

Measured volume of runoff was determined in an indirect way – using results of continuous measurements of outflow from the analyzed catchment in the main collector.

Table 3. Comparison of measured and computed runoff volumes.

Rainfall	Measured volume of runoff [m^3]	Computed volume of runoff [m^3]	Relative error of volume [%]
A	41900	49200	17
B	34300	39100	14
C	25700	27200	6

The computed runoff volumes presented in Table 3 are larger than respective measured values. It can be explained by the assumption that a runoff from pervious surface does not participate in the total outflow from a catchment. The difference between computed and measured values of runoff, which can be measured by a relative error of volume is directly proportional to the duration time of rainfall, the number of peaks in hydrograph and the distance between them. The influence of these factors can be explained by neglecting hydraulic losses (wetting losses, surface retention) and evaporation in modeling. This situation results in an increase of a computed rainfall volume and consequently in a positive value of its relative error. For instance, for a rainfall A characterized by a relatively long duration time and several peaks separated by interruptions of rainfall, participation of hydraulic losses and evaporation in total volume of runoff is significant and a generates high value of a relative error.

A relative error can be additionally influenced by neglecting spatial variability of the rainfall in computation. The depth of the rainfall measured at station A and B is substituted by an average value which is used for the whole catchment. In the presented analysis the largest differences in rainfall depth measurements were observed for rainfall A, which probably was the reason of the high value of a relative error.

4.3 Criteria of Results Assessment

For comparing the results of simulations with measurements of outflow the following indicators advised in literature [6, 7] were used in this study:

- ISE integrated square error
- AVR average values ratio
- R linear correlation coefficient
- RS special correlation coefficient;
- R^2 model efficiency by Nash & Sutcliffe [5]
- ΔQ_{max} relative error of maximum flow

Indexes *m* and *s* denote respectively measurements and simulations. Symbol *n* refers to the number of points used for description of hydrographs. Ordinates of them are denoted by Q_{max}

Depending on the values of the considered indicators the model can be classified as belonging to one of the five classes specified in Table 4.

Table 4. Classification of the model quality based on the values of selected indicators [7]

Class of model	Indicator and scope of its value	
	R, RS [-]	ISE [%]
excellent	1.00-0.99	0-3
very good	0.99-0.95	3-6
good	0.95-0.90	6-10
fair	0.90-0.85	10-25
poor	below 0.85	above 25

For indicators not specified in the head of the table i.e. AVR, R^2 and ΔQ_{max} described above classification cannot be applied. Instead of that the extreme values responding to ideal agreement of hydrographs are defined. They are equal to 1.0 for AVR, R^2 and for ΔQ_{max}.

5 Results Analysis of Runoff Simulation

5.1 Simulations for Option 1 of Hydraulic Width Computations

For all computed hydrographs a coincidence of peaks appearance (time to peak from the beginning of rainfall) with measurements is observed. Some differences in peaks flows appearing in all hydrographs result from spatial variation of rainfalls [11], which was not considered in this analysis.

Table 5. Values of model quality indicators for option 1 of subcatchment width determination

Indicator		Rainfall		
		A	B	C
ISE	[%]	0.853	0.913	1.045
AVR	[-]	0.979	0.992	0.953
R	[-]	0.885	0.903	0.988
RS	[-]	0.969	0.973	0.987
R^2	[-]	0.759	0.777	0.949
ΔQ_{MAX}	[%]	-12.8 (0.1)	-0.1	-5.4

In order to assess the correctness of model simulations statistical indicators were used. The smallest values of statistical indicators R, RS and R^2 were obtained for rainfall A (Table 5).

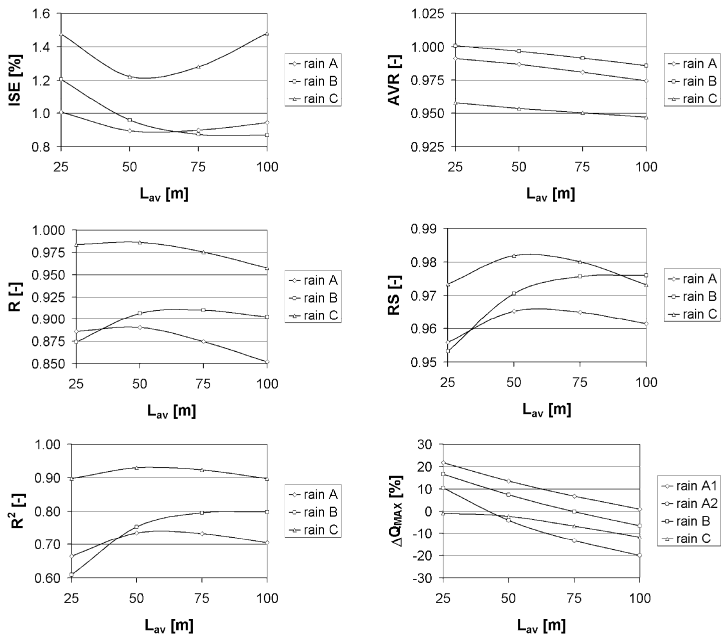

Fig.3. Changes of model quality indicators as a function of average runoff path length

For this rainfall the highest differences between rainfall measurements on station A and B were observed (among all considered in analysis rainfall events). Due to the presence of two comparable maxima of the measured value of outflow, the indicator ΔQ_{max} was computed two times) for each peak separately) and both values were shown in Table 5. The highest values of indicators were obtained for the rainfall C indicating the smallest spatial variation.

For all three rainfalls the computed maximum runoff values were smaller than the respective measured values. These differences were enclosed in an acceptable interval of 15% [4]. An evaluation of hydraulic width according to Option 2 resulting in smaller values of subcatchments hydraulic width in comparison with Option 1, would increase the difference between computed and measured values of discharge and depth and make the values of indicators worse.

5.2 Simulations for Option 3 of Hydraulic Width Evaluation

The largest number of the best indicators was obtained for the runoff path length L_{av}= 50 m (for rainfalls A and C) and L_{av}=75 m for the rainfall B. An analysis of the results presented in Fig. 3 leads to the conclusion that the majority of the indicators considered (ISE, AVR, R and RS) are dependent to a small degree on the runoff path length. Only R^2 and the relative error of maximum runoff ΔQ_{max} are more sensitive to this parameter. Generally one can conclude that a change of the runoff path length has a relatively small influence on the results of runoff simulations.

6 Conclusions

The results of runoff simulations from the exemplary Piasnica catchment obtained for Option 1 of subcatchment hydraulic width evaluation are comparable with the results for option 3. If referred mainly to the shape of hydrographs (mainly the sequence of peak flows), the computed peak flows are generally a bit lower than the respective measurement results. Taking into consideration the fact that the outflow from catchment is computed from kinematic wave equations, peak flows are directly proportional to the square root of the surface slope and inversely proportional to the Manning surface roughness coefficient. A better adjustment of peaks would require the slope increasing of the subcatchment or a reduction of the Manning coefficient. Changing of the third basic model parameter – subcatchment hydraulic width in option 1cannot be taken into consideration, because the hydraulic width is determined by the length of the main collector L_{col} (Eq. 4). In case of Option 3 the highest values of indicators are obtained for runoff path length from the interval 50 -75 m. For a catchment with comparable to Piasnica catchment parameters and drainage network simplifications (neglecting storm sewers of D<500 mm) computations of hydraulic width of subcatchments can base on equal value of average runoff path selected from this interval.

Assuming that the sequence of peak flows depends on the values of three basic parameters specified above i.e. slope s, roughness coefficient n and subcatchment width B or runoff path L_{av} a computed length of runoff satisfies the condition.

$$\frac{\sqrt{0.005}}{0.015 \cdot 75} \leq \frac{\sqrt{s}}{n \cdot L_{av}} \leq \frac{\sqrt{0.005}}{0.015 \cdot 50} \tag{12}$$

Based on Eq. (12) a range of computed length of runoff path can be computed for other catchments characterized by two remaining parameters (slope s and surface roughness coefficient n). A sensitivity of SWMM model to changes of subcatchment hydraulic width depends on parameters of rainfall [9]. One value of hydraulic width assuring compatibility of computed and measured hydrographs does not exist.

Acknowledgments

This work has been financially supported by the Ministry of Science and Higher Education of Polish Government (Grant No. 1253/H03/2006/30).

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A10**

Alternatives for Leachate Treatment from the Solid Waste Landfill-Centar Župa

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Abstract. The paper presents the results of the analyses of alternatives for leachate treatment from the solid waste landfill located in the municipality of Centar Župa. The main goal of the analyses is to consider the possibilities for construction of a combined treatment plant for leachate from the landfill and waste water generated from the inhabitants of the municipality of Centar Župa. The analyses of the leachate treatment site selection are consisted of two alternatives: at solid waste landfill Centar Župa site (treatment of leachate at the landfill site) and at the municipal waste water treatment plant for Centar Župa site (treatment of leachate in combination with domestic sewage). Also, two alternatives for transporting of leachate to the selected waste water treatment site are analyzed: use of tankers and construction of pressurized pipe.
Also, three alternatives for integral solid waste and water management in the view of costs estimation and set up tariffs for collecting and disposing of solid waste are presented. Based on economic analysis the most favorable scenario is found for the public utility enterprise and for sustainability of the investment.

Keywords: solid waste landfill, leachate, treatment, water balance method

1 Introduction

Radika River and its tributaries are located in the west and north-west part of Macedonia. The natural and cultural heritage of the River Radika basin is amazing. The Radika River basin is specific eco-system. The flow of the river is in the canyon of which is some 40 km long and is surely one of the most picturesque, as well being the steepest in Europe. Biggest part of the district is within the protected by law Mavrovi-Anovi National Park. In the Radika district there are about 20,500 inhabitants living in 63 villages, organized in two municipalities: Rostuse&Mavrovi Anovi and Centar Župa. Main occupation of population is livestock breeding that is quite extensive. In the district there are several small industrial capacities, mainly textile and food processing facilities [1]. The economic development in the region of the Radika river basin, and the changed way of living in the last decades have contributed towards a more intensive use of natural resources, as well as generating different substances that are harmful to environment and human health. Due to the improper management and disposal, the solid waste is considered to be the main polluter of the environment, because it is not collected and disposed (the only exception is the area of the former Municipality of Mavrovi Anovi). In addition, none of the villages has proper sewage system or waste water treatment plant.

Therefore, the local and state governments have been working intensively on Radika River Valley Environmental Protection for a long time. The Bureau for Economic Undeveloped Areas within the Ministry of Local Self-Government of the Republic of Macedonia, has undertaken a number of activities during a longer time period connected to the protection of the River Radika basin. Until now, many documents and different design documentation have been prepared with a main goal to create conditions for an appropriate protection of the complete River Radika basin [1], [2], and [3]. After the main design for the solid waste landfill in the municipality of centar zupa has been finished [3], the need for design and construction of appropriate leachate treatment and waste water treatment plant has been considered. Consequently, preliminary analyses of the variants for landfill leachate treatment have been performed, consisted of technical and economical analyses.

The main goal of the technical analyses is to study technical possibilities for construction of a mutual leachate and waste water treatment plant, as a basis for further design phases. Analyses include also: leachate treatment site selection, selection of alternatives for transport of leachate, selection of technological scheme of the treatment plant (for combined leachate and domestic waste water), waste water treatment structures and estimation of the investment costs for the leachate and waste water treatment plant. The analyses of the leachate treatment site selection are consisted of two alternatives: at solid waste landfill Centar Župa site (treatment of leachate at the landfill site) and at the municipal waste water treatment plant for Centar Župa site (treatment of leachate in combination with domestic sewage at the waste water treatment plant). Also, two alternatives for transporting of leachate to the selected waste water treatment site are analyzed: use of tankers and construction of pressurized pipe (force main) [4].

In parallel with the technical alternatives for leachate treatment, three alternatives for integral solid waste and water management in the view of costs estimation and set up tariffs for collecting and disposing of solid waste are presented. Based on economic analysis the most favorable scenario is found for the public utility enterprise and for sustainability of the investment [4].

2 Leachate Treatment Site Selection

While performing the analyses for site selection of leachate treatment, two alternatives were taken into consideration:

- at solid waste landfill Centar Župa site (treatment of leachate at the landfill site)
- at the municipal waste water treatment plant for Centar Župa site (treatment of leachate in combination with domestic sewage at the waste water treatment plant).

Considering the local topographical conditions, the available area at the landfill site and the necessary area for landfill construction it has been concluded that there are spatial constrains for locating the leachate treatment plant at the landfill site. As an alternative to the leachate treatment at landfill site, the treatment of leachate in combination with domestic sewage at the waste water treatment plant (WWTP) for Centar Župa is proposed. Treatment of leachate at the WWTP is a potential disposal opportunity because the access to the WWTP is available in the vicinity of the landfill site, presented on Fig. 1.

Fig. 1. Review map of the landfill and waste water treatment site

3 Transport of Leachate

Two alternatives for transporting of leachate to the selected WWTP site are analyzed:

- use of tankers, and
- constructing of pressurized pipe (force main).

Also, analyses of the possibility of transporting the waste water from settlements in the municipality of Centar Župa by gravity to the landfill site and treating the waste water in combination with leachate were performed. Taking into consideration the topographical conditions at the landfill and at the WWTP site it has been concluded that it is not possible to transport the waste water by gravity to the landfill site. Regarding already mentioned spatial constrains at the landfill site, the only possible location for WWTP (combined for leachate and waste water treatment) is in the vicinity of village of Centar Župa, i.e., the already considered location (Fig. 1-location No 1) by the municipality of Centar Župa authorities.

Transport of leachate from the landfill to the WWTP by use of tankers will be on a local road from the landfill with a length of 1250 m and on the asphalt road (road to the municipality Centar Župa) to the WWTP with a length of 3700 m. For the estimated leachate generation of 25 m^3/day, 5 tankers with capacity of 5 t each should be used to transport the leachate to the WWTP in Centar Župa.

The other alternative for transport of leachate to WWTP is by construction of pressurized pipe (force main) with length of around 4700 m. The alignment of the pressurized pipe is presented on Fig. 1. It is suggested that pipe should be from polyethylene material PE 100 with outside diameter OD 90 mm for pressure up to 10 bar. At the landfill site a pump with capacity of 1 l/s and total operating pressure head of 85 m is planned for pumping of the leachate from the reservoir. Two reservoirs are designed with dimensions 3*3*3 m each [3]. The effective volume is around 2,65*2,65*2 m= 14 m^3 for one reservoir or around 28 m^3 for both reservoirs. The capacity of the pump should allow pumping of the maximal quantities of leachate which will appear in the landfill filling phase. Two pumps are designed for the first phase and in the exploitation phase there should be one reserve pump. There is a recommendation to use mobile pump for aggressive medium, which can be dismantled and can descended into the leachate reservoirs. This pump should be equipped with complete electrical system and possibility of automatically exclusion. The pump total operating pressure head of 85 m is calculated from the bottom of collecting reservoirs (level 664,30 m a.s.l.) to the maximal elevation of the pressurized pipe (force main).

In the previously designed book 3 -main design for the Solid Waste Landfill in the Municipality of Centar Župa [3], an evaporation pool (lagoon) for evaporation of leachate is designed from where the leachate will be directed for sprinkling over the landfill body. The aim of the evaporation pool is to reduce the quantity of leachate, and the surplus leachate will be used for intensive processes of biochemical anaerobic decomposition of the waste in the landfill body. Excess leachate that remains in the evaporation pool is planned to be transported to the WWTR of Centar Župa.

Since the greatest quantities of leachate production occur during the colder period of the year when monthly values of precipitation show higher values and the evaporation is low, it is considered that the pumps should be selected to have total operating pressure head for pumping leachate from the leachate reservoirs. Due to the analyzed alternative for pumping of the leahate to the WWTP in Centar Župa, the purpose of the leachate reservoirs and the evaporation pool is modified. Therefore, it is recommended to recalculate pump flow and total operating pressure head and the necessity for evaporation pool construction in the forthcoming design phases.

Also annual consumption of energy (E) for pumping of leachate from the leachate reservoirs to WWTP is estimated on the basis of pump total operating pressure head (h) and annual generation of leachate (W). The annual energy consumption has been estimated for the exploitation period of 25 years. It is in the range from 1000 kWh to

3000 kW per year, depending on the annual leachate generation. The average annual energy pump consumption is estimated on 2200 kWh.

4 Leachate Generation

Leachate generation for Centar Župa landfill has been estimated within the Environmental Impact Assessment Study for the Solid Waste Landfill in the Municipiality of Centar Župa [1].

The calculations of leachate generation for certain periods of the exploitation of the landfill were based on the climatic and meteorological conditions, as well as parameters from the practical exploitation of sanitary landfills in developed countries, the generating intensity of the leachate production and intensity of precipitation infiltration in the landfill. Taking into consideration leachate recycling, the results from this Study that are presented in Table 1 indicate that in case of realizing the suggested work concept, the leachate generation will range from 0,023 to 0,17 l/s.

Table 1. Intensity of leachate generation in different periods of exploitation of landfill

Period	Intensity of leachate generation		
	l/sec	l/h	m^3 /day
2007-2009	0,092	331,2	7,9
2010-2012	0,148	532,8	12,8
2013-2015	0,161	579,6	13,9
2016-2029	0,169	608,4	14,6
2020-2023	0,145	522	12,5
2024-2026	0,114	410,4	9,8
2027-2028	0,084	302,4	7,3
2029	0,059	212,4	5,1
2030	0,038	136,8	3,3
2031	0,023	82,8	2,0

Source: Environmental Impact Assessment Study for the Solid Waste Landfill in the Municipality of Centar Župa

The scenario for the future exploitation of the landfill indicates that in case of realizing the suggested work concept, the leachate generation will be limited on one to three tanks per day (i.e. maximal volume in most critical conditions of around 15 m^3/day). From this reason, in the Study it has been proposed that the leachate should be treated in the nearest municipal waste water treatment plant i.e. in the future WWTP of Centar Župa.

It has been expected that the intensity of generation of the leachate is going to be considerably changed in cases of extreme weather conditions. The intensity of leachate generation for average, maximal and minimal precipitation for Debar meteorological station has been performed in the Study. Maximal values of the leachate are observed for the layer 5 of the landfill. They vary in the range that is presented in Table 2.

In order to verify the estimated quantity of generated leachate and also to determine a value that is acceptable for wastewater treatment plant design, the amount of leachate that will be generated within the landfill has been estimated once again using the *Water Balance Method (WBM)* and *Hydrologic Evaluation of Landfill Performance*

(HELP) method. Using climate and meteorological data for the nearest meteorological station in Debar presented in table 3 and in table 4 and the WBM, the annual generation of leachate has been estimated for period of 28 years.

Table 2. Intensity of leachate generation with minimal and maximal annual precipitation (in l/s)

Evaluated process parameter	Unit measure	Layer 1	2	3	4	5	6	7	8	9	10
Intensity of leachate generation with annual average precipitation in period 1961- 2004	l/s	0,09	0,15	0,16	0,17	0,14	0,11	0,08	0,06	0,04	0,02
	m^3/ day	7,9	12,8	13,9	14,6	12,5	9,8	7,3	5,1	3,3	2,0
Factor of saturation (FK)		0,54	0,43	0,36	0,31	0,27	0,25	0,25	0,25	0,25	0,25
Intensity of leachate generation with annual average precipitation in period 1961- 2004 (with included flow)	l/s	0,16	0,24	0,25	0,26	0,22	0,17	0,12	0,09	0,06	0,03
	m^3/ day	13,7	20,6	21,9	22,5	19,1	14,8	10,8	7,5	4,8	2,8
Factor of saturation (FK)		0,54	0,42	0,35	0,29	0,26	0,23	0,23	0,23	0,23	0,24
Intensity of leachate generation with minimal annual average precipitation 1961	l/s	-0,02	-0,004	0,01	0,02	0,02	0,02	0,01	0,01	0,008	0,006
	m^3/ day	-1,6	-0,3	0,9	1,4	1,6	1,5	1,2	0,9	0,7	0,5
Factor of saturation (FK)		0,54	0,45	0,39	0,35	0,31	0,29	0,29	0,29	0,29	0,29
Intensity of leachate generation with maximal annual average precipitation 1981	l/s	0,25	0,37	0,38	0,39	0,33	0,25	0,18	0,13	0,08	0,04
	m^3/ day	21,9	31,8	33,0	33,6	28,3	21,8	15,9	10,9	6,9	3,9
Factor of saturation (FK)		0,54	0,40	0,33	0,27	0,23	0,21	0,21	0,21	0,21	0,22

Source: Environmental Impact Assessment Study for the Solid Waste Landfill in the Municipality of Centar Župa"

It has been estimated that the maximal amount of 8630 m^3 leachate per year might be generated after 20 years of exploitation of the landfill. Results of the annual generation of leachate are presented in Figure 2. It has been estimated that the maximal value of generated leachate per day amounts 23,7 m^3/day.

The Hydrologic Evaluation of Landfill Performance (HELP) model has been used to determine theoretical leachate generation rates in the landfill. The calculations of the leachate by HELP model were constrained by a lack of daily meteorological data. As the HELP model requires daily data for main meteorological parameters, assumptions for the precipitation and temperature data have been made. Regarding meteorological data it is necessary to specify a nearby city. The nearby city was set to Pittsburgh since it has similar latitude as Centar Župa and it was assumed that both cities would have similar solar radiation. Since there was no vegetation on the landfill the leaf area index was set to zero. Average wind speed and the humidity values are used from the nearby meteorological station in the town of Debar. Beside the various output results from the HELP model, the most important ones for the particular study are average monthly and average daily leachate generation. The monthly average leachate generation results of the HELP model are presented in Table 5. It is very important to note that many of the inputs to this model were approximations, resulting in some uncertainty of the output values.

Table 3. Meteorological data from Meteorological station in Debar (temperature in ^oC: precipitation in mm; distribution of precipitation and relative humidity in %)

Meteorological parameter	I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII	Average
Average monthly and annual air temperature	0,5	2,5	6,5	11,0	15,5	19,2	21,7	21,5	17,6	12,5	7,0	2,1	11,5
Maximal monthly average air temperature,	4,5	7,0	10,7	15,2	20,9	24,8	27,6	27,6	23,7	17,6	11,6	6,2	16,5
Minimal monthly average air temperature,	-2,7	-1,6	1,0	4,8	9,1	12,0	13,7	13,4	10,5	6,2	2,8	-0,8	5,7
Absolute maximal air temperature	17,2	22,5	26,0	28,9	33,4	37,0	40,1	37,8	35,0	30,0	23,7	20,5	40,1[1)]
Absolute minimal air temperature	-19,0	-18,2	-15,0	-2,5	0,0	5,0	6,0	6,0	-1,5	-7,5	-13,0	-19,0	- 19,0[2)]
Average monthly and annual Sum of precipitation, (until 2004)	85,2	74,6	76,6	74,2	74,1	42,8	35,6	37,6	69,8	86,4	111,6	124,9	893,4
	9,5	8,3	8,6	8,3	8,3	4,8	4,0	4,2	7,8	9,7	12,5	14,0	100,0
Average relative humidity	80	78	72	66	67	67	63	62	67	72	77	82	71
Average number of days with fog	2,0	1,0	0,6	-	-	-	-	-	-	0,5	1,2	2,4	7,5

Source: Environmental Impact Assessment Study for the Solid Waste Landfill in the Municipality of Centar Župa

Table 4. Evaporation according to climate data for meteorological station in Debar

Month	I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII
Average (61'-90') mm/day	0,33	0,42	0,67	1,05	1,31	1,57	2,11	2,13	1,40	0,89	0,49	0,29
E(mm/month)	10,04	11,73	20,86	31,64	40,47	47,25	65,39	66,08	42,07	27,72	14,80	8,96
E(mm/year)	387,03											

Source: Environmental Impact Assessment Study for the Solid Waste Landfill in the Municipality of Centar Župa

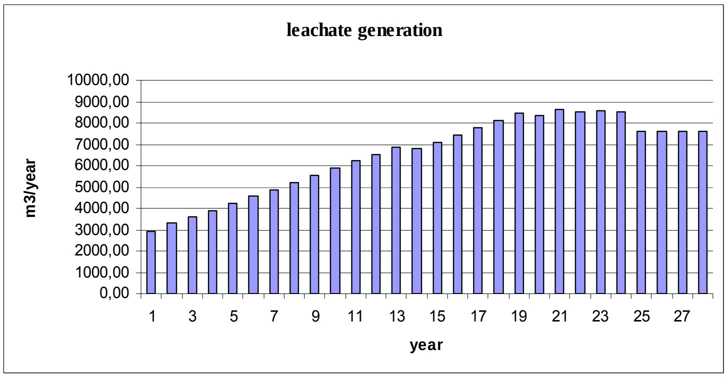

Fig. 2. Annual generation of leachate (m^3 per year) -results of the Water Balance Method

Table 5. Average monthly leachate generation results of the HELP model

Month	I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII
mm/month	52,5	25,8	22,9	24	34	39	27	49	46	58	45	57,8
(m^3/day)	25,4	13,3	11,1	12,0	16,5	18,9	13,1	23,7	23,0	28,1	22,5	28,0

Analysis of the results is showing that the distribution of the monthly values of generated leachate is not fitted to the local climate and meteorological conditions. Taking the average monthly values for precipitation and evaporation from the nearest meteorological station in Debar into the water balance calculations, it is expected that the highest amount of the leachate production is going to occur in the fall, winter, and spring. The summer months are expected to have very low leachate production rate or no leachate generation at all, due to the low precipitation and high evapotranspiration values.

However, the results from HELP model are showing that the average amount of annual leachate generation is around 9000 m^3/year, or 24,7 m^3/day. It is accepted that the most appropriate value of the generated leachate for designing the wastewater treatment plant and other engineering facilities should be set to 25 m^3/day.

It is also very important to note that for the forthcoming design phases, leachate generation should be carefully recalculated using HELP model and daily values of the climate and meteorological data for the nearest meteorological station.

5 Leachate Composition

Composition of leachate varies significantly from landfill to landfill. The most important factors affecting the composition of leachate are: solid waste composition, landfill age (age of waste), operation of the landfill, climate, conditions within the landfill such as moisture content, temperature, pH etc. The same factors influence the types, amounts and production rates of contaminants appearing in the leachate at the landfill site. Landfill leachate shows significant temporal variability in terms of quantity and leachate composition. As it is very difficult to quantify the impact of the factors on composition of the leachate, it is necessary to rely on data and experiences from other landfill investigations.

Design of treatment systems for landfill leachate must consider the variability of the flow rates and the complexity and temporally variability of the leachate composition. For the particular treatment system for landfill leachate the following leachate contamination concentrations were selected [5]: $C_{BOD_5} = 1000\ g/m^3$, $C_{COD} = 5000\ g/m^3$, $C_{TKN} = 200\ g/m^3$, $C_P = 25\ g/m^3$, BOD_5:COD=0,2.

6 Analyses of Sustainability of the System for Collecting and Disposing of Solid Waste

In parallel with the technical alternatives for leachate treatment, three alternatives for integral solid waste and water management in the view of costs estimation and set up tariffs for collecting and disposing of solid waste are analysed.

In a contest of 'decentralization' the effective start up and the sustainability of the operations in the years to come are in charge of the local communities. The established local public utility enterprises, that will facilitate and maintain public infrastructure and services, are expected, by law, to be financed through fee collection from citizens.

The analysis of zero based budget of the two groups of services, municipal solid waste disposal (MSWD) and waste water treatment (WWT), based on provisional data, and highlighted several reasons why the above mentioned sustainability is in danger due to the declared, by the local authorities, limited capabilities of the local population to pay for the costs of the services.

In order to mitigate the risk of an unsuccessful implementation, specific action should be taken by the National Government facilitating the reduction of costs, as consequence of the increase of efficiency, and facilitating also the collection of fees by the Local Authorities.

Sustainability of the system is analyzed through several alternative scenarios, based to different forecasts:

- First scenario forecasts waste collection in Mavrovo and Rostuše, Centar Župa and waste from Debar will only be processed. This scenario is most favorable and most sustainable with 3 € per household;
- Second scenario is waste collection in Rostuše, Centar Župa. Waste from Debar is only processed and Mavrovo is entirely excluded, due to the fact it proceeds disposing in Gostivar. In accordance with this alternative monthly costs per household will be 4.2 €.
- Third scenario predicts waste collection and processing in Rostuse and Centar Župa only. Mavrovo and Debar are excluded, which is unfavorable due to the monthly tariffs per household that reach 10.2 €.

Such a circumstances intended to the conclusion of exploiting the potentials of the "economies of scale" enlarging the area served by the MSWD, if possible, managed by one public municipal enterprise, due to the reduction of administrative costs.

7 Conclusions

As a result of the performed analyses of the combined treatment plant for leachate from the landfill and waste water generated from the inhabitants of the municipality of Centar Župa, the following conclusions can be made:

- The preliminary idea to treat leachate from the landfill and domestic waste water from the settlements in the municipality of Centar Župa was found to be feasible.
- The design of treatment system for leachate and waste water must consider the variability in the leachate flow rates and the complex and temporally varying composition.
- The design of the combined treatment plant for landfill leachate and municipal waste water should be made very carefully in terms of defining the degree to which the influent leachate can be accepted without disrupting the ongoing treatment of the municipal waste water.

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A11**

Wastewater and Landfill Leachate Treatment Plant for the Municipality of Centar Župa

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Abstract. Wastewater from the settlements in the municipality of Centar Župa are discharged into the nearby recipients without any treatment. However, the very low discharge of the rivers, particularly during the summer months results in water quality degradation. For the particular case of the settlements in the municipality of Centar Župa it is necessary to highlight that the location is close to the Debar Lake. As a multipurpose reservoir, beside for energy production, Debar Lake is also used for recreation of the inhabitants of Debar and nearby villages. So, the need for wastewater treatment plant construction is obvious. Also, leachate from the nearby solid waste landfill Centar Župa is planned to be transported and treated together with the wastewater from the settlements. The paper presents the results of the analyses of the wastewater quantities and constituents from the Centar Župa municipality and leachate quantities and leachate composition, mass loading from the wastewater and also from the leachate. Total wastewater quantities and total wastewater flow pollutant concentrations have been calculated. Results of the calculations of the wastewater treatment structures are presented in the paper.

Keywords: Wastewater, landfill, leachate, treatment plants

1 Introduction

Wastewater from the settlements in the municipality of Centar Župa are discharged into the nearby recipients without any treatment. However, the very low discharge of the rivers (particularly during the summer months) results in water quality degradation. For the particular case of the WWTP in Centar Župa it is necessary to highlight that the location is close to the Debar Lake. As a multipurpose reservoir, Debar Lake is also used for recreation of the inhabitants of Debar and nearby villages. So, the need for wastewater treatment plant construction is obvious.

1.1 Site Selection

Wastewaters from the villages: Centar Župa, Bajramovci, Žitinani, Mal Papradnik i Crno Boci is planned to be treated in the Wastewater treatment plant in Centar Župa. Main collectors from Centar Župa and Žitinani are already constructed and gravitate towards location that is planned for construction of the WWTP (labeled as location No. 1 – Fig. 1). Major part of the wastewaters from the other settlements (Mal Papradnik, Bajramovci, Crno Boci) can not reach the WWTP location No. 1 by gravity. Significant part of wastewaters should be pumped in order to be transported to the WWTP location No. 1 and treated.

Location No 1 for WWTP Centar Župa has been selected in previousy performed anlyses. The location is situated about 400 m below the village of Centar Župa in the vicinity of the river Žitinska Reka. The site elevation is 699 -702 m a.s.l. The topographical conditions of the selected site are not favorable for sitting all the necessary structures of the WWTP for treating leachate and wastewaters, mainly due to the great slope of the terrain on the both sides of the river. Also, for sitting of the WWTP structures it is necessary river training works of Žitinska Reka to be performed. A great part of the wastewaters from the villages situated below the location No. 1 (Fig. 1) should be transported to the WWTP by pumping.

The designers recommend further analysis of alternative location (labeled as location No. 2) as possible location of WWTP for the next design phase.

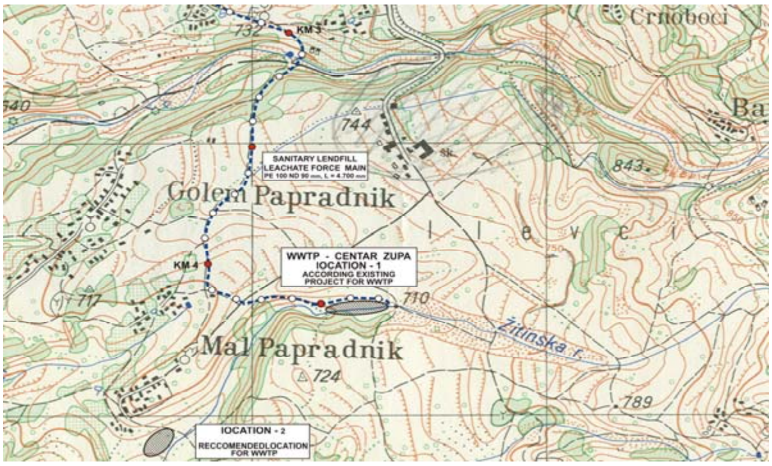

Fig. 1. Review map of possible location for WWTP "Centar Župa"

The recommended location No. 2 is situated below the village of Mal Papradnik at 670 m above sea level. Compared to the location No. 1, alternative location No. 2 (Fig. 1) has the following advantages:

- favorable topographical conditions for sitting all the necessary structures of the WWTP, and
- major part of the wastewaters can be transported to the location of the WWTP by gravity (no need of pumping).

According to the available data, there is a necessity of additional construction of around 800 m length wastewater collector from Centar Župa and Žitinani.

1.2 Wastewater Constituents

Typical data on the total quantities of wastewater discharged per capita per day (dry weight basis) from individual residences are: BOD_5 = 60 (g/capita/day), COD = 120 (g/capita/day), N = 11 (g/capita/day), P= 1.8 (g/capita/day)

Wastewater flow pollutant concentrations are the following: $C_{BOD_5} = 300 \quad g/m^3$, $C_{COD} = 600 \quad g/m^3$, $C_{TKN} = 55 \quad g/m^3$, and $C_P = 9 \quad g/m^3$

1.3 Effluent Quality

The WWTP effluent quality is suppose to meet EU Directive (91/271/EEC) standards and existing national legislation -Water Law (Off. Gazette of RM, No. 4/98). According to the standards the effluent should fulfill the standards of $BOD \leq 25 \quad (mgO_2/L)$ and $TSS \leq 35 \quad (mg\, SS/L)$. For the particular case of the WWTP in Centar Župa which is close to the Debar Lake, a lake with high water quality, the biological treatment is designed for BOD removal and nitrification.

2 Wastewater Quantities and Constituents from Communal Wastewater and Leachate from Sanitary Landfill

Wastewater treatment plant in Centar Župa is planned to treat wastewaters from the villages: Žitinani, Centar Župa, Bajramovci, Crno Boci and Paparadnik. The number of population growth until 2030 are 3000 equivalent peoples.

Wastewater flow rates can vary depending on the quantity and quality of the water supply; rate structure; and economic, social, and other characteristics of the community. For the Centar Župa region, the wastewater norm per capita per day is assumed on 200 l/capita/day.

Design Flow rates. The development and forecasting of flow rates is necessary in order to determine the design capacity as well as the hydraulic requirements of the waste treatment plant. Flow rates are predicted for the time horizon 2030 as end of the planned period (2030).

The flow rate from various case and components are present in Table 1. In Table 2 are presented the constituent of mass loading for various cases of BOD_5, COD, TKN and P, while in Table 3 are presented the constituent concentration for various case. In the previous three table are present flow rate, mass loading and constituents concentration for communal wastewater, leachate from sanitary landfill and together dilute water from communal wastewater and leachate, also in Table 3 are presented BOD5/COD ratio.

Table 1. Flow rate for various case

	Q_{aver}	dry flow $Q_{max/d}$	wet flow $Q_{max/d}$	Q_{min}
	(m^3/d)	*(m^3/h)*	*(m^3/h)*	*(m^3/h)*
Community	600	33,33	66,67	16,25
Leachate	25	1,05	3,6	0
Together	625	34,38	70,27	16,25

Table 2. Constituent of mass loading for various case

	BOD_5	COD	TKN	P
	(kg/d)	*(kg/d)*	*(kg/d)*	*(kg/d)*
Community	180	360	33	5,4
Leachate	25	125	5	0,625
Together	205	485	38	6,025

Table 3. Constituent concentration for various case

	C_{BOD5}	C_{COD}	C_{TKN}	C_P	BOD/COD
	(mg/L)	*(mg/L)*	*(mg/L)*	*(mg/L)*	*(%)*
	Average flow				
Community	300	600	55,0	9,00	0,50
Leachate	1000	5000	200,0	25,00	0,20
Together	328	726	60,8	9,64	0,45
	Maximum dry weather flow				
Community	225	450	41,3	6,75	0,50
Leachate	1000	5000	200,0	25,00	0,20
Together	249	587	46,1	7,31	0,42
	Maximum wet weather flow				
Community	113	225	20,6	3,38	0,50
Leachate	1000	5000	200,0	25,00	0,20
Together	126	298	23,8	3,70	0,42

Source: McBean Edward, Rovers F.A., Farquhar G.J., 1995, Solid Waste Landfill Engineering and Design, Prentice Hall

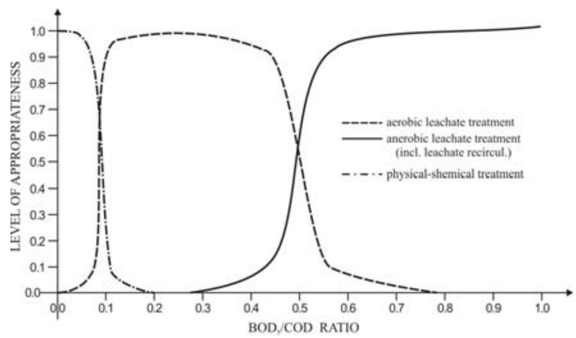

Fig. 2. Proposed appropriateness levels for biological and physical-chemical leachate treatment

As BOD_5:COD ratio for leachate water is in the range of 0.2, until BOD_5:COD ratio communal wastewater and dilute water from communal wastewater and leachate is about 0.42 to 0.5. According to the previous values and the graph from Fig. 2 (McBean *et al.*), aerobic biological treatment is appropriate.

3 Technological Scheme of the Wastewater Treatment Plant

The technological scheme of the wastewater treatment plant for combined treatment of domestic wastewaters from the settlements and leachate from the solid waste landfill is made according to the consulted literature for such treatment, expert knowledge, local conditions and under the expert consultancy provided by the donor. The main aim of the technological scheme is to enable treatment of leachate in combination with domestic sewage at the wastewater treatment plant in Centar Župa using more simple treatment techniques in the terms of operation and maintenance.

The technological scheme of the wastewater treatment plant for the wastewaters from the settlements and the leachate from the landfill is consisted of separate processes and facilities. The treatment system include: fine screening, grit chamber, imhoff tank, trickling filter, secondary clarifier, disinfection with contact chamber, and drying beds (Fig. 3).

Also other operations and facilities like: pump stations, flow meter chamber, distributor chamber, pipe network for treated water, recirculation, primary and secondary sludge are included in the WWTP.

3.1 Leachate Tank

Leachate aeration tank (No. 3 in Fig. 3) is designed for two days accumulation of the leachate from the municipal solid waste landfill in Centar Župa. Leachate is aerated in the leachate tank and then discharged before the fine screens. Leachate is transported from the municipal solid waste landfill in Centar Župa toward WWTP by pressurized pipe. The pipe diameter is designed to be OD 90 mm and the length is around 4700 m.

3.2 Fine Screening

Fine screens (No. 5 in Fig. 3) are designed to hold floating materials, because these floating materials might interfere the following treatment operations. The screened material, is discharged and sacked in a suitable container and than transported into the municipal solid waste landfill Centar Župa.

Fine screen type FC Screw filter is recommend. The screw filter is suitable for many applications, in particular for the micro-screening. The type FC consists of one multifunction screw and a semi-cylindrical filtering screen, which is, in its turn, formed by longitudinal wedge wire bars. The screw is controlled by a sturdy gear motor ensuring the filtering screen cleaning, the lifting, compacting and sacking of the screened material.

The screened material, before being discharged and sacked in a suitable container, is submitted to compaction and de-watering involving an about 50% decrease in weight. The screw filter is able to totally eliminate the suspended solids having diameter bigger than the size of the filtering gap, and considerably cut down the polluting load.

3.3 Grit Chamber

Grit Chamber (No. 6 in Fig. 3) is designed for separation of solid inorganic particles, mainly for sand removal. The type of the recommended grit chamber is Vortex-type grit chamber. Wastewater enters and exits tangentially. The rotating turbine maintains constant flow velocity, and its adjustable pitch blades promote separation of organics from the grit. The action of the propeller produces a toroidal flow path for grit particles. The grit settles by gravity into the hopper in one revolution of the basin's contents. Solids are removed from the hopper by a airlift pump.

The sand is pumped towards sand compartment (No. 7), and than transported into the municipal solid waste landfill Centar Župa.

3.4 Primary Sedimentation Tank

Primary sedimentation – Imhoff tank (No. 9 in Fig. 3) is a primary sedimentation process which performs two functions, the removal of settleable solids and the anaerobic digestion of those solids. The Imhoff tank consists of a two-story tank in which sedimentation occurs in the upper compartment and the settled solids are deposited in the lower compartment. Solids pass through a horizontal slot at the bottom of sloping sides of the sedimentation tanks to the unheated lower compartment for digestion.

The Imhoff tank has no mechanical parts and is relatively easy and economical to operate. It provides sedimentation and sludge digestion in one unit and should produce a satisfactory primary effluent with a suspended solids removal of 40% to 60% and a BOD reduction of 15% to 35%. The two-story design requires a deep overall tank. The Imhoff tanks is best suited to small municipalities and large institutions where the tributary population is 5,000 or less.

Also sludge from secondary clarifier (No. 19 in Fig. 4) enters into Imhoff tank. Stabilized sludge is pumped from the Imhoff tank towards drying beds (No. 20). The aim of the drying beds is to dewater the sludge. The drained water from the drying beds is returned into the treatment process (No. 21) at the pump station (10).

3.5 Trickling Filters

Trickling filters (TFs) (No. 11 in Fig. 3) are used to remove organic matter from wastewater. The TF is an aerobic treatment system that utilizes microorganisms attached to a medium to remove organic matter from wastewater. These systems are known as attached-growth processes.

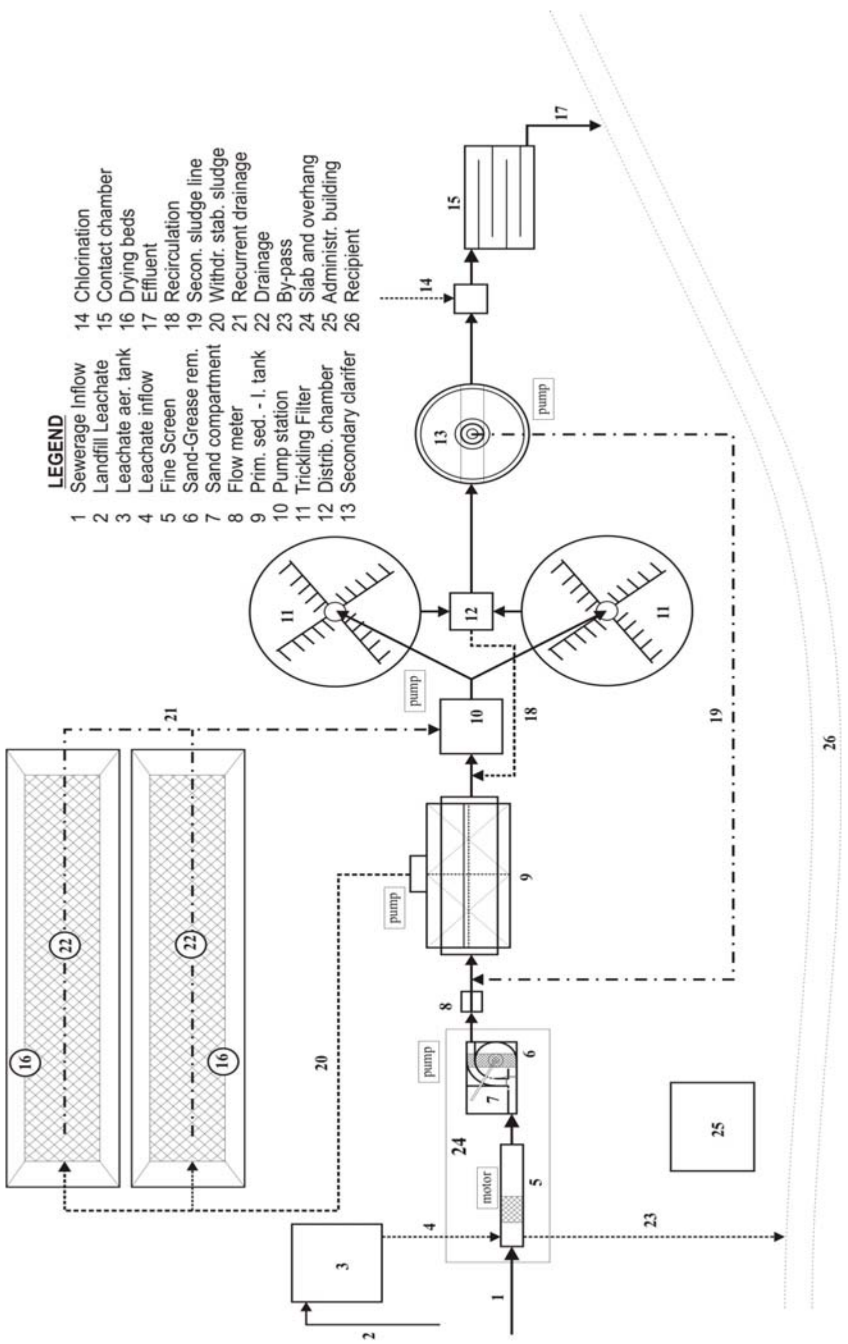

Fig. 3. Schematic flow diagram of WWTP Centar Župa

TFs enable organic material in the wastewater to be absorbed by a population of microorganisms (aerobic, anaerobic, and facultative bacteria; fungi; algae; and protozoa) attached to the medium as a biological film or slime layer (approximately 0.1 to 0.2 mm thick). As the wastewater flows over the medium, microorganisms already in the water gradually attach themselves to the rock, slag, or plastic surface and form a film. The organic material is then degraded by the aerobic microorganisms in the outer part of the slime layer. As the layer thickens through microbial growth, oxygen cannot penetrate the medium face, and anaerobic organisms develop. As the biological film continues to grow, the microorganisms near the surface lose their ability to cling to the medium, and a portion of the slime layer falls off the filter. This process is known as sloughing. The sloughed solids are picked up by the under drain system and transported to a clarifier for removal from the wastewater.

Design criteria. A TF consists of permeable medium made of a bed of rock, slag, or plastic over which wastewater is distributed to trickle through. Rock or slag beds can be up to 60 m in diameter and 0.9-2.5 m deep with rock size varying from 2.5-10.2 cm. Most rock media provide approximately 150–40 m^2/m^3 of surface area and less than 40 percent void space.

The design of a TF system for wastewater also includes a distribution system. Rotary hydraulic distribution is usually standard for this process. Recently some distributors have been equipped with motorized units to control their speed. Distributors can be set up to be mechanically driven at all times or during stalled conditions.

In addition, a TF has an under drain system that collects the filtrate and solids, and also serves as a source of air for the microorganisms on the filter. The treated wastewater and solids are piped to a settling tank where the solids are separated. Usually, part of the liquid from the settling chamber is re-circulated to improve wetting and flushing of the filter medium, optimizing the process and increasing the removal rate.

Typical application, process loadings criteria, and effluent quality are summarized in Table 4.

Table 4. Loading based on packing surface area

Application	Loading		Effluent quality	
	unit	range	unit	range
BOD removal	kgBOD/m^3 d	0.3-1.0	BOD, mg/L TSS, mg/L	15-30 15-30
Combined BOD removal and nitrification	kgBOD/ m^3 d	0.1-0.3	BOD, mg/L	< 10
	gTKN/ m^2 d[(a)]	0.2-1.0	NH4-N, mg/L	< 3

Intermediate trickling filter with rock size (30 to 50 mm) packing with specific surface area (80 to 100) m^2/m^3, and filter depth 2.5 m is recommended.

Design for BOD removal and Nitrification. The volume of rock packing required for 70 % TKN removal in trickling filter with depth of 2.5 m for the given wastewater characteristic is determined.

The specific area of rock packing material is assumed on 90 m^2/m^3. Hydraulic application rate and mass loading are presented in Table 5.

Table 5. Hydraulic application rate and mass loading

Wastewater characteristics		
parametar	unit	value
Flow	m^3/d	925
BOD_5	g/m^3	161
TKN	g/m^3	41.45
TSS	g/m^3	70

Determine the specific TKN removal rate:

$$R_n = 0.82 \cdot \left(\frac{BOD}{TKN}\right)^{-0.44} = 0.82 \cdot \left(\frac{161}{41.45}\right)^{-0.44} = 0.82 \cdot (3.884)^{-0.44} = 0.45 \quad (1)$$

Determine the TKN removal in dry weather:

$$Q_{\max/d} = Q_{av/d} \cdot k_{\max} + 25 = 600 * 1.33 + 25 = 825 \quad (m^3/d) \quad (2)$$

$$TKN \text{ remov} = 0.70 \cdot 41.45 \cdot 825 = 23937.375(gTKN/d) \quad (3)$$

Determine the required surface area (m^2) of packing:

$$A_S = \frac{23957}{R_n} = \frac{23937.375}{0.45} = 53194.17 \quad (4)$$

Determine the volume (m^3) of packing material:

$$Vol = \frac{53194.17 \text{ m}^2}{90 \text{ m}^2/m_3} = 591 \quad (5)$$

Determine the horizontal surface (m^2) of trickling filter:

$$A = \frac{volume}{depth} = \frac{591}{2.5} = 236.40 \quad (6)$$

- Hydraulic loading in dry weather:

$$q = \frac{Q}{A} = \frac{825}{236.40} = 3.49 \quad (m^3/m^2 d) \text{ or } (0.040\ L/m2\ s) \quad (7)$$

- Hydraulic loading in wet weather:

$$q = \frac{Q}{A} = \frac{1625}{236.40} = 6.87 \quad (m^3/m^2 d) \text{ or } (0.0796\ L/m2\ s) \quad (8)$$

- Diameter of Trickling filter:

$$D=\sqrt{\frac{4\cdot A}{2\cdot 3.14}}=\sqrt{\frac{4\cdot 236.4}{2\cdot 3.14}}=12.27 \quad (9)$$

Two trickling filters with diameter each D = 12.5 (m) are accepted.
To meet the minimum hydraulic application rate of 0.5 (L/m^{2}s) recirculation will be required.
Determine the BOD loading based on volume and surface area:

- Loading based on volume:

$$BOD\,load=\frac{0.7\cdot 205\; kgBOD/d}{\frac{12.5^2\cdot 3.14}{4}\cdot 2\;\; m^3}=\frac{0.7\cdot 205\; kgBOD/d}{613.28\;\; m^3}0.23\;\;(kgBOD/m^3d) \quad (10)$$

- Loading based on area:

$$BOD\,load=(0.23\;\; kgBOD/m^3d)\left[1/90\;\; m^2/m^3\right]\cdot(10^3\;\; g/kg)=2.55\;\;(gBOD/m^2d) \quad (11)$$

Determine the volumetric oxidation rate (VOR):

- VOR in dry weather

$$VOR=\frac{\left[S_o+4.6\cdot\left(NO_x\right)\right]\cdot Q}{V\cdot\left(10^3\;\; g/kg\right)}=\frac{\left[161+4.6\cdot(0.8\cdot 41.45)\right]\cdot 825}{613\cdot 1000}=0.42\;\;(kg/m^3d) \quad (12)$$

- VOR in wet weather

$$VOR=\frac{\left[S_o+4.6\cdot\left(NO_x\right)\right]\cdot Q}{V\cdot\left(10^3\;\; g/kg\right)}=\frac{\left[161+4.6\cdot(0.8\cdot 41.45)\right]\cdot 1625}{613\cdot 1000}=0.83\;\;(kg/m^3d) \quad (13)$$

The computed value for BOD loading, based on the packing material surface area, is within the range reported by Parker and Richards (1986). The computed value for VOR in wet weather is in range from 0.75 – 1.0 (Daigger, 1994), but in dry weather is little lower.
According to the previous calculations, and also taking into consideration both BOD removal and Nitrification, two intermediate trickling filters with diameter D=12.5 m with height of filter media 2.5, and recirculation R=1 are accepted.

3.6 Secondary Clarifier

Secondary clarifier (No. 13 in Fig. 3) which receive the biologically treated flow undergo zone or compression settling. Sludge from the bottom of the sludge compartment is re-circulated (No. 18) before Imhoff tank. The constitutive part of the secondary clarifier is denitrification compartment. Denitrification process enables oxidation of nitrates in gas nitrogen.

3.7 Disinfection

Chlorination (No. 14 in Fig. 3) is the most widely used method of disinfection and is accomplished with gaseous chlorine. For small treatment plants, liquefied chlorine gas is delivered in pressurized containers usually about 45 kg cylinders.

The design of any attendant chlorination facility at a wastewater plant must provide automatically controlled forced venting of chlorinator and chlorine tank storage rooms, and a chlorine contact chamber (No. 15) with a detention period of not less than 20 min following chlorine injection.

3.8 Dewatering

Dewatering of the sludge is on drying beds (No. 16 in Fig. 3). Drying beds are flat areas separated by banks or concrete partition, where stabilized sludge from Imhoff tank is deposited. The bottom of drying beds will be covered with geomembrane (synthetic foundation protective layer). Drainage layer will be set on the bottom of the drying beds. A drainage pipe (No. 22) for drained water collection and transportation will be set.

Drained water from the drying beds is recirculated into the treatment process (No. 21). Dewatered sludge is transported and disposed of into the municipal solid waste landfill in Centar Župa.

3.9 Other structures

Wastewater treatment plant is designed with all surrounding facilities for small WWTP such as: administration building with laboratory (approximate area of 50 m^2). Considering topographical and hydrological conditions of the terrain at the WWTP site it is necessary river training works of Žitinska Reka to be performed. In addition, corrections of the alignment of the access road and construction of internal road are necessary.

4 Conclusions

As a result of the performed analyses of the complex treatment plant for waste water and leachate, the following conclusions can be made:

- There are high oscillations of the leachate quantity and quality during time. Leachate contains many constituents and its quality is multidimensional. Due to that, when designing leachate treatment plant, special attention should be paid on the leachate design parameters in terms on its quantity and quality.
- Selection of the technological scheme of the wastewater treatment plant for combined treatment of domestic wastewaters and leachate from the solid waste landfill should be made very carefully, taking into consideration the latest scientific achievements, expert knowledge, local conditions and the necessity for operation and maintenance.

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International Symposium on Water Management and Hydraulic Engineering

Ohrid/Macedonia, 1-5 September 2009

Paper: **A12**

Application of Computer Simulation in Assisting of Water Supply System Management

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Abstract. In this paper the procedures for the construction of the calibrated hydraulic model of the water supply system of Wroclaw city are presented. The model was built using the EPANET 2 computer program, which included such parameters as time horizon of simulation and step allowing testing water supply system (WSS) under actual dynamic conditions. This hydraulic model of WSS is valuable analytical tool for the investigation of diversity of simulated random events that may occur during its operating. Taking into account the results obtained use of the model it is possible to establish the rules of rational in-service operation of water-pipe network. Using the results of computer analyses which simulate the functioning of the water distribution system under both normal and extreme condition offers the possibility of taking steps that will minimize the effects of unexpected random events and, in consequences, reduce the risk which the management of WSS will have to taken.

Keywords: Water supply system (WSS), water pipe network, hydraulic model, calibration of dynamic model, Thiele's inconsistent statistics

1 Introduction

For current operation conditions of water supply system (WSS), shaped among others by constant fall in water demand, widely interpreted reliability operation management of a water pipe system gets significant meaning: on one side ensuring high quality of the product, on the other side establishing acceptable water price level. Recently, ensuring of high system reliability, concerning amount as well as quality of delivered water to a consumer, become a significant factor determining the product price. Quality aspect results from a fact that water as a fundamental life element cannot constitute any harm for a consumer health.

The quality management system ISO 9001:2000 standard as well as notations of article 8 of Council Directive 98/83/EC lay upon water pipe enterprises a duty of analysis of widely understood risk concerned to both water consumer and water producer. Producer risk is shaped by random events causing a disruption of regular system exploitation conditions resulting in financial losses. Risk management of water pipeline systems requires identification of the most important factors shaping possibilities of not meeting the task charged water pipeline enterprises by local societies to water delivery. This management requires continuous solving of complex WSS operation problems, where supporting decisions making tool are computer simulations.

During a few last years, wide choices of variety and professional computer programs were arisen. They are very supportive and useful for design and management of urban water supply system and sewage system. Present level of computer techniques allows using them in estimation of water-pipe systems operating and in analysis of their critical elements, as aspect of technological process and engineer solutions [1], [2], [3], [4], [5], [6], [7], [8]. They allow simulating different variants of random events of different probability of occurrence. More often they are using to hydraulic analysis of designed and actual operating water distribution subsystem and frequency determine of taking water sample for quality analysis. It has made a possibility of determining critical points in the production subsystem as well as in the distribution subsystem, which failure states may be the highest risk of not fulfilling basic tasks of the system, which is ensuring a continuous water supply to buyers. Making use of computer analysis of results gives opportunity of taking steps that minimize effects of that events, and what is more, minimizing risk taken by water-pipe enterprises and increasing consumer safety. The hydraulic model may also be of help in making decision about how to modernize or develop the water-pipe network in rational way. The calibrated hydraulic model forms the basis for the construction of the dynamic quality model. The aim of the quality software packet is to forecast water quality variations during transport to the user under variable conditions of the hydraulic functioning of the water-pipe network.

2 Water Supply System of Wroclaw City

The beginnings of Wrocław water pipes are connected with the privilege of the Prince Henry IV the Right given to the inhabitants on 31 January 1272. This privilege let people to take water from the Odra river to supply water pipes and moats and to dump sewage. From the XV century some information appear about next water taps ie. buildings used to take water: The Grate Scooper next to Dolne Młyny, the Maciej Scooper, the Cat Scooper and the Hause of Pump. Water was distributed by wooden water pipes set under the ground at the depth of 4-5 feet. In that way water was supplied to the town until 1871. In this year, the municipal water works “Na Grobli” started to operate. The water-tower and the system of cast iron pipes, equipped with gates and hydrants. The plant supplied water to the town from the Odra river. Water was pumped through sand filters, and after treatment it was directed to the water-tower at the height 38 m, and then gravitationally by a pipe ∅ 762 mm to the municipal water supply system. The daily production was 11000m^3. At present Wroclaw’s

WSS is a source of drinking water for over 680 000 customers. Newsday the system meets the demand for water which is 120,000 cubic meters per day. The current demand for water is equal to 50% of the system disposal capacity. The water supply systems in Wroclaw take water from the Oława River, which is supplied by switching water system from the Nysa Kłodzka River. Water is delivered to Wroclaw by three independent water supply structures (table 1).

The city is divided into four water supply zones. WTP Na Grobli supplies water to about 350 000 inhabitants of so called "inner zone", i.e. populous city center. WTP Mokry Dwor supplies "outer zone" of the city, which encircles inner zone and, through the Bystrzycka pump-station, northeastern suburban (about 265.000 inhabitants). Between these zones there is located so called mixed zone, whereas fourth water supply zone covers settlements placed in Lesnica. This zone is supplied by deep water delivered to about 5.000 inhabitants form WTP Lesnica.

Table 1. Capacities of the water treatment plants for the WSS of the city of Wroclaw

Water Treatment Plant- WTP	Mean production [m^3/d]	Production capacity level [m^3/d]
WTP "Mokry Dwor"	58 000 …	98 000
WTP "Na Grobli"	61 000	120 000
WTP "Lesnica"	1 000	1 100

The water pipeline network of Wroclaw is widespread ringed system of water pipelines of differential diameters of range Ø1400- Ø25 mm. Total length of the water pipeline network is 1 790.41 km. The water pipeline network of the city typifies also high diversification of materials and high variety of exploitation periods of its elements. Fundamental materials being used for construction during 130-years of operation of the WSS were cast iron, steel, PVC as well as lead.

Essential element of the Wroclaw distribution subsystem is central pump station – Bystrzycka, placed in western part of the system. The station ensures operation of high pressure zone, which delivers water to inhabitants of settlements: Nowy Dwor, Kozanow, Gadow, Muchobor Maly. Moreover, in the subsystem of Wroclaw, there are working two zone hydrophone stations - Orzechowa and Krynicka which are to locally elevate water pressure at area of Gaj settlement. Besides technical structures that stabilize hydraulic operation of the water pipeline network are tanks with treated water placed in the area of water treatment stations. There are placed two central tanks and two tanks accumulating water which total capacity is 45 000 m^3 [8].

3 Hydraulic Model of WSS of Wroclaw City

Mathematical hydraulic model of the Wroclaw water supply system was done with use of computer program - EPANET 2. The parameters of hydraulic simulation using EPANET 2 are pressure in the junctions of model and flow. These parameters allow describing hydraulic model of water-pipe network as the function of two model category: junctions and links of model. The model of system's junctions is based on a principle of conservation of mass (principle of the flow continuity) and the model of pipe connections is based on a principle of conservation of energy. Above mentioned principles allow to balance water flow in distribution system. The balance of energy by formula of Bernoulli (1) is the primary rule of mathematical simulation of first group of hydraulic model of water-pipe network [2], [9]:

$$\frac{p_1}{\gamma_w} + z_1 + \frac{v_1^2}{2g} = \frac{p_2}{\gamma_w} + z_2 + \frac{v_2^2}{2g} + \sum h \tag{1}$$

where:

p_1, p_2 pressure in node 1and 2 (N/m^2)
γ_w specific gravity of water (N/m^3)
z_1, z_2 geometrical head of node 1 and 2 (m)
v_1, v_2 mean velocity in node 1 and 2 (m/s)
g gravitational acceleration (m/s^2)
Σh sum of head loss (m)

The second group of hydraulic model equations of water distribution system (2) is mass balance formulas in nodes of network [2], [9]:

$$\forall n \in N: \sum_{\substack{l \in L_{n+} \\ l \neq l_{d_n}}} Q_l(t) - \sum_{\substack{l \in L_{n-} \\ l \neq l_{p_n}}} Q_l(t) = Q_{l_{p_n}}(t) - Q_{l_{d_n}}(t) \tag{2}$$

where:

n node of model
N whole number of model nodes
L model link, L – the whole number of model links
+/- - input and output index
T simulation time (s)
$Q_{l_{p_n}}$ water demand in node (m^3/s)
$Q_{l_{d_n}}$ water delivered to node (m^3/s)
$Q_l(t)$ intensity of water inflow and outflow from link l (m^3/s)

The model of EPANET takes into account only most fundamental water intakes of the system. Nodes of the model were unambiguously defined by defining its space coordinates *x, y* and *z,* amount of daily water demand and demand category. In the process of schematizing topography of the water pipeline network there were taken into consideration all main and distribution pipelines and network terminals of diameter above 100 mm. Complete identification of links of the model were achieved by defining its diameter, length, start and end nodes, material of the link pipe, its age and roughness factor. It was taken into consideration, in the spatial structure of the hydraulic model of Wroclaw, three supply structures: WTP Na Grobli, WTP Mokry Dwor and WTP Lesnica. Next elements of the model structure are central pumping station - Bystrzycka and hydrophone stations - Orzechowa and Krynicka. Mathematical model of the Wroclaw WSS defines patterns of hourly water demand variety, basing on real operation parameters of the system from exploitation period 2004-2006. The above model of Wroclaw WSS composes of 781 nodes, 1053 links, 3 supply sources, 25 pumps and 24 valves (Fig. 1) [8], [9].

The above mathematical model developed during realization of research project in 2005-2006 was subjected to calibration. Compliance evaluation was done with use of the Thiel's inconsistency statistic, through compliance evaluation of simulated time series and real values of pressure at nodes and flow values of links. Fit inconsistency of simulated and real values can result from model weakness or large number of random variables. Taking into consideration the above things, Thiel has divided mean-square error of modeling process into systematic part (mean value and variance) resulting from incorrectly verified dynamic model and into random part shaped by random operation conditions of real dynamic systems (covariance).

$$\delta^2 = \frac{1}{n}\sum_{t=1}^{n}\left(S_t - A_t\right)^2 = \left(\overline{S} - \overline{A}\right)^2 + \left(s_S - s_A\right)^2 + 2(1-r)\cdot s_S \cdot s_A \tag{3}$$

where:

δ^2 mean-square error
N observation frequency
t simulation time
S_t simulation value of parameter at the moment t
A_t real value of parameter at the moment t
$\overline{S}, \overline{A}$ mean value suitably for simulation and real quantity
s_S, s_A standard deviation of simulation and real quantity
r correlation coefficient between simulation and real quantity

The analysis of the consistency of real and simulation values is made for 10 measuring points:

- two nodes delivered water from WTP Mokry Dwor by water main pipe of diameter 1200 mm and from WTP Na Grobli using water main pipe of diameter of 1200 mm too,
- three nodes of central pumping station Bystrzycka including: the input node of water main pipe of 1200 mm diameter and two nodes of output delivered water to the inhabitants of Nowy Dwor housing estate by water main diameter of 800 mm and the Kozanow housing estate by water main diameter of 1000 mm,
- two nodes of hydrophore Gaj supply water by distribution water-pipse in Orzechowa street (diameter 200 mm) and in Krzywoustego street (diameter 250 mm)
- tree nodes of measuring chambers in Boleslawa Krzywoustego street (diameter 600 mm), Krolewska street (diameter 500 mm) and Kosmonautow street (diameter 500 mm).

Values of obtained Thiel's inconsistency characteristics indicate a nonsystematic error (Table 2). The obtained, as a part of a calibration process, average value of mean-square error equals 1.05 m of water (variation range 0.0355 – 5.4234) and corresponds to conventional standards of fitting of hydraulic models of water pipeline networks (absolute error < 2.5 meters of water) of pressure function.

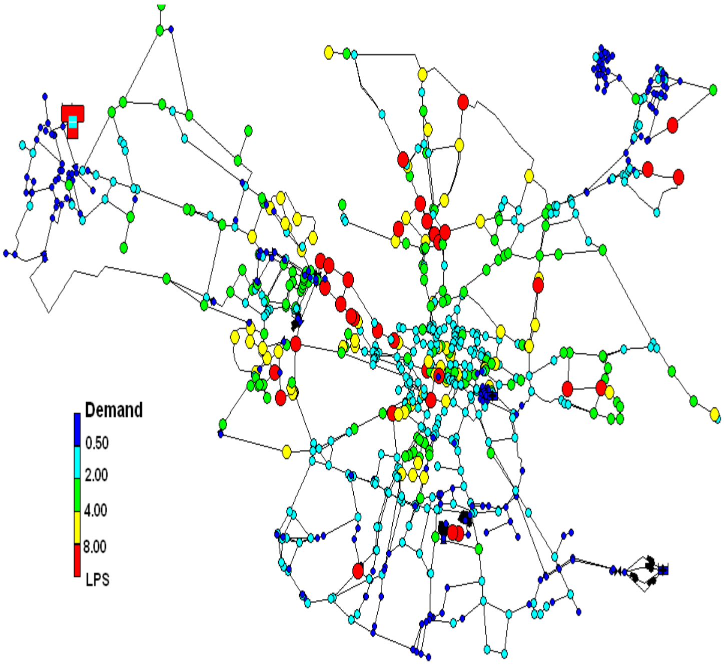

Fig. 1. Hydraulic model of water supply system of Wroclaw: daily water demand

4 Conclusions

The calibration hydraulic model of Wroclaw's water supply system is the useful analytical tool assisting in the management of operation and development of distribution system as well as production system of water. The computer analysis of water supply system operation gives the possibility of taking the most effective steps that minimize results of unexpected random events appearing during its functionality. Moreover, that are useful in taking decision concerning modernization and development range of this system.

This research showed that the calibration method a Thiel's inconsistent statistics based is very useful and dimensional accuracy of hydraulic parameter as: pressure and velocity flow.

The simulation of water supply system operation under extremely operation conditions in the year 2007 was made too. This simulation included minimal and maximal capacity witch appeared in 8th April (92 220 m^3/d) and 28th April (129 280 m^3/d). The obtained results of Thiel's inconsistent statistics give acknowledgement that the hydraulic model is very good and strongly fitting to real water supply system.

The calibrated hydraulic Wroclaw's model forms the basis for the construction of the dynamic quality model. The aim of the quality software packet of EPANET is to forecast water quality variations during transport to the consumers.

Table 2. Thiele's inconsistent statistics for an average capacity of the WSS of Wroclaw city

Measuring points	Thiele's inconsistent statistics		
	mean	variance	covariance
Mokry Dwór_1001	0.041666	0.06166	0.896674
Na Grobli_22	0.104146	0.08004	0.815814
Królewiecka_1319	0.188290	0.03899	0.772720
Krzywoustego_336	0.000001	0.09738	0.902619
Kosmonautów_1262	0.006419	0.000042	0.993539
Orzechowa Orzechowa_a	0.030806	0.015515	0.953679
Krynicka_1086a	0.004397	0.281445	0.714158
Bystrzycka Mokry Dwór_320	0.109598	0.017936	0.872466
Bystrzycka Nowy Dwór_1203	0.051736	0.088069	0.860195
Bystrzycka Kozanów_1202	0.080927	0.068841	0.850232
Mokry Dwór_1001	0.041666	0.06166	0.896674

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A22**

Pilot-Scale Operation of Household Wastewater Treatment Plant with Immersed Membrane Module

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Abstract. The aim of this research is to get a practical experience with operating of a household MBR plant under real conditions with real domestic wastewater that differs from the municipal wastewater. For household MBR plant is typical that the system must to cope with a long-time zero load or vice-versa with more concentrated wastewater in big amounts (i.e. on weekends), extreme temperatures i.e. less than 5-6°C, high pH values and the system is also influenced by the use of detergents.

Keywords: Domestic wastewater, pilot-scale MBR WWTP, membrane fouling

1 Introduction

Decentralized wastewater treatment is used to treat and dispose relatively small volumes of wastewater, generally originating from groups of dwellings and businesses that are located relatively close together, but are not attached to a central sewer system collecting the wastewater up to the wastewater treatment plant (WWTP). The MBR technology integrates biological degradation of wastewater pollutants with membrane filtration, ensuring effective removal of organic and inorganic contaminants and biological material from domestic and/or industrial wastewaters [1].

In this study the treatment plant is fed by real domestic wastewater. In contrast to most other investigations with small-scale WWTPs, the wastewater does not originate from a sewer system. Several difficulties must therefore be overcome: this wastewater is not diluted by rainwater or infiltrated groundwater, it contains hair and particles, the water flow and pollutant load to the plant fluctuates greatly and is not controllable, and neither the wastewater composition nor the concentrations in the raw influent can be measured [2].

2 Description of the Household MBR Plant

Experiment is carried out in the garden of a four-person house. All the wastewater is produced within the house flows through the treatment plant. The plant has no possibility of bypass or emergency overflow. The effluent is stored in a tank and can by used for watering lawn and garden and cleaning floors.

The pilot-scale MBR plant, shown in Fig. 1, consists of three chambers in series; volume of each is 0.58 m^3. The two first chambers are used as a preliminary treatment stage. In these settle chambers the majority of the solids are removed from the raw wastewater by sedimentation. The pretreated wastewater (from settle chambers) flows into the biological activated sludge reactor equipped with immersed membrane module, which parameters are shown in Table 1. Membrane module is from company A3 Water Solutions GmbH. Aeration is provided with fine-bubble aerator (beside the membrane module) and coarse-bubble aerators placed under the membrane module.

A hydraulic retention time (HRT) in whole household MBR plant is 7.2 days, HRT in the preliminary stage is 4.8 days and HRT in the biological reactor is 2.4 days; a volumetric loading is around 0.35 kg COD m^{-3} d^{-1}.

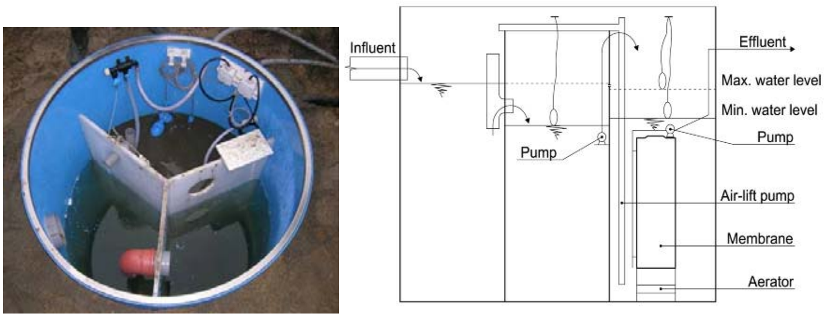

Fig.1. Pilot-scale MBR plant

Table 1. Technical parameters of the membrane module

Parameter	Unit	Value
Membrane type	-	Flat sheet
Membrane material	-	PVDF
Pore size	μm	0.1
Membrane area	m^2	6.7
Membrane parameters	mm	185 x 1090 x 316 (w x h x l)
Pump power demand	W	35 (1. period) , 90 (2. period), 38 (3. period)
Blower	L min^{-1}	80

2.1 Experimental Methods

Most of samples were taken two times per week, on Monday morning (as a weekend load) and Thursday (as work day load). The experiment contained three experimental periods: 1. winter period, 2. spring period, 3. summer and autumn period. All the relevant indicators and parameters were analysed, i.e. quality of influent and effluent, pH, temperature, membrane flux; microbial morphology, physiology and activity of sludge etc.

3 Results and Discussion

3.1 The Quality of Raw Wastewater and Effluent

The studied household pilot-scale MBR WWTP was inoculated by return activated sludge from municipal WWTPs again in each period. The concentration of MLSS was whenever around 1 g L^{-1}. Specific biomass yield (SBY) was during the whole experiment at the level 0.1-0.3 g sludge g^{-1} COD.

As can be seen in Table 2 and in Figure 2 the concentrations of COD in the influent and supernatant quite fluctuated during every period. COD of permeate was relatively sustained value and the average value during monitored season was 53.4 mg L^{-1}. BOD_5 concentrations in effluent varied from 0.2 to 8 mg L^{-1}, the removal efficiency was approximately 99.5%. Although, the initial effluent values of COD (125 mg L^{-1}) and BOD_5 (8 mg L^{-1}) were relatively higher, they fulfilled legislative demands for household WWTP without problems during the whole experiment [3], [4].

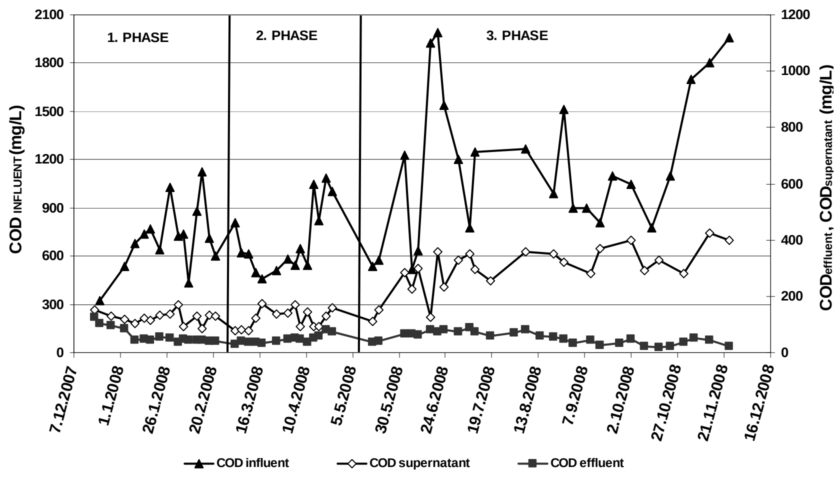

Fig. 2. Comparison of COD values (influent, supernatant, effluent) during three periods

During the first two periods, only a minimal anaerobic degradation of primary sludge took the place in settling tanks (which was confirmed particularly by visual and smelling control of taken samples, the sludge was brown and without decomposition of large particles – beware application of this kind of sludge).

Factors which contribute to these troubles are:

-large particles of primary sludge are not grid in short sewer to the household WWTP;

-during winter and spring period (1st and 2nd period), low temperature had a great influence on the anaerobic degradation in tanks (markedly below 10 ^{0}C);

- high pH (regard to the high concentrations of nitrogen – Fig. 4).

During third period were temperatures higher and anaerobic degradation partly started. Sludge was black and unfortunately started floating and foaming. Gradually decreased pH even more intensifies anaerobic degradation (mainly hydrolysis).

Table 2. The quality of raw wastewater (influent) and the effluent from WWTP

Parameter			COD	BOD_5	NH_4-N	N_{tot}	P_{tot}
1. PERIOD	Influent (mg L^{-1})	average	708,6	496	147,5	200,8	16,7
		min-max	320 - 1125	224 - 787	89,6 - 250	94 - 322	11,1 - 28,9
	Effluent (mg L^{-1})	average	59,4	3	79,6	151,7	11,6
		min-max	37,2 - 125	0,4 - 8	49,9 - 129	80,4 - 238	7,2 - 15,6
		η (%)	91,2	99,4	-	-	-
2. PERIOD	Influent (mg L^{-1})	average	697,8	488,5	166,5	174,7	14,1
		min-max	462 - 1086	323,4-760,2	110 - 226	129 - 208	9,9 - 21,2
	Effluent (mg L^{-1})	average	47,9	2	60,4	123,2	9,0
		min-max	27,9 - 81,8	0,3 - 4	43,1 - 92	67,4 - 149,8	4,11 - 13,25
		η (%)	93,0	99,5	-	-	-
3. PERIOD	Influent (mg L^{-1})	average	1167,7	796,9	157,3	241,3	24,7
		min-max	518 - 2000	362,6-1400	59,4 - 214	153 - 360	11,6 – 40,1
	Effluent (mg L^{-1})	average	52	2,2	14,1	130,0	14,4
		min-max	35,4 - 89	0,2 - 3,6	0,3 - 75,5	61,8 - 198,5	4,1 - 20,3
		η (%)	93,9	99,7	-	-	-
AVERAGE VALUES (all three periods)	Influent (mg L^{-1})	average	917,6	593,8	151,8	213,8	18,6
	Effluent (mg L^{-1})	average	53,4	2,3	44,9	137,1	11,7
		η (%)	92,7	99,5	-	-	-

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and smelling control of taken samples, the sludge was brown and without decomposition of large particles – beware application of this kind of sludge).

Factors which contribute to these troubles are:

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- high pH (regard to the high concentrations of nitrogen – Fig. 4).

During third period were temperatures higher and anaerobic degradation partly started. Sludge was black and unfortunately started floating and foaming. Gradually decreased pH even more intensifies anaerobic degradation (mainly hydrolysis).

3.2 Quality of Activated Sludge and its Parameters

In the activated tank were also observed sludge sedimentation properties. Sedimentation was verified in each period that means with four inoculums. In Figure 3 is shown the evaluation of sludge volume index (SVI) and mixed liquor suspended solids (MLSS). During first two periods was household MBR WWTP inoculated by activated sludge from the municipal WWTP with worse SVI. At the beginning of first period (SVI of inoculum was 210 mL g^{-1}) SVI decreased, but after ca. 1 month the sedimentation rapidly got worse. Subsequently SVI gradually decreased, but only as a consequence of increased MLSS concentration. During the second period was the inoculum even more bulking (SVI 461 mL g^{-1}). SVI gradually decreased in the same way. The real 30 minutes sediments were so high that it was not possible to separate supernatant; and the zone of free liquid above the sludge layer was minimal.

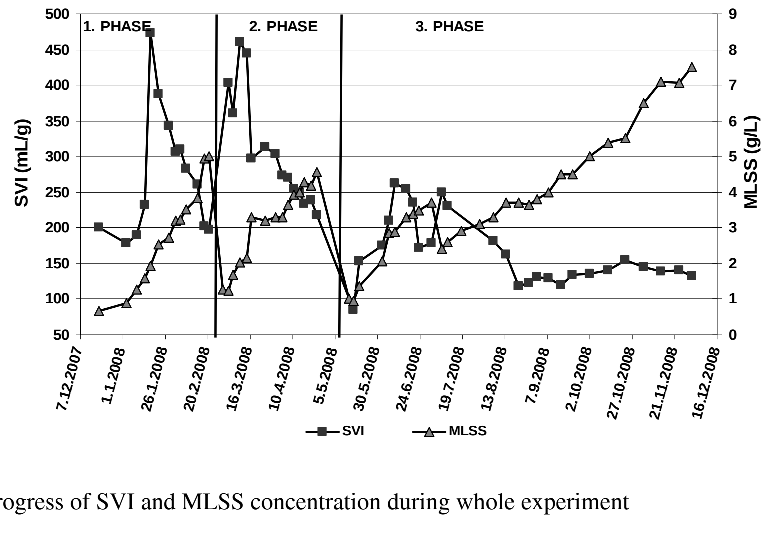

Fig. 3. Progress of SVI and MLSS concentration during whole experiment

The dominant filamentous bacteria were *Microthrix Parvicella* - the amount was 5 from 6 according to Jenkins 1993 [5]. In this situation of massive sludge bulking, the household MBR plant offered advantage over the conventional WWTP by preventing failure of biological system due to biomass loss. The membrane is a physical barrier and this implies that all suspended solids had been retained in the system. In the third period was inoculum from another WWTP and it did not contain filamentous bacteria in such high amount (SVI less than 100 mL g^{-1}). Even though in the third period after 1.5 month SVI increased above 250 mL g^{-1} (max. 350 mL g^{-1}), but subsequently gradually decreased to 200 mL g^{-1} and the separation of clean water would be achievable by settling. From the results it can be seen that in household WWTPs sludge bulking may be a real problem (and if inoculum with filamentous bacteria is used, this problem is much more accentuated). In conventional activated sludge system (without membrane filtration) such bulking sludge will leak out in the outflow.

The activity of the sludge was measured by respiration rates. Despite of specific conditions in household WWTP, the activated sludge had a standard activity (respiration rates were typical for low loaded activated sludge process [6]). The total respiration rate $r_{ox,tot}$ was 33 mg O_2 $g^{-1}h^{-1}$, the average endogenous respiration rate $r_{ox,end}$ was 7.3 mg O_2 $g^{-1}h^{-1}$.

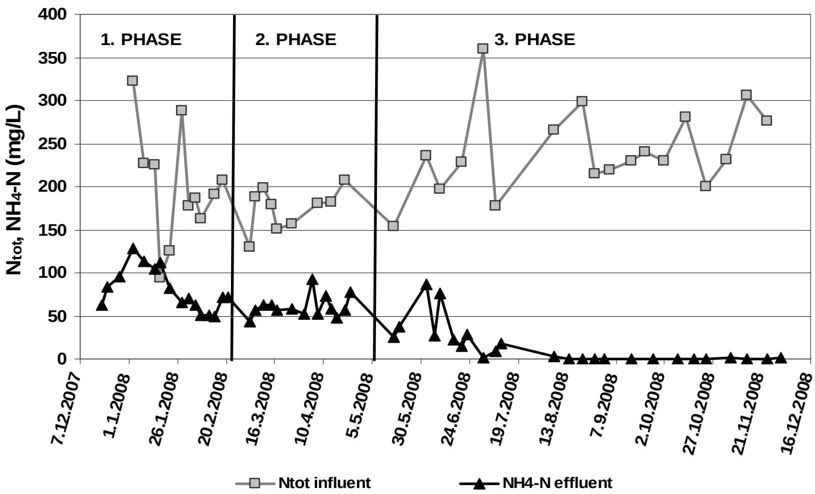

Fig. 4. N_{tot} influent and NH_4-$N_{effluent}$ concentrations

It was well known that nitrification could easily proceed in MBR because of complete rejection of nitrifier with membrane. However, the nitrifier still needed appropriate conditions, such as temperature, DO, pH etc., to live normally [7]. And concentrated wastewater from household WWTP could be a problem.

High concentration of N_{tot} in influent (usually above 150 mg L^{-1}, Fig. 4) incurred higher pH (during 1st and 2nd periods normally above 9) thus in activation tank was substrate inhibition achieved (inhibition with undissociated NH_3) [8]. During the low liquid temperature (during 1st and 2nd periods mainly less than 11°C; winter weeks even less than 7°C), it was logical that nitrification was not complete. In the household WWTP the nitrification started only when the temperature was 8-9 °C, despite of sufficient sludge age, high sludge concentration and high concentration of dissolved oxygen (constantly over 5-6 mg O_2 L^{-1}). NH_4-N concentrations less than 50 mg L^{-1}

were achieved when the temperature was above 20 °C. In a household WWTP may be the request of complete nitrification problematic.

3.3 Flux and Membrane Fouling

A special attention was fixed on a flux. The filtration in 1st period started without regulation of flux or transmembrane pressure. The initial flux was 45 L m^{-2} h^{-1} and we did not affect the system. We respected the probable situation that majority of owners and users of household WWTP would not be wastewater treatment experts, and they would not pay attention to the flux or pressure regulation. The flux decreased from the value 45 L m^{-2} h^{-1} below 10 L m^{-2} h^{-1} after ca. 3 months (it corresponds to 22 m^3 or 3.2 m^3/m^2 of filtered wastewater through the membrane). We changed the membrane module to new one and started the 2nd period. For the possibility of the flux regulation we installed a throttle at the effluent conduit. At the start-up we operated membrane module under the flux 13 L m^{-2} h^{-1} for 3 days, then the flux was set at 20 L m^{-2} h^{-1}. After three months the flux rapidly decreased to 6 L m^{-2} h^{-1} again. The membrane was regenerated by 0.5 % solution of acetic acid before the start of 3rd period. The membrane module was operated at the low flux below 10 L m^{-2} h^{-1} in this period. This value of flux appeared to be steady.

To membrane fouling probably contributed more factors:

- low temperature and sludge bulking - the overgrowth of filamentous bacteria could result in much more release of extracellular polymeric substances (EPS), and did great harm to membrane permeation [9,10],

- high concentration NH_4-N in concentrated domestic wastewater caused higher pH and precipitation of phosphates PO_4-P. Incipient precipitation may foul membrane. In winter is this problem more striking, because in activated tank the nitrification does not work and so pH does not decrease,

- high flux, mainly in first days after start up (membrane producers recommend flux below 15 L m^{-2} h^{-1}).

The significant impact of temperature on MBR fouling suggests that winter is the critical time for membrane operation. To control the possible intensification of membrane fouling under winter conditions, it is suggested to run the MBR at lower filtration flux, if possible, and to intensify the coarse bubble aeration [11].

4 Conclusions

The small household pilot-scale MBR plant ran continuously for nine months, in order to investigate the overall process performance and more specially the sludge and membrane behaviour to specific conditions like extreme temperature, high pH, long time zero load, etc. From this experiment the following conclusions could be drawn:

1. Average influent: organic pollution COD=917.6 mg L^{-1}, BOD=593.8 mg L^{-1}, P_{total}=8,6 mg L^{-1} , N_{total}=213.8 mg L^{-1}; NH_4-N=151.8 mg L^{-1}.

2. Effluent quality COD=53.4 mg L^{-1}, BOD_5=2.3 mg L^{-1} fulfilled legislative demands for household WWTPs without problems. Despite of sufficient sludge age request of complete nitrification in household WWTP may be a problem, mainly because of low temperature and substrate inhibition of nitrification.
3. High pH and low temperature may markedly slow down anaerobic stabilization of primary sludge in settle tanks.
4. Sludge separation by settling in a clarifier would be impossible because of massive sludge bulking, only installed membrane module guaranteed the perfect effluent quality.
5. Low temperature, sludge bulking (supported the overgrowth of filamentous bacteria, which have great impacts on the performance of MBR system because it led to more release of EPS), high pH (coupled with precipitation PO_4-P), request for minimum service work - difficult flux regulation in conditions of household WWTPs (particularly during start up the initial flux should be less than 15 L $m^{-2}h^{-1}$), a suitable and certain permeate pump (with references!) are the factors which may endanger function of membrane process in conditions of household WWTP.

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A23**

Application of Hydroinformatic Tools in Water Supply Management

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Abstract. Occurrence of water in time and area is not equal, and demand is not equal as well. These aspects create problems with water supply - insufficient water supply of population with potable water. Areas with insufficient water resources, insufficient quality or quantity water are necessary support from oversupply areas. Water supply networks are building for solution of this problem. These networks distribute water even in small cities or in big areas, which connect together big water resources and more demands. Management of water supply networks answers questions necessary for well and effective operation of water supply system as good understanding of system, define weak and strong point, determine potential of system for the future, possibilities of expand and increase of efficiency. These and other problems must solve operator or manager of water supply network. Article refers to possibilities of solving these problems with modern hydroinformatics tools support.

Keywords: Water supply management, hydroinformatic, mathematical modeling, GIS – Geographical Information Systems, SCADA

1 Introduction

Water supply networks are built for distributing water from oversupply water sources areas to deficit water sources areas, or for securing alternative water supply from more water resources for long distances. Not even quantity but quality of water source is important parameter. In Slovakia there are areas with sufficiency underground water resources, but quality of these water resources is not good for using them as water resources for potable water. Therefore, it is necessary to find solutions to supply these areas with water from water resources usable for water supply of inhabitants.

Building of water supply networks, which integrate more areas water supply networks into one well working water supply network can by a solution. Water supply network doesn't have to occur at the area of one water supply company, but can integrate more regional water networks into one over-regional water supply network.

Management of water supply network, or management of water supply company is not efficient without understanding of principle of whole system. It is important to know history of the system, evolution of the system, present condition of the system and of all objects on it, technical conditions and to formulate development plan for the future. It is complex process and all these components are important. Putting together theoretical knowledge's with practical information's from years of operation can bring positive effect for whole water supply system.

In past all the information were stored only in paper form. Computer technique allowed storing this information in electronic form. Though working stuff with a lot of years of experiences from operating water supply system and with a lot of information in their "heads", they have been determining segment in water supply company. All this knowledge they acquire through years, building the system, reconstructions, failures etc. In water supply companies at Slovakia began, slowly but surely, generational exchange. Therefore it is very important to collect and store all available information. On these purpose is convenient to use geographic-information system (GIS), which provide information not only about topology of network, but also information by over single objects on the network (pipes - diameter, material, roughness, construction age..., water tanks - volume, shape, elevation, age..., valve - type of valve, minor losses, material, manufacturer...). Better understanding of operation existing system is through knowledge network topology, objects on it, their condition and hydraulic principles of the network.

Parallel with the data gathering, problem of hydraulic analysis of water supply system is given higher importance. Especially in these days, days of strong economical growth of society connected with increasing of building industry, increasing the living standard of inhabitants and therefore increased requirement of potable water demand. On the other side request of rational utilization of water sources, which is strongly re-bound with apprehensions from decreasing capacity of water sources due to climate changes.

Construction of hydraulic model, calibration based on field measurement campaign or information from dispatching, can us allow understanding water supply system behavior during normal operation conditions and allows us to prepare special studies for simulating nonstandard operational conditions (failures, future development plans, reconstructions...).

2 Water Supply Management

Management is organizational process containing a strategic planning, goals determination, resources management, and organization of human resources, technical and financial sources which are necessary for achievement goals and for measurement of results. Management also contains recording and storage of data for later evaluation.

Management of water supply system is complex process and involves wide problematic of distribution water for long distances. Management of water supply system includes also management of water resources, management of water distribution and management of demand.

Management of water supply system must solve new task and problems like:

- securing of suitable hydraulic conditions for water distribution and accumulation;
- securing of water quality during distribution;
- establishing economical rate or sustainable costs of building water supply system;
- securing of water supply safety.

Shortfall of these aims requires application of integrated and multidisciplinary approach linking technical aspects (hydrological, biophysical, chemical, physical) and non-technical aspects (economic, institutional, legal and planning). For more effective management of water supply system is convenient to use modern hydroinformatics tools such as dispatching, GIS and mathematical modeling.

3 Hydroinformatic

Hydroinformatic as a science discipline has reinvented at the end of the past century. Development of hydroinformatics began with effort of integration single science disciplines in water management - from hydrology over hydraulic to informatics. Intense development of computer technique and referring areas, measurement technique, methodology of collection and data processing, GIS at cetera, results into creating tools for simplification and streamline operation of water network infrastructure. Hydroinformatic through active and passive market affects not only economic, ecological but also social aspects of environment. Hydroinformatic systems therefore contribute toward various factors such as protection of environment, increase of directness and pressure on legislative and institutional relations, reversible and irreversible processes in the industrialization, for example influence of employment in the specialization. One from most widely used tools of hydroinformatic is mathematical model.

Wide spectral features of mathematical models give real prerequisite on application this tool for support of management and operational water supply system. Optimalization tasks of fluvial or pipe systems are made in Slovakia several years. Question of crisis management and application of modern tools by the creation early warnings systems is very important these days.

Determinative factor for using hydroinformatic systems is especially price and demand on quality data inputs. Future application depends on qualified personnel able to work with modern software products. Universities began implement into educational schemes subject about application of computing tools for requirement of preparation of qualified staff for requirement of practice.

Modern software tools are necessary for solution problems with operation of water supply system such as hydraulic, qualitative, and economic and safety problems.

Management is complex tool for administration and operation of water supply system. It contains modern software tools divided in of three groups:

- Mathematical models
- SCADA (Supervisory Control And Data Acquisition)
- GIS

These processes allowed us manage whole system more effective with information in real time. All three systems works with information data of conditions of water supply system. They are connected together, information's can provide to each other (Fig. 1).

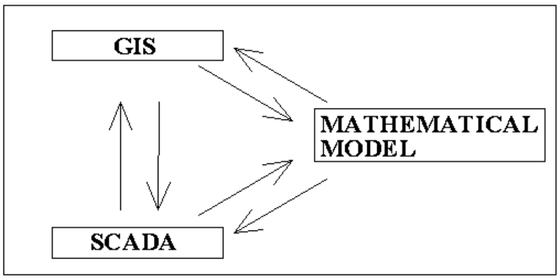

Fig. 1. Data connection of GIS, SCADA and mathematical model in water supply management

3.1 Water Supply Dispatching

For effectively control over chosen technological system such as water supply system it is advisable have sufficiency actual and historical data about system condition and to have possibilities of operative intervention in technologies. These allows operate and monitoring system SCADA (Supervisory Control And Data Acquisition) - water dispatching. Task of dispatching is automatically collect, transfer, archive and analyze information from monitored technological tools of objects, alarm by the creation deflect from normal conditions, register these events, compile messages and protocols.

Remote monitoring and control elements of water supply system means connection operator with the devices on short even long distances. Modern water dispatching is two-way control system. It means not only monitoring production and distribution of water, but also based on collected and processed data from monitoring to evaluate situation and to enter commands for controlling chosen system elements.

Water dispatching has four main features:

- Information function - PC in the dispatching enables to display incoming data from entire water system;
- Control function - remote control of chosen elements of water system;
- Automatization function - without intervention control process of production and distribution of water;
- Archive function - dispatcher centre archives all received data, complete reviews or trends for future usage.

Monitoring of water systems starts from operational requests of the system. It is transmission of necessary data from production objects and distribution of water as:

- water resources;
- water treatment plants;
- water tanks, accumulation tanks;
- pumping and automatic pressure stations;
- armatures on pipes installed in chambers.

Measured and transferred data on dispatching have various characters. First of all they are:

- quantitative data (flow, water head level, pressure);
- qualitatively data (concentration of disinfecting medium, temperature, pH);
- engineering-technology data (operation of pumps, valves, chlorinators etc.);
- safety data (disruption of objects, floating of space, overrun of allowable content of chlorine and others alarms);
- energetic inputs and outputs (presence of voltage, voltage loss);
- communications data (loss of communication);
- display and acoustic data (safety cameras, acoustic or light alarms).

3.2 Geographic Information Systems

GIS (Geographic information systems) is used since 1960. Operations which is using have been done manually also before 100 years, but single history of GIS started with development of computer techniques.

Geographic information system is computer based tool for mapping and analysis objects and events of real world. GIS belongs into group of information technology which connect current database operations as entering tasks and statistical computing with unique possibilities of displaying and spatial analyses which presents map. Technology of GIS can display information's and is able through the special programs execute different operations, classify them, evaluate and export these processed data into another information systems. Geographic information system is set of technical and programmatic tools for storing, processing and using geographic information's in two forms - graphic and database, mutually linked and topologically ordered.

From mentioned results that the GIS is a computer based tool orientated on processing of geographical information presented through map. Basic principle is on the figure 2. All information's have been processed by hand before beginning of "computer era". Resultant maps displayed specific status. If there is necessary to change some data, whole map must be changed. It is long process. From these reasons is for processing of geographic data necessary to use program tools, which allows manipulation with them and displaying of the results in digital or paper form.

According to this definition, GIS is complex tool for transformation and management of geographic information, situate on the face of the earth. Geographic information means spatial, topological, thematically and dynamical description of the object. One of the largest users of GIS is administration of engineering networks.

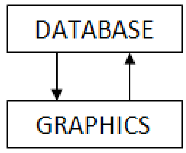

Fig. 2. Basic principle of GIS

An application of GIS technology has three general areas: creation and displaying of maps - GIS as a tool for administration with map underlayer; creation and processing of maps - GIS as a tool for collecting of data connected with geographical position; analytic function - GIS as a tool for spatial analyzes and synthesis of data.

3.3 Mathematical Modeling

Creation of hydroinformatic system, which can proved feasibly and objectively project of real situation on chosen element of water supply infrastructure has been unfeasible conception in the past. Analysis of flow in water pipes has been one of the first programmatic applications on tube computers in the 60 years. Progress in the software and hardware areas allows solutions of the systems consist from thousands of pipes in few second or minutes. Mathematical models are one of the most usefull toll of hydroinformatic. Programmatic solution of mathematical model of water supply systems allows analyzing hydraulic parameters and qualitative indicators.

Classic problem of flow determination in water pipes solves question: What is flow and pressure in the system by existing consumptions and inflows into system. Resolution of problems requires two equations. Second solves non-linear relation between flow and losses in the pipes, Hazen-Williams or Darcy-Weisbach equation. If system contains loops or more than one water source, solution consists of a number of non-linear equations. Resolution of these equation requires iteration, solve these iteration require support of modern computer technique. Most of water supply systems consist of loops, mathematical modeling is necessary for their analysis.

Most frequent usage of hydroinformatic tools in urban areas is exploitation of mathematical model for requirements of simulation and behavior of the engineering network. In purpose of modeling of water and sewage system in Slovakia are used several type of software's: Mike Urban, WaterCad, SewaCad, Sewdes, Epanet, Mike Net, Mouse and others. They are used for modeling of water supply systems and sewage systems.

Simulation of the water supply system by hydroinformatic tools is not exactly-there must by simplification. Therefore is necessary to make calibration and simplification of the system. Key factor of building model and for the best results is "engineering" knowledge of operator. Single model is not able to replace human resources and decision by management, but is very usefull as a strong decision making tool.

Usage of model can by divided into three basic groups:

- forecast (entrance - known, the system - known, result - unknown);
- identification (entrance - known, the system - unknown, result - known);

- detection (entrance - unknown, the system - known, result - known).

Basic review of all model possibilities for water supply system is given Fig. 3.

Mathematical models can help to improve and streamline distribution system. Other advantages are:

- systematic organization, editing and verifying of errors by necessary input data for model;
- support of displaying the results, such as color results maps (color pipes based on diameter, material, roughness...), displaying of time series of demands, etc.;
- connection with another software's such as databases, CADs, GIS;
- possibilities to make another analysis for the system, such as optimization of pipe diameters, optimization of water pumps, automatic calibration, modeling of water quality.

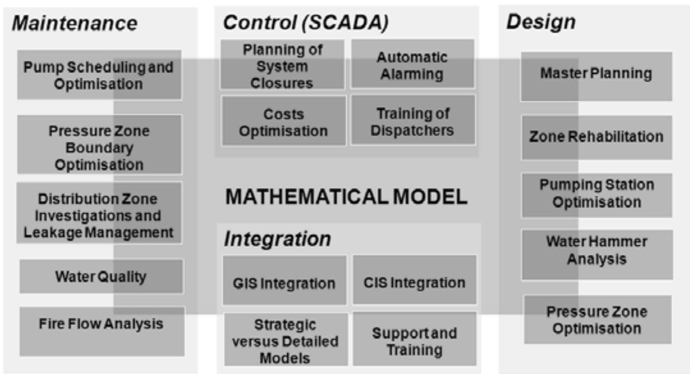

Fig. 3. Application of mathematical model for management of water supply system

4 Conclusions

Basic, but not only one function of construction of water supply system is transport of water from oversupply areas into deficit areas. Joining of water supply systems into big, above regional units brings number of advantages as well as problems with their operation. Main advantages is increasing of safety of water supply for inhabitants with potable water because of connection more water sources into one system and possibility of their exploitations in non-standard operational conditions. On the other side springs up new problems with mixing water, long time of water retention in system, capacity of water sources, accumulation objects, economic efficiency, preservation of safety of the system etc.

Management of water supply system brings answers for correct and effective operation of water supply. Correct understanding of the system, definition of weak and strong aspects, definition of potential of the system for the future, possibilities of expansion and efficiency. These and many others problems based on concrete water supply system must solve manager of water supply system. Tools of hydroinformatic are very usefull for solving these tasks. From the mentioned tools, the firs was water dispatching. In present the water companies began pay attention for retrieving, collecting and storing great amount of information about water supply system and objects on it. For managing such as great amount of information in tabular, database and graphic form are usefull geographic information systems. This process is time consuming and can take several years from single proposal of database system after collecting and verification all information's. It is never ending story and design of databases and update of all data are necessary for the future. Third tool of hydroinformatic is mathematical modeling. Creation of mathematical model is for functioning and effective management of water supply system very usefull. For creation and calibration of mathematical model are used information from GIS and from water dispatching. Advantages are in simulation of non-standard operational conditions (failure on water network, failure of pumping station...), simulation of future plans of water company (future development plan), making operation of water supply network more effective etc. Single model is very strong decision-making tool for correct management of water supply system.

Acknowledgement

The article has been prepared based on the support of Research Grant KEGA 3/5125/07 and VEGA 1/0854/08 dealt with at the Department of Sanitary and Environmental Engineering of the Faculty of Civil Engineering of the Slovak University of Technology in Bratislava.

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A40**

Wastewater from Small Urban Areas -Impact of Environment in Slovakia

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Abstract. After entrance of Slovak Republic to European Union has Slovakia to follow the requirements EU directives in the area of waste water disposal and treatment. These standards are implemented in Slovak Water Act No. 364/2004 Z.z.and in the Slovak directives and standards. Today is in Slovakia valid new European legislation, but to use it in real conditions is not so easy. It is dilemma to find compliance with measures, which is following the strict requirements of EU by discharging of waste water in receiving waters with lack of funds required for the construction of new sewage systems and WWTPs in small municipalities. This problem concerns especially municipalities with the equivalent population over 2,000 (EO), that are 2505 municipalities, where lives about 1.65 million inhabitants. The solution for these municipalities is decentralized waste water disposal.

Keywords: Sewage systems, waste water treatment, small urban areas, legislation in Slovakia

1 Introduction

Majority of Slovak municipalities belongs to the small municipality category and predominantly they are located in areas with less affected environment, the sewage solution needs to meet the technical and financial requirements, but also the aforementioned dilemma must be reduced to the acceptable degree and the sewage network must be integrated sensitively with the environment. Preferred approach is sewage network – Waste water treatment plant (WWTP), as well as sewage – natural environment – life environment. These links need to be given a priority not only in terms of design planning, execution and operation of the construction, but also in terms of contradiction waste water – surface water. Whereas the waste water, carried off via a sewage network represents progress for the society, mainly in terms of improved health and hygiene, the waste water is detrimental to the surface water and subsequently also to the natural environment.

2 Connection to Water Supply System and to Sewage System in Slovakia

There is a great disproportion between connection to water supply system and connection to sewage system in Slovakia. More than 86,16% of population is connected to public water supply system, but only 58,3% of Slovak population is connected to public sewage system. You can find some districts where the connection is lower than 30%. We have to solve this problem in the near future. There is coming great amount of finances for building a new sewage system in Slovakia. But at the beginning we have to find the general idea how to built this system with the view of technical and economical point of view.

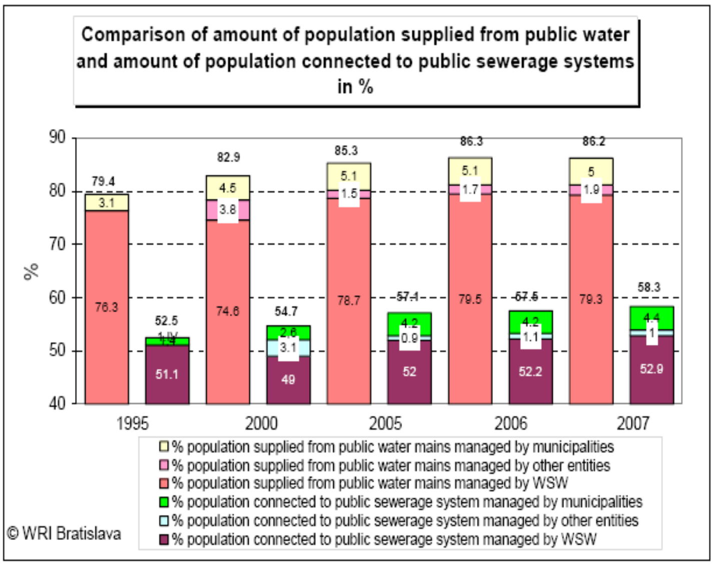

Fig. 1. Comparison of amount of population supplied from public water supply systems and amount of population connected to public sewage systems.

3 Waste Water Collection and Treatment

Development of public sewerage system falls behind the development of public water supply network in the Slovak Republic. In 2007 the number of residents connected to public sewerage system increased by 39.1 thousand to the total number of 3,147.0 thousand inhabitants. Unfavourable situation is in particular regions and districts. Trnava, Žilina and Nitra regions are below the nationwide average value. At the district level the worst situation is in the districts of Komárno, Námestovo, Čadca and Košice-environs where the proportion of inhabitants living in houses connected to public sewerage system is lower than 30%.

Development of public sewerage system and volume of discharged wastewater through public sewerage system administrated by state companies of water and sewage works is listed in Fig. 1.

Existing trend leads to significant differences in development of water supply systems and sewerage system that have an effect on environment and in connection with the requirements of the EU directives they increase investments for their implementation into practice in the Slovak Republic.

4 Sewage, Division and Execution Status

Sewage is a set of equipment allowing harmless removal of waste water, including its treatment. It consists of two sub-systems – sewage network with construction objects and waste water treatment plant.

There are a number of municipalities, which have addressed waste water treatment partially, or not at all. In terms of the world standard in this area the municipalities can be split into the following categories:

- ideal status, municipality with sewage network with waste water treatment, located before the receiving water;
- interim status, sewage system prior to expansion, reconstruction;
- interim status, sewage network in place, insufficient effectiveness WWTP;
- unsatisfactory status, sewage system without WWTP in place;
- critical status, no sewage system.

5 Sewage, Division and Execution Status

Construction of sewage systems in small housing centres requires the preparation of warranted, prudent and forward looking concept. Formulation of such concept needs to be entrusted with experienced specialists with know-how on the subject. Considering the required distances, sewage cost per person in smaller estates are higher than average. Therefore a great care is necessary when deciding, which technical solution to apply. Two possible solutions for the treatment of household effluent are available:

- Central drainage and treatment of waste water
- Individual waste water treatment

6 Central Drainage and Treatment of Waste Water

This is a case of a construction of new municipal sewage network and its connection to the existing regional waste water treatment facility or a construction of a new local WWTP. From the technical and operational aspect, it is the optimal solution. Currently a Water Act has come into force in Slovakia, stating, that by the year 2015 every municipality with population over 2000 will have to be connected to the public sewage system and waste water from this sewage must be subsequently treated in WWTP with biological treatment level effectiveness, which will be determined by the respective water management body according to the pollution degree of the receiving waters, which will be receiving the treated water. At the same time however it is necessary to consider the question of financial effectiveness of the given solution. The sewage system construction and its financing will be the responsibility of the municipalities, who find it increasingly difficult to raise the substantial amounts of funds required for such purposes. For example a construction of 1 m of gravity-fed sewage system, outside the road, costs 100–300 Euro, depending on the contractor. Considering the adverse financial position of our municipalities we are encountering more and more often individual solutions for household waste water storage and treatment.

7 Individual Waste Water Treatment

Household effluent can be drained from individual houses into drain-wells, septic tanks or into individual waste water treatment facilities.

7.1 Drain-Wells

Drain-wells are used as storage tanks for household effluent. In majority of cases they are built as enclosed monolithic concrete tanks in the vicinity of the house. Disadvantage of the drain-wells is that they are used only for storage purposes and not as a separation or stabilisation tanks and therefore the content has to be removed and transported to the WWTP.

7.2 Septic Tanks

These are flow-through tanks used for accumulation, sedimentation and partially stabilisation purposes. They were known and used already towards the end of the last century, when waste water from cities across England was treated in this manner. They work as a small anaerobic filter. The sludge is separated from the water, which is then filtered through a filtration layer. Thus reducing substantially the amount of sludge, this needs to be removed from the septic tank. However this sludge is not sufficiently stabilized and therefore it needs to be further processed. In principle there are three solutions available: removal, stabilization and final treatment at the municipal WWTP.

Removal– the sludge is taken to agricultural and other lands, sludge lagoons, possibly can be used for composting purposes.

Sludge stabilization – carried out in the form of wet oxidization or aerobic – thermal processing.

Final treatment in municipal WWTP – currently this is the only realistic method in case of high volume of sludge. However, WWTP must be suitable for sludge processing, since the sludge causes uneven peak loads for the treatment plant and therefore causing the run-off quality deterioration. It is accompanied by a strong odor and at the same time it has detrimental effects on the facility's equipment. When draining the sludge from septic tanks to WWTP, compliance with the following principles is recommended:

- maximal distance, effective for sludge transportation to WWTP is 20 – 25 km,
- minimal size of WWTP, for sludge processing is 10,000 EP.

Sludge drainage represents a technical problem. If the sludge is drained directly, it can disrupt the treatment plant operation and reduce the quality of treated water. Current trend to combat this problem is by building as storage tank for sludge. The size of the tank depends on the number of EP, which the WWTP is capable of handling. From this tank the sludge is evenly fed, even prior to the mechanical treatment. The technology in the processing of sludge prior to treating in WWTP has been addressed also by several domestic companies. The approach adopted abroad was to bring the sludge directly to heated putrefaction tanks. However, the practice has shown that this sludge does not have sufficient sedimentation properties. Therefore it is simpler to introduce the sludge into the waste water feeder at the WWTP. Of course, construction of such tank requires additional funding required for reconstruction of the treatment plant, which needs to be secured by the operator, i.e., the municipality.

7.3 Household Waste Water Treatment Plants

As a last of the offered solutions are household waste water treatment plants. Over the past several years we have been witnessing their construction with increasing frequency also in Slovakia. In price terms they are comparable with quality septic tanks, without having to deal with the problem of residual sludge. This sludge is aerobically stabilised, which means that it is hygienically harmless. It can be used in agriculture, thickened in concentration tanks or drained in sludge presses. Its thickening or drainage properties are comparable with properties of excess sludge produced in municipal WWPT.

Function of household waste water treatment plants. Treatment plants are designed for treating normal household effluent. They are scaled to accommodate approximately 5-50 population equivalents. Waste water treatment takes place in two steps. In the first step, during the mechanical pre-treatment, mechanical debris is removed from the water. Second step represents biological treatment in the form of fine-bubble aeration activation. At the same time the treatment process is extended by removal of biological elements of nitrogen and phosphorus in the form of denitrification and nitrification, which makes the majority of household treatment plants compliant with the European requirements with respect to effluent treatment. Waste water treatment plant itself comprises of the delivery unit placed on the concrete plate. It is necessary to ensure that the whole unit is watertight, since often it is placed below the water-table level and also to prevent the waste water seepage.

Technological treatment line design. Household waste water treatment plants are offered on our market by several companies in various technological modifications. They use either bio-filtration or a long-term activation with aerobic sludge activation. In our paper we will focus on the description of household WWTP technology and operation, which was used also by residents of a new housing estate in Bernolákovo. Since it is a new development within the boundaries of municipality, which does not have a public sewage system, it was necessary to conduct a study of effluent management for the area. Following the assessment of investments required for the construction of effluent sewage and related connection to the municipal WWTP, a decision on behalf of about 25 households was made, to construct individual household waste water treatment units to address the household effluent issue. Waste water treatment takes place in a circular tank, in the form of long-term activation with aerobic sludge activation. The principle of comprehensive waste water treatment in the proposed technological solution is based on biological treatment by heterogeneous biological sludge, maintained in the deposit, with prior denitrification, where the source of carbon for denitrification processes is the introduced organic contamination of waste water. In order to oxidise the biological treatment process and to maintain the concentrate in the deposit, an aerating system of fine-bubble aeration is applied. Air is delivered through fan powered by electric motor. Treated waste water is lead to the collection tank, where tertiary treatment, using disinfectant agent is introduced. Excess, aerobically stabilised sludge, is removed from the treatment process by effluent truck once or twice per year, depending on the sludge production.

Operation of household WWTP. It is very important to know that well functioning WWTPs, not requiring regular maintenance and audit do not exist. Therefore it is necessary to look after your WWTP and to follow the supplier instructions for maintenance and operation. Biological treatment is based on the biomass growth, which needs for its existence regular supply of nourishment in the form of organic contamination in waste water and also sufficient amount of oxygen, which is supplied to the system through fine-bubble aeration. Any disruption of this system can lead to deterioration of treatment effectiveness. Long-term incorrect operation of WWTP results in dying of the biomass, followed by total disruption of the treatment process. Although the treatment plant operation is automated, it still requires supervision. At least once a week the fan needs to be checked and the treatment process in the reactor, as well as the quality of treated water in the accumulation tank, need to be checked visually.

Household waste water treatment unit is designed for treating normal household effluent, therefore it can be disabled by the introduction of excess amounts of substances, which should not be present in municipal waste water. These are mainly the following:

- greases in higher concentration (frying oil);
- household softener solutions;

- paints, varnishes and solvents;
- powerful disinfectants and acids;
- low degradability materials (plastics, rubber, textiles).

Reliable treatment requires daily supply of effluent, in order to facilitate biological processes in the treatment unit. In case of absence over a period of 2 to 4 weeks, without new effluent for the WWTP, the micro-organisms start slowly to die. However, following the re-introduction of regular effluent supply they have the capacity to adapt and recommence their reproduction. However the air supply must be maintained also during the absence of waste water supply, otherwise organisms could start to decompose and rot. Only in case of absence from home for several months it is recommended to shut down the whole unit and remove the content.

Handling of treated water. Treated water can be used for watering of lawns and fruit trees. However it is not to be used for watering of plants for direct consumption, since it can contain substances harmful to human digestive tract.

Handling of excess sludge. When the separation tank is filled with sludge, it needs to be removed in order to prevent the sludge entering the treated water, hence reducing the quality at the exit point from WWTP. Excess sludge is sufficiently stabilised, i.e., hygienically harmful and it can be used in agriculture or used for further treatment in the municipal WWTP.

In closing, we would like to present several price comparisons, which can assist the consumers in selecting the most suitable solution for the treatment of their household effluent. These are for reference purposes only, since there is a number of suppliers and operators of sewage systems and waste water treatment plants.

8 Conclusions

Currently the trend is the preference for individual solution and each home owner has the freedom to select the most suitable equipment. Be it a drain-well, which is more affordable, but the operation requires regular emptying the full content, hence increased cost of removal or more costly septic tank, where only thickened sludge is being removed. However it is not stabilised and therefore a hygienic treatment at the municipal WWTP is necessary. Available is also a third solution, being highly promoted in the past few years – household waste water treatment unit. Although it is more demanding in terms of operation, there are however no problems with excess sludge and treated water.

Acknoledgement

The Research Grant KEGA 3/7452/09 and VEGA 1/0854/08 held by the Department of Sanitary Engineering Faculty of Civil Engineering, Slovak University of Technology Bratislava has supported this paper.

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **41**

Analysis of Stormwater Management and its Influence on Receiving Water: Case Study-Banska Bystrica

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Abstract. This paper analyses the influence of the combined sewer overflows (CSO's) on the recipient. A case study in town Banská Bystrica on river Hron (Slovak Republic) has presented four feasible alternatives of storm sewer management (different mixing ratio, different size of storm tanks).

Keywords: Water quality, water quality modeling, combined sewer overflows, receiving water, storm water management

1 Introduction

When evaluating the influence of sewer water on the receiving water we can follow emission and immission principles. The emission principle is based on the determination of volume limits, or the concentrations limits of harmful substances in waste waters, not taking into consideration the effects, which the waste water will have on the recipient when they mix. The immission principle on the other hand, requires the fulfilment of certain criteria in the recipient, without taking into consideration the harmful effect of the waste water. In order to fulfil the criteria to the degree most possible, a combination of the two principles is used, known as the emission-immission principle.

The emission and immission evaluating principle for evaluating the influence of sewage water on the recipient, has been known for a long time and is worked out in detail in literature and also in the legislation, where it is successfully used for evaluating the influence of the continuous outflow of waste waters – from the communal waste, industrial sewage plants etc. What are the effects of discontinued sources of pollution, for example of the CSO chambers on individual sewage networks?

2 Methods Used for Evaluating the Effect of Waste Water on the Recipient

Nowadays only the emission method is used to check and to approve the discharge of the waste waters from combined sewer overflows (CSO) in the legislation of Slovak republic, which is based on a so called mixing ratio. For sewage networks with large catchment, which have a more than ten CSO structures, there is a limitation for CSO events per year (Gov. Order nr. 296 / 2005 Coll.). Other methods are also known from abroad, which are based on regulating of the amount of overflow water (or the amount of pollution) taking into consideration the size of the catchment (for example the amount of waste water from CSO in m^3 per ha of the dewatered area per annum, kg BOD_5 per 1 ha per annum etc.)

The immission method for evaluating the influence of sewage water from CSO on the receiving waters is known from foreign literature and legislations. One of the most complex manuals dealing with this subject is the British UPM (UPM, 1998), which requires complex monitoring of the water quality in the receiving waters, and also requires model studies for certain cases.

The basic approach towards solving the effect which waste water has on the receiving water is based on the achieving of the required quality of the receiving water and preventing of discharging out the waste water in large amounts, according to what the receiving water is used for (protecting the fish, water and bathing water). It is necessary to follow water quality criteria for surface waters, in order to ensure a minimal concentration of dissolved oxygen in the receiving water, in other words the maximal concentration ammonia (NH_3-N) in reference to the pH and temperature of the water in the recipient, with the values LC_{50} (50% death rate of fish with the stated concentration and exposition of the pollution concentration).

This method for evaluating the influence of sewage water from CSO structures on the recipient requires the use of the most up to date technology when devising and evaluating sewage networks, in other words evaluating the effects on the recipient using the runoff models (which can include the quality module), then the surface water quality models and the last stage requires the simulation of processes in sewage plants.

This kind of a complex approach is not feasible due to the amount of input information need to evaluate the influence, taking into consideration the conditions in Slovak republic (but I would think also conditions in other countries). The needed information either does not exist, or is not available in practice. Another problem is the price of the needed models, inadequate or no experience with modelling, problems with the calibration and verification of the models (the verification of the most basic model requires a large amount of field measuring).

It is apparent that proving the fulfilment or the inability to fulfil the required limits is possible only by direct continual measurements in the field or mathematical models.

Despite all the stated difficulties and problems we think, that the immission method for evaluating the influence of sewage water outflows from CSO is possible, the question of how we can apply it in the conditions, that the mathematical models are available, but we do not have the data needed.

3 Used Methods–Mathematical Model for the Evaluation of the Influence of Waste Water from CSO

In order to solve these questions we have decided to carry out a case study for the city Banská Bystrica, in the framework of the grant task given by the Department of Sanitary and Environmental Engineering SvF STU Bratislava, which deals with the analysis of the influence of outflow of sewage water form CSO on the recipient – the river Hron.

The first part of the case study was to create a mathematical model of the runoff of the sewage network of the city Banská Bystrica and a mathematical model for the simulation of the quality and CSO overflows of the river Hron. For the mathematical simulation of the water quality in the river Hron we used the model MIKE 11 (Danish Hydraulic Institute, DHI). In the study we used data from a former research projects, worked out at Dpt. of Sanitary and Environmental Engineering (partial study task 06 "Point pollution sources and managing the water quality of the rivers Váh and Hron" VTP 27-34, research leader was the Water Research Institute in Bratislava)

From a space point of view we modelled the section of Hron from the river kilometre 261.300 (Valkovňa profile) up to the estuary (km 0.000), which is practically the whole river except for a short section near the spring.

We must however state that when modelling the section of the river, we did not have an adequate amount of information concerning the hydraulics of the river (roughness coefficients), or the kinetics of the physical, chemical and biological processes in the river (dispersion coefficients, re-aeration, deoxygenation coefficients etc.)

We consider the lack of adequate information concerning the tributaries of the river Hron (the tributaries Čierny Hron, Bystrica, Slatina and Sikenica are the only ones regularly monitored) to be the biggest problem. However this is a problem, which is far more complex and concerns all the Slovak rivers.

Two alternative simulations were carried out in order to evaluate the influence of the point sources of pollution on the modelled section of Hron – with long-term average flow (Q_a – for the river Hron, and also in all the tributaries) and for flows, which are close to the minimal flows –for a 355 day flow- Q_{355}. In the first of the alternatives (flow Q_a) we used average pollution concentrations to simulate the water quality; in the second alternative (Q_{355}) we used for the simulation the value c_{90} (90% distribution percentile of the most unfavourable value of concentration). As input values we used information gained from the annual water quality report [SHMÚ 1999, SHMÚ 2000].

For the simulation of the water quality the following processes were simulated:

1. Re-aeration
2. Degradation of organic substances –oxygen depletion
3. Nitrification
4. Denitrification

The second part of the project was modelling the outflow of discharged water from the CSO structures in the city of Banská Bystrica. The model DHI – MOUSE was used for the simulation. The primary intention of the whole model study was to use information on rain events for the time period of 28 years from the station Sliač, but the time needed for such a long-term simulation did not suit our conditions. Then we decided for an alternative solution- using block rainfalls with the periodicity of 0.033, 0.1, 0.2, 0.5, 1.0 a 5.0 with a duration of 15, 30, 60 and 180 minutes. We simulated 4 alternatives of the storm water management (according the SK legislation):

1. Dilution ratio (mixing) with the ratio of 1:4 (minimum required according the SK legislation)
2. Dilution ratio (mixing) with the ratio of 1:8 (maximum required according the SK legislation)
3. Storm tank with the volume for accumulation of discharged storm volume with the periodicity p=0,5
4. Storm tank with the volume for accumulation of discharged storm volume with the periodicity p=5

According to the SK legislation it is required that the outflow of mixed waste waters, running into the sewage plant undergo at least the mechanical treatment.

We took into account the quality of waste waters discharged from CSO structures according to a study, elaborated by the KZI Bratislava in cooperation with VÚVH in the years 1996-1999. (Sztruhár et al.2001). When the CSO was placed in the WWTP we took into account that some of the pollution will be removed due to the mechanical treatment.

The modelled flows and the quality of the waste waters discharged from CSO's were used as input to the model of the water quality in the river Hron. In advance we evaluated only the alternative, where the discharged waste waters will be discharged into the average river flow (Q_a), but we deal also with the alternative that the discharged waste waters will be let out and mix with the 355 day flow (Q_{355}).

4 Results and Discussion

Simulation results of the water quality shown by the IDF (Intensity – Duration - Frequency) lines of concentration of the dissolved oxygen concentration, also for BOD and $N\text{-}NH_4$, for various judged alternatives are displayed in Fig. 1-3. These lines show the intensity of the negative effects and the duration of this exposition on the biocoenosis of the river.

As shown in the Figs. 1-3, the discharging of sewage water represents a relatively short-term stress on the recipient. It is necessary to mention that the catchment of Banská Bystrica is relatively specific – it has a relatively big slopes and a large amount of CSO structures, which means that the large volumes of the sewage water gets to the recipient through the CSO structures very fast (very small retention capacity of the system, a total absence of accumulation structures in the system).

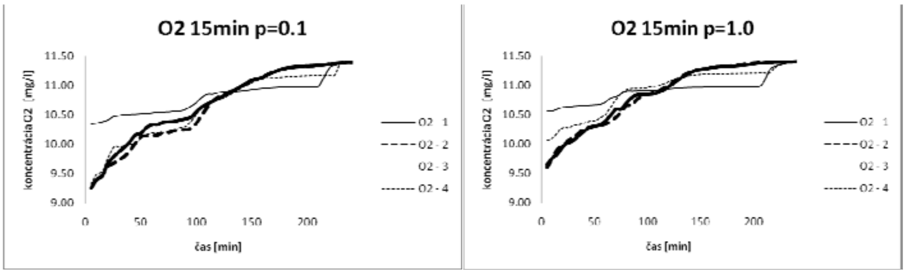

Fig. 1. IDF lines of the dissolved oxygen concentration (O_2) in the recipient during a 15 minute rain event with a periodicity of occurrence p=0,1 and 1,0 (the numbers in the legend represent the storm water management alternative)

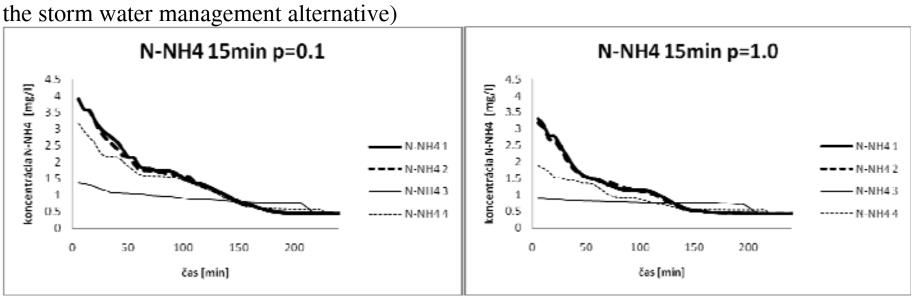

Fig. 2. IDF lines for the ammonium nitrogen (N- NH4) concentration in the recipient during a 15 minute rain event with a periodicity of occurrence p=0,1 and 1,0 (the numbers in the legend represent the storm water management alternative)

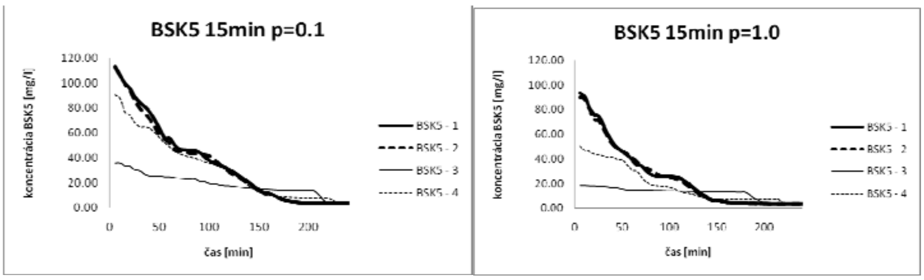

Fig. 3. IDF lines for the biochemical oxygen demand (BOD_5) concentration the in the recipient during a 15 minute rain event with a periodicity of occurrence p=0,1 and 1,0 (the numbers in the legend represent the storm water management alternative, BSK5 means BOD_5)

When comparing individual methods of storm water management (Figs. 1-3), the following can be stated:

1. The size of the mixing ratio for short-term intensive rain events practically doesn't have any effect on the quality of water in the recipient.
2. The most effective method of regulation is, in spite of many disagreements and problems, the building of storm tanks.

Ad 1: the judged alternatives represent minimum and maximum requirements according to the current Slovak legislation. As can be seen on Fig. 1-3, during short-term rain events with a relatively low periodicity of occurrence, there isn't a big difference between these two alternatives concerning the quality of water in the recipient. The difference is shown more during longer rain events or during heavy rains with higher periodicity. Also when using a higher mixing ratio, the periodicity of occurrence of the CSO events (discharges the sewage water into the recipient) decreases, which can in some cases have a significant importance.

Ad 2: this conclusion is not in any way surprising, these results were expected. Nevertheless they are shown here, because they easily show the effect of storm tanks on the quality of water in the recipient – the concentration of harmful substances in the recipient in the described cases is three to four times lower (alt. 3) than alternatives without storm tanks.

For information integrity, also the longitudinal profiles with contaminations concentrations in the river Hron (Figs. 4-5) are presented.

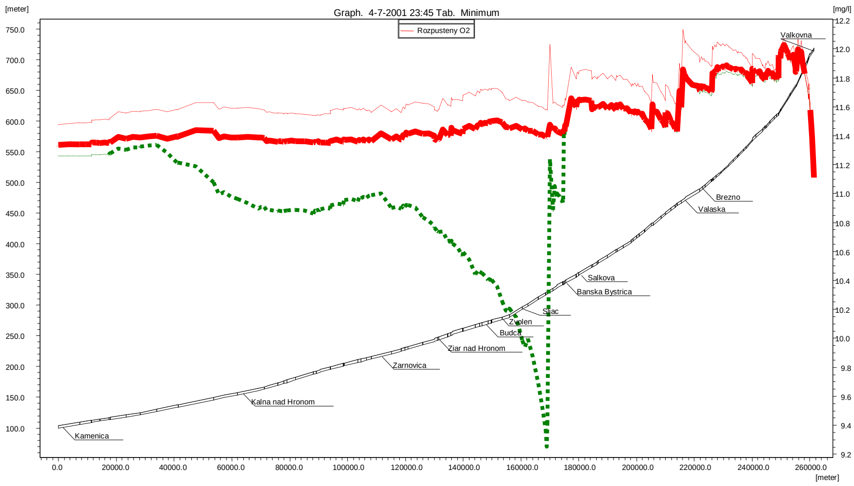

Fig. 4. Concentration of soluble oxygen (O_2) in the recipient (longitudinal profile) during a 15 minute rain with a periodicity of occurrence p=0.1 and 1.0, alternative No. 1 (Legend: full line – continuous state, dotted lines – minimum or maximum concentration)

When modelling the quality of water in the river, we have come to the conclusion that the pollution concentration in the river Hron isn't decreased only by the self-cleaning ability of the river, but also by the hydrology and the hydraulics of the stream, which is the dilution, mixing and dispersion. For example after the CSO event, due to the lateral inflows of Hron, discharge in river doubles within a few hours (from profile Sliač to profile Žarnovica), due to this fact the concentration decreases by half. This fact has a significant effect on the modelling of the concentration of dissolved oxygen – water with a large amount of oxygen flows in from the lateral inflows (mountain areas, big slope of inflows), which improves the oxygen balance of the Hron river after the CSO event. This situation is relatively specific, even though it is possible to predict it on a large number of rivers and streams in Slovakia.

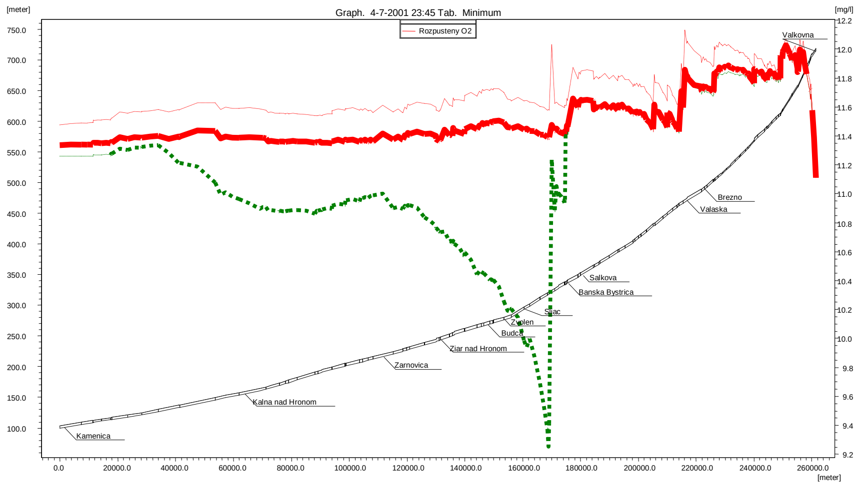

Fig. 5. Concentration of BOD_5 in the recipient (longitudinal profile) during a 15 minute rain with an occurrence periodicity p=0.1 and 1.0, alternative No. 1 (Legend: full line – continuous state, dotted line –maximum concentrations)

5 Conclusions

Thanks to the analogy from the continuous outflow of sewage water (where average values of pollution concentrations and the average discharge are used for the outflow from the WWTP and nearby the minimum value- Q_{355} of the discharge in the recipient), it is possible to determine the effect of discharging the raw sewage water from CSO structures on the recipient with similar approach.

Results of this study are the IDF curves for concentrations of selected quality parameters (BOD_5, dissolved oxygen, temperature, NH_4-N, NO_3-N, particular and dissolved P) in longitudinal profile along the whole river after a CSO event in the city of Banská Bystrica..

The basic problem, as stated above, is the lack of necessary input data for the mathematical model, due to which we then have to approach various simplifications and assumptions of conditions in the recipient, or on the urbanized catchment (and it is still not clear, whether such conditions will even occur).

The results of the study described in this paper, show the effect of storm tanks in the sewage system of the urbanized catchment. The found out result is, that when a storm tank is included into a sewer system, the pollution concentration in the recipient can be three to four times lower than in network schemes without storm tanks. Results of this modelling study also show that such modelling approach can be very useful for modelling of impacts on the water quality in receiving water and can be also very helpful in evaluation of various strategies and management practices focused on the receiving water quality improvement.

Acknowledgement

This paper was written with the support of the science grant agency VEGA in terms of solving the grant task no. 1/0854/08 „Optimalizácia návrhu a obnovy inžinierskych sietí v urbanizovanom území, ich alternatívne využitie“ (Optimization of the design and the rehabilitation of engineering networks in the urbanized areas, its alternative usage).

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A42**

Determination of Longitudinal Dispersion Coefficient in Sewer Networks

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Abstract. This paper presents a part of research project VEGA Nr. 2/0101/08 titled "Determination of dispersion phenomenon dependencies on principal hydraulic parameters of streams". There is a co-operative project between Institute of Hydrology Slovak Academy of Sciences and Faculty of Civil Engineering-Department of Sanitary and Environmental Engineering Slovak University of Technology in Bratislava. Described part of this project is focused on determination of dispersion coefficients in sewers, it means in prismatic stream channel with relatively constant roughness of streambed.

Keywords: Sewer network, open channel, dispersion, free-surface flow, longitudinal dispersion coefficient, field measurement, tracer experiment, contaminant transport, numerical simulation

1 Introduction

This paper deals with partial results of running project VEGA Nr. 2/0101/08 entitled: "Determination of dispersion phenomenon dependencies on principal hydraulic parameters of streams". Project solves this problem in collaboration of Institute of Hydrology, Slovak Academy of Sciences and Department of Sanitary and Environmental Engineering, Faculty of Civil Engineering, Slovak University of Technology in Bratislava.

The aim of project is to increase the reliability and ability to apply model tools for simulation of transport processes in complicated natural conditions and various hydrological situations. It will be achieved by completing and specifying of insufficient information about crucial input characteristics of transport processes, by formulation of dispersion coefficient relations that will be valid for basic channel shape. One of the main aims will be determination of these coefficients as a function of basic hydraulic parameters of stream.

2 Basic Theoretical Terms

Dispersion, from hydrodynamic point of view, is the spreading of mass from highly concentrated areas to less concentrated areas in flowing fluid. Mass in flowing water is not transported only in the reach of streamline, but it is also gradually spreading to outside in consequence of velocity pulsations and mass concentration differences. Mass dispersion with advection is basic motion mechanics of particles, transported in water. Reductions of maximum concentrations are results of their effects. The main characteristics of dispersion are dispersion coefficients in relevant directions. Determination of these dispersion characteristics is the key task for solving problem of pollutant transport in streams and for modelling of water quality.

The most simple description of the mass spreading in water is one-dimensional advection-dispersion equation, which takes the phenomenon in longitudinal direction x (well-proportioned distribution of a mass concentration is required along a depth and a width of a stream). The form of this equation is:

$$\underset{\mathrm{I}}{\frac{\partial AC}{\partial t}} + \underset{\mathrm{II}}{\frac{\partial QC}{\partial x}} - \underset{\mathrm{III}}{\frac{\partial}{\partial x}\left(AD\frac{\partial C}{\partial x}\right)} = \underset{\mathrm{IV}}{-AKC} + \underset{\mathrm{V}}{C_s.q} \tag{1}$$

where: C is a mass concentration (mg.l^{-1}); D is the longitudinal dispersion coefficient (m^2/s) - $D = d_f + \varepsilon$, where d_f is the coefficient of turbulent diffusion, ε is the coefficient of molecular dispersion, d_f is often disregarded, because $\varepsilon >> d_f$; A is a discharge area in a stream cross-section (m^2), Q is a discharge in a stream (m^3/s), K represents a rate of growth or decay of contaminant (s^{-1}), C_s is the concentration of a source, q is discharge of a source, x is a distance (m), t is time (s).

Part I in Eq.(1) expresses pollutant concentration change in time, part II represents pollutant transport through the velocity field, part III describes pollutant transport by diffusion and dispersion, part IV means chemical or biological non-conservation of pollutant and part V represents pollutant sources in the stream.

Equation (1) covers two basic transport mechanisms:

- advection (or convection) transport by fluid flow,
- dispersion transport by concentration gradient.

Such one-dimensional approach is applicable for rivers or streams with comparatively non–wide channel or e.g. for sewers. In this case the pollutant spreading has markedly one-dimensional character. However, this assumption is not acceptable for reservoirs, where spreading phenomenon has three-dimensional character, meaning that hydraulic characteristics and their values are varying also along a width and a depth of the discharge cross section.

The reliability of models is influenced by the fact that the numerical simulations mean always a simplification of the complicated natural conditions. Finally the rate of reliability is closely connected with the level of input availability and validity. The models simulating a pollution transport in open channels require a determination of the dispersion coefficients among other input data. These coefficients can be exactly obtained by way of field measurements, directly reflecting conditions in existing part of a stream. It is not always possible to obtain these coefficients in the field, because of financial or time reasons. Several foreign authors [1], [3], [4], [5], [6], [7], [8], [10], tried to get the empirical relations, especially for longitudinal dispersion coefficient. Their studies could be used to estimate an approximate value of the dispersion coefficient. Empirical relations have often limited validity, values given by them are useful only in specific conditions and in some cases there is not possible to use anyone. Therefore it is necessary to give attention to conditions in which the relations were obtained and if those conditions were approximately consistent with conditions in which the relations will be applied.

Problem of the contaminant spreading is not only in the case of the modelling of water quality in natural streams, but also in urbanity structures – in sewer networks. For that reason it is necessary to give attention to this fact. Therefore one aspect of the project is determination of longitudinal dispersion coefficient for prismatic channels, in this eventuality for sewers, from field experiments. And this paper just deals with this determination.

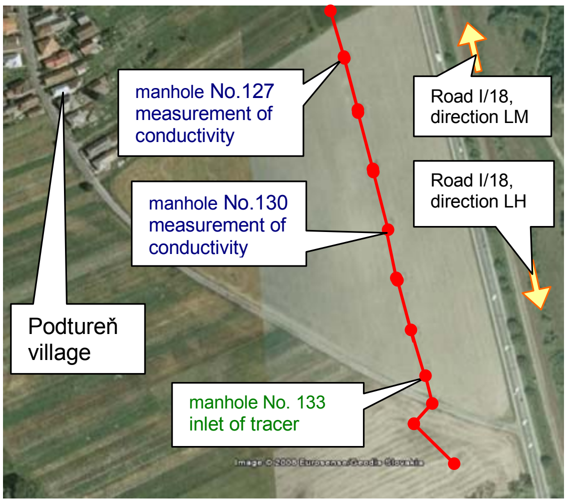

Fig. 1. Sewer collector – straight part (above Podtureň village)

The field measurement was done at the experimental hydrological base of Institute of Hydrology in Liptovský Mikuláš. The part of sewer network which was built in 2004-2005 under project ISPA „Development of environment in Liptov region" (more specifically the connecting collector between Liptovský Hrádok and Liptovský Mikuláš) was selected for field measurements. The collector has profile DN 500 or 600 mm and it lies in low slopes, which are near to minimal slopes. After more detailed reconnaissance there were selected two collection parts for measurements – the first one is above Podtureň village and the second part is closely over the autocamping Borová Sihoť, near Liptovský Hrádok. In the first part (Fig. 1) distributions of tracer concentration in time were measured at various distances in straight line.

In the second part there are some incurvings of sewer track (30°, 45° a 90° trajectory diversion) in which the distributions of tracer concentration in time were measured at various parts of sewer. The aim was to determine the influence of these trajectory diversions on size of dispersion coefficient (Fig. 2).

The common salt (NaCl) was used as a tracer and this one influenced on the variation of wastewater flow conductivity. The colour (fluorescein) was added to the tracer to monitor the course of tracer substance in mensural profile. The dosage of tracer was 5 l and it was discharged to sewer instantly.

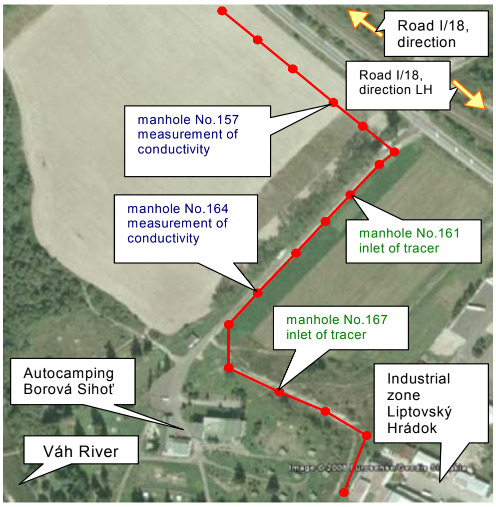

Fig. 2. Sewer collector – incurving of sewer track (near Autocamping Borová Sihoť)

The measurement of conductivity was performed with the movable conductivity meter in mensural manhole. The conductivity meter probe was situated in the centre of wastewater flow. The conductivity values were showing on display of conductivity meter in digital form. The stopwatch was located next to the conductivity meter. Behaviour of values was recording by camcorder and the record of measurement was manually digitised subsequently.

4 Results and Discussion

Output of measurements is the record of tracer concentration distribution in time by way of the distribution of wastewater conductivity in sewer. The example of graphic expression of a record is in Fig. 4.

The evaluation of experiment results consists in simulation of tracer experiment (concentration distribution) for various values of longitudinal dispersion coefficient. The base for this numerical simulation is the analytical solution of Eq. (1) for instantaneous injection of tracer [2]:

$$C(x,t) = \frac{G}{2A\sqrt{\pi D t}}.\exp\left[-\frac{(x-\bar{u}t)^2}{4Dt}\right] \tag{2}$$

The difference between the measured and simulated values is evaluated. The minimal difference determines the value of the longitudinal dispersion coefficient for each one of experiments. Although the probe was located in streamline it was ignored irregular distribution of concentration (conductivity) along the width of mensural profile. It was reason to add the correction coefficient to the model calculations, which comes out from ratio of inflow tracer volume and outflow tracer volume.
The preliminary results show the values of longitudinal dispersion coefficient in range 0,09-0,12 $m^2.s^{-1}$.

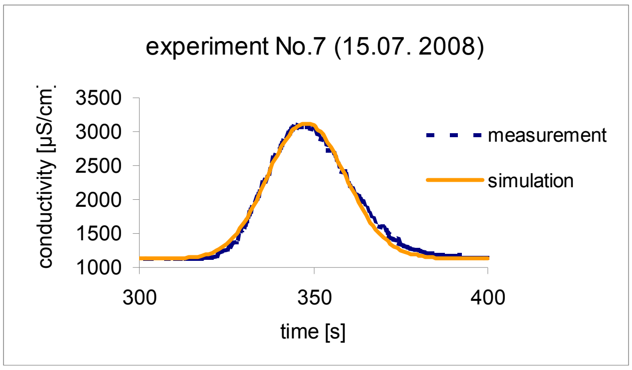

Fig. 3. Behaviour of conductivity in mensural profile (manhole No. 127), experiment No. 7, 15.7.2008 (inlet of tracer – manhole No. 133)

5 Conclusions

This contribution informs the specialist public about project VEGA č. 2/0101/08 “Determination of dispersion phenomenon dependencies on principal hydraulic parameters of streams“. There is a co-operative project between Institute of Hydrology Slovak Academy of Sciences and Faculty of Civil Engineering-Department of Sanitary and Environmental Engineering Slovak University of Technology in Bratislava.

The total aim of the project is to increase reliability and using of existing models for transport of water pollution and flowing matters. It has to be reached by specification of information about the crucial input characteristics of transport processes and by formulating of relationships for determination of dispersion coefficients. One of the main aims of project is determination of dispersion coefficients as a function of base stream hydraulic parameters (it is possible to use it also for sewers as a case of free-surface flow). So far the field measurements in the sewer network have been performed. The sewer network was selected as a model of prismatic channel. The tracer experiments were located to the straight part and to the part with modifications of sewer track direct. Until now there were realised field measurements with two various discharges. The analysis of measurement results and determination of dispersion coefficients for particular parts and measurements is going on at the present time. The longitudinal dispersion coefficient reaches the value 0,09-0,12 $m^2.s^{-1}$ in the straight path according existing results.

In the next time the effort of the research team will be to analyse the influence of hydraulic characteristics on dispersion coefficient values, to obtain dependency of longitudinal dispersion coefficient that will be valid for some shapes, which are found in the practice by using Fischer's formula, to verify obtained relation for longitudinal dispersion coefficient and to apply the obtained dependency in mathematical and numerical modelling of pollution spreading in channel (pollution accidents or pollution spreading from point sources).

Acknowledgement

This work was supported by the Science and Technology Assistance Agency (Slovakia) under the contract No. VEGA 2/0101/08.

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A43**

Iron and Manganese Removal from Small Water Resources

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Abstract. Water intended for drinking purposes is necessary to be treated in many cases to meet the requirements under the Regulation of the Government of the Slovak Republic No. 354/2006 on drinking water. There is a tendency to find technology with new more efficient and cost-effective materials and technologies. The goal of this study is to compare activated natural zeolite – clinoptilolite (rich deposits of clinoptilolite are in the East Slovakia Region - found in the eighties of the past century) with imported materials Birm and Greensand in removal of iron and manganese from water. Obtained results from experiments carried out in Rohatec (Czech Republic) prove that Klinopur-Mn is suitable for removal of iron and manganese from water and it is comparable with other imported materials.

Keywords: Water treatment, removal of iron, removal of manganese, filtration materials, Clinoptilolite, Birm, Greensand

1 Introduction

In Slovakia there are a number of groundwater resources. Larger resources are unevenly distributed throughout the territory and therefore the water intended for drinking purposes is supplied from smaller ones. In evaluation of water quality in small resources more than 300 resources with higher concentrations of iron, manganese, nitrates, ammonium ions, arsenic and antimony were identified. This issue was dealt with by a team of experts at Water Research Institute Bratislava who developed the criteria for water quality as well as other technical and economic aspects of using the water resource for supplying the population with drinking water since there are several regions with many municipalities without connection to public water supply located out of large distribution systems [1].

In case of groundwater used for drinking purposes is mostly needed water treatment – removal of iron and/or manganese. Concentrations of dissolved iron and manganese are evaluated every year within the groundwater monitoring done by the Slovak Hydrometeorological Institute for the whole territory of Slovakia. In searching for suitable water treatment technology the emphasis is placed on new more efficient and cost-effective methods and materials as compared to technology used by now [2].

One of the methods is removal of dissolved manganese by using the oxidized film on grains of filtration medium. A film is formed on the surface of filtration medium by adding potassium permanganate (not only $KMnO_4$ but also other strong oxidizing agents). This film serves as a catalyst of oxidation – grains of filtration medium are covered by higher metal oxides. In such a case, it is special filtration so-called "contact filtration" – filtration on manganese filters. Oxidation state of the film of MnO_x(s) medium has an important role in removal of dissolved manganese. Efficiency of manganese removal is a direct function of MnO_x(s) concentration and its oxidation state. On different filtration media the films with different ability to remove dissolved manganese from water are formed [3, 4, 5, 6, 7].

2 Study Objectives

Based on present knowledge concerning this natural material and works dealing with removal of dissolved manganese from water through oxidizing films on various filtration materials we have performed experiments with zeolite from Nižný Hrabovec where rich deposit of this material is located. The objective of technological trials in the locality of Rohatec (well ST-2) was to verify efficiency of manganese and iron removal in water treatment using filtration medium on a basis of chemically modified natural zeolite (Klinopur-Mn). At the same time the efficiency of manganese and iron removal was compared with imported materials Birm and Greensand (USA) which are often used abroad for dissolved manganese and iron removal from water in small-scale water treatment plants (small water resources). It concerns filtration materials with natural MnO_2 layer (Greensand) or artificial MnO_2 layer (Birm, Klinopur-Mn).

3 Experimental Part

The well ST2 located in the grounds of Eurokapital Ltd. in Rohatec is currently used as a resource for supplying the company with drinking water with a capacity of 2 $l.s^{-1}$. It is a dug well equipped with reinforced concrete shaft rings 50 cm high and 100 cm in diameter. The well is 8.1 m deep and 32 years old. Based on repeated analyses, groundwater in the well ST2 contains higher concentrations of iron (about 0.1 to 0.4 $mg.l^{-1}$) and manganese (0.3 to 0.5 $mg.l^{-1}$). The limit value for manganese in drinking water is 0.05 $mg.l^{-1}$ and for iron it is 0.2 $mg.l^{-1}$. The pH value is in the range from 7.12 to 7.26.

Drinking water quality in Slovakia shall meet the requirements of the Regulation of the Government of the Slovak Republic No 354/2006 Coll. and in the Czech Republic it shall be in compliance with the Regulation No. 252/2004 Coll. (limit values for drinking water are the same in both standards).

Methodology for verification of suitable filtration materials for iron and manganese removal is based on their properties and possible technological applications in the water treatment process. The following technological method of water treatment was proposed: raw water → filtration and adsorption.

Raw water (water from well) was pumped by submersible pump to filtration equipment without any pre-treatment and, therefore, removal of Fe^{2+} and Mn^{2+} ions took place directly in a medium of filtration columns.

The following was used as a filtration material:

- natural activated zeolite with MnO_2 (Klinopur-Mn),
- Birm,
- Greensand.

3.1 Filtration Materials

Klinopur-Mn is produced in Slovakia and it is activated zeolite – clinoptilolite (Table 1 and 2). On the surface of grains there is a factory-made film consisting of manganese oxides enabling this material to be used in contact filtration. This filtration material is much cheaper compared to imported materials from the USA. Based on experiments (pilot tests) performed by experts of the Department of Sanitary and Environmental Engineering at the Faculty of Civil Engineering of the University of Technology in Bratislava [8] it can be stated that the surface of clinoptilolite activated by manganese oxides is comparable with imported material Birm and it is possible to use it for removal of Fe and Mn from water.

Table 1. Mineralogical analysis of zeolite from the deposit in Nižný Hrabovec

Mineral	Content (%)	Mineral	Content (%)
Clinoptilolite	84	Illite	4
Cristobalite	8	Quartz	traces
Feldspar	3-4	Carbonate minerals	traces (<0,5%)

Table 2. Chemical analysis of clinoptilolite from the deposit in Nižný Hrabovec

Compound	Content (%)	Compound	Content (%)
SiO_2	66.4	MgO	0.56
Al_2O_3	12.2	Na_2O	0.29
K_2O	3.33	MnO	0.02
CaO	3.04	TiO_2	0.15
Fe_2O_3	1.45	P_2O_5	0.02

Clinoptilolite $(NaK)_6(Al_6Si_{30}O_{72})\cdot 20H_2O$ is one of the most frequently used natural zeolites. At present it is also applied to water treatment process. Sufficient mechanical resistance, chemical stability and abrasion values, even if they categorize it among soft materials, enable clinoptilolite to be used as a filtration material. Basic properties of clinoptilolite are shown in Table 3.

Table 3. Basic properties of clinoptilolite

Clinoptilolite			
Colour	grey-green	Effective diameter of pores	0.4 nm
Compressive strength	33 MPa	Absorbability	34 – 36%
Specific gravity	2200 - 2440 kg.m^{-3}	Water solubility	0
Mass density	1600 - 1800 kg.m^{-3}	Thermal stability	do 450^{o}C
pH	6.8 – 7.2	Stability against acids	79.50 %

Birm is a granulated filtration medium (imported from the USA) used mainly for removal of iron and manganese from water. It is a specially developed material with MnO_2 film on the surface (serves as a catalyst). Properties of Birm are listed in Table 4. Birm is recommended to be used at lower concentrations of iron (up to Fe^{2+} = 6,0 mg.l^{-1} and Mn^{2+} = approx. 3,0 mg.l^{-1}) and for household water treatment. It can be used in gravity or pressure filters. Treated water shall not contain oils, sulphates, organic substances and high concentration of chlorine. Water with low oxygen level has to be pre-treated by using aeration.
Efficiency of Birm also depends on pH value. Water with pH < 6.8 should be treated by adding the alkaline agents. The most suitable pH value is in the range from 8.0 to 9.0.

Table 4. Basic properties of Birm

Birm			
Filtration material	Operating range	Filtration material	Operating range
Manganese content	< 2 mg.l^{-1}	Alkalinity	> 2x (SO_4^{2-} Cl^-)
Iron content	< 8 mg.l^{-1}	Organic matters	< 5 mg.l^{-1}
Temperature range	3 to 45°C	Free chlorine (Cl_2)	< 0.5 mg.l^{-1}
pH	6.8 to 9.0	H_2S	= 0 mg.l^{-1}
Dissolved oxygen	>15% of Fe content	Oils	= 0 mg.l^{-1}

Greensand is a glauconitic mineral of zeolite-type structure (imported from the USA). It is produced from glauconitic sand which is activated by potassium permanganate ($KMnO_4$). The resulting product is a granulated material covered by MnO_2 film on its surface and other higher oxides of manganese. It is used for removal of iron, manganese and hydrogen sulphide from water. Dissolved iron and manganese are oxidized and precipitated in contact with higher oxides of manganese on the surface of Greensand. Undissolved iron and manganese are trapped in „greensand medium" and removed by backwash. After exhaustion of oxidizing capacity the bed is regenerated by using $KMnO_4$ solution. Regeneration frequency depends on amount of iron, manganese and oxygen in water as well as filter size. We recognize two regeneration processes – with discontinuous or continuous regeneration.
The pH value of water is important factor influencing the efficiency of filters. If the pH is lower than 6.8 the efficiency of greensand is reduced. Greensand operation conditions are listed in Table 5.

Greensand has been used for more than 20 years for removal of Fe and Mn from water. The advantage is that water with low oxygen content does not have to be pre-oxidized. Greensand can be used in cases with higher content of iron (over 10 $mg.l^{-1}$) and manganese. It can be also used in industry. The content of organic substances, oils and hydrogen sulphides has adverse effect on its efficiency. Selected parameters of filtration materials used in experiments are listed in Table 6.

Table 5. Conditions of Greensand use

Greensand			
Parameter	Operating range	Parameter	Operating range
pH	6,8 do 9,0	Organic matters	< 5 $mg.l^{-1}$
Dissolved oxygen	> 15% of Fe content	Oils	= 0 $mg.l^{-1}$
Fe^{2+}/Mn^{2+}	< 15 $mg.l^{-1}$	Temperature	5 - 30°C
Alkalinity	> 2x (SO_4^{2-} Cl^-)	H_2S	< 5 $mg.l^{-1}$

Table 6. Filtration materials and some selected parameters

Material	Clinoptilolite	Birm	Greensand	Silica sand
Grain size [mm]	0,3 – 2,5	0,48 – 2,0	0,25 – 0,8	0,7 – 2,0
Specific gravity [$g.cm^{-3}$]	2,39	2,0	2,4 – 2,9	2,66
Apparent density [$g.cm^{-3}$]	0,84	0,7 – 0,8	1,36	1,55
Porosity [%]	64,8	-	-	41,7
Abrasion [%]	8,2	-	-	0,57

The quality of raw water (Fe and Mn content) and treated water at outlet from separate filtration columns was monitored during the experiments. At the same time the amount of water at inlet to filtration columns and water discharge at outlet from the columns were measured by water meter.

3.2 Water Treatment Model

To verify the efficiency of iron and manganese removal from water resource in the locality of Rohatec three filtration columns containing Birm, Greensand and Klinopur-Mn were used. Adsorption column was made of glass. The parameters of adsorption column are as follows: diameter = 5.0 cm; height = 2 m; surface = 19.635 cm^2; filtration medium height = 120.0 cm (Greensand) and 125 cm (Birm, Klinopur-Mn).

The results of analysis of water from ST-2 well are listed in Table 7 (it is an analysis of some selected parameters).

Raw water passed through the filtration columns in downward direction and the average filtration rate was in the range from 8.56 $m.hour^{-1}$ (Klinopur-Mn, Greensand) to 9.02 $m.hour^{-1}$ (Birm). Filtration conditions are shown in Table 8.

Table 7. Results of ST-2 well water analysis

Parameter	Unit	ST-2	Parameter	Unit	ST-2
pH		7,12	NH_4^+	mg.l^{-1}	0,03
conductivity	mg.l^{-1}	80	Fe total	mg.l^{-1}	0,44
colour	mg.l^{-1}Pt	7	Mn	mg.l^{-1}	0,517
turbidity	ZF	2	TOC	mg.l^{-1}	2,2
RL(105°C)	mg.l^{-1}	490	Humic substances	mg.l^{-1}	2,30

Table 8. Filtration conditions

Parameter	Birm	Greensand	Klinopur-Mn
Grain size (mm)	0,42 – 2,0	0,25 – 1,0	0,1-2,5
Height of filtration medium (cm)	125	120	125
Volume of filtration medium (cm^3)	2453	2355	2453
Average discharge through column (ml.min^{-1})	295	280	280
Average filtration rate (m.hour^{-1})	9,02	8,56	8,56
Filtration total time (hour)	652	652	652
Average volume of water flown through (m^3)	11,54	10,95	10,95
Average residence time in column (min)	8,32	8,41	8,76

4 Results and Discussion

The results of experiments are best described in Fig.1 and Fig. 2 showing iron and manganese concentrations in raw water (iron and manganese concentrations were reduced by gradual water pumping) and values measured after water passed filtration materials. The figures also show the limit value for manganese (0,05 mg.l^{-1}) and iron (0,2 mg.l^{-1}) in drinking water pursuant to the Government Regulation No. 354/2006 Coll. Regeneration time of filtration media is indicated by the arrow.

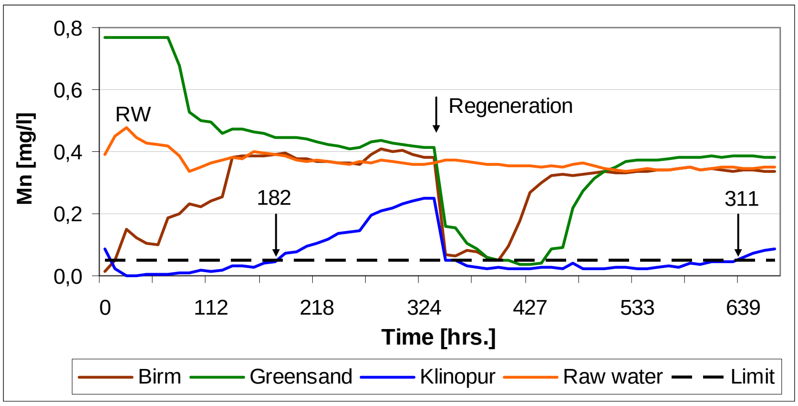

Fig. 1. Manganese removal process during water filtration in Rohatec

The best results were obtained by using Klinopur-Mn (Fig. 1) and therefore we continued in monitoring of manganese removal efficiency (up to exceeding the limit

0.05 mg.l-1) and duration of filtration cycles, i.e. operation of this filtration medium. Measured values are shown in Fig. 3. The total time of filtration of water from ST-2 well was 1293 hours – 21.5 m^3 of water was filtered during this period.

In the first filtration cycle the concentration of manganese exceeded the limit value in treated water after 182 hours of operation, in the second filtration cycle after 311 hours and in the third filtration cycle after 432 hours (Table 9).

Table 9. Measured values during filtration – filtration time

Filtration cycle	Filtration total time [hour]	Filtration time until exceeding the limit 0,05 $mg.l^{-1}$ [hour]
1st cycle	328	182
2nd cycle	421	311
3rd cycle	544	432

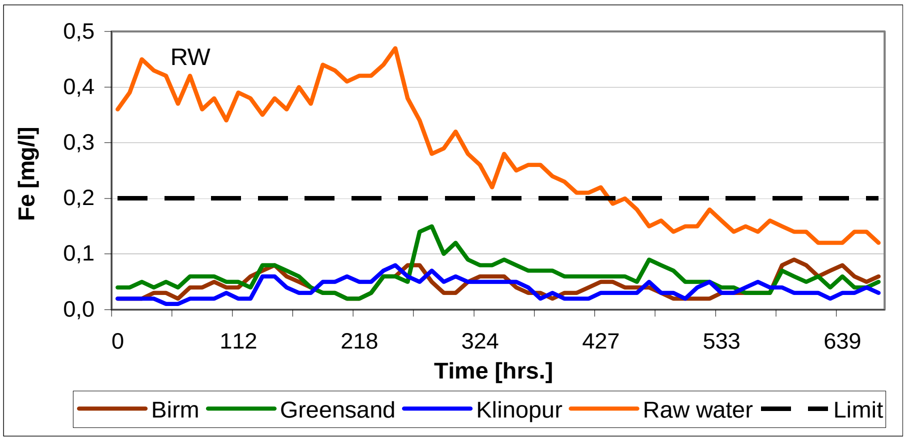

Fig. 2. Iron removal process during water filtration in Rohatec

Filtration media were backwashed approximately one time in five days (according to amount of precipitated ferric hydroxide trapped in a filter). After some time, as can be seen in Fig.3, the concentration of manganese in treated water increased to 0.05 $mg.l^{-1}$ after filtration by Klinopur-Mn and then the Klinopur medium was regenerated by potassium permanganate solution (0.5 % solution). After regeneration, the values of dissolved manganese were in compliance with the Regulation of the Government of the Slovak Republic No. 354/2006.

Filtration time period without regeneration was gradually extended. It means that industrially activated clinoptilolite (Klinopur) is necessary to be used several times directly on site which will result in prolonged filtration cycles. After some time no regeneration will be required.

Measured values of water filtered through the filtration column with Klinopur and efficiency of manganese removal (considering limit value 0.05 $mg.l^{-1}$ Mn) in individual filtration cycles are shown in Table 10.

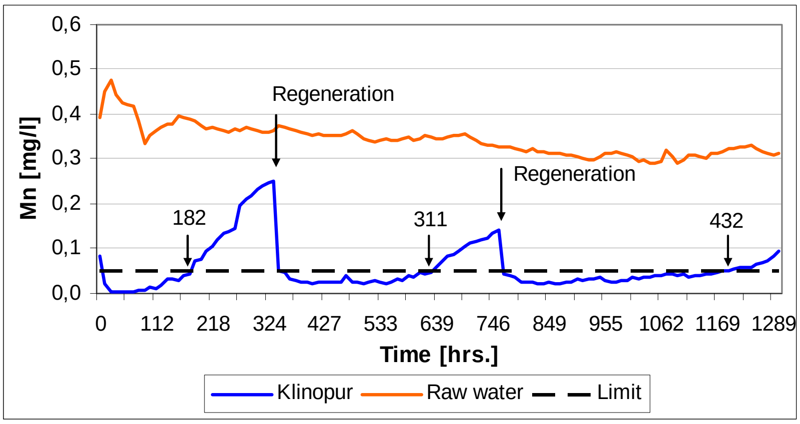

Fig. 3. Manganese removal process during filtration using Klinopur-Mn

Table 10. Measured values during filtration – amount of filtered water

Filtration cycle	Total amount of filtered water (m^3)	Amount of filtered water until exceeding the limit 0,05 $mg.l^{-1}$ (m^3)
1st cycle	5.51	3.05
2st cycle	6.85	5.22
3rd cycle	9.14	7.25

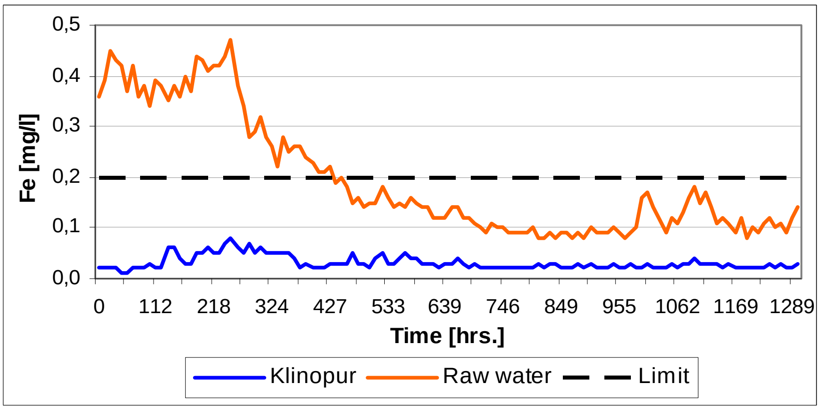

Fig. 4. Iron removal process during water filtration using Klinopur-Mn

Klinopur-Mn can also be used for removal of iron from water. Our results show that during the whole period of measurements (1293 hours) iron concentration did not exceed the limit value 0.2 $mg.l^{-1}$ specified pursuant to the Government Regulation No. 354/2006 (Fig. 4). Sequential pumping of water from well led to decrease of iron concentration in treated water below the threshold value 0.2 $mg.l^{-1}$.

5 Conclusions

Obtained results provide *the basis* for using the Klinopur-Mn in manganese and iron removal from water (so called contact manganese removal). Other monitored materials showed lower efficiency of manganese removal from water (Fig. 1); however, they were effective in iron removal (Fig. 2). Pilot measurement was also used for testing the filtration rates, backwash time, filtration medium regeneration using $KMnO_4$ and the effect of pH (pH did not influence the efficiency of proposed procedure).

Usability (advantage) of this method in water treatment is also demonstrated by the ratio of filtered water volume and filter medium volume (2453 cm^3), (Table 11).

Table 11. Filtered water volume/ filter medium volume ratio

Parameter	Klinopur-Mn
Filtered water volume/ filter medium volume ratio	8854
Filtered water volume/ filter medium volume ratio up to limit (1st cycle)	1243
Filtered water volume/ filter medium volume ratio up to limit (2nd cycle)	2128
Filtered water volume/ filter medium volume ratio up to limit (3rd cycle)	2955

Technological trials were performed within the grant project VEGA 1/4208/07 and the project APVV-0379-07. At the same time we would like to thank the staff of the company Eurokapital in Rohatec for helping us in the performance of experiments.

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A44**

An Additional Contribution to the Pressure Management

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Abstract. The paper deals with technique of the remote control of water demand and pressure in the districts of water distribution system. Application of new practices of remote control allows to monitor the network in detail and more efficient. For example, district meter areas set out the procedures of detail system control. In the paper are given the results of such remote procedures in two districts areas of water network. The results of remote measurement and its events were used to analyse the hydarulic parameters, water losses and were used to optimisation of network as well.

Keywords: Water supply, water losses, water consumption, pressure reduction, pressure management

1 Introduction

Operation of public water supply system requires permanent monitoring. To perform reliable monitoring it is necessary for provider to know the whole water supply system. It means not only topological but also hydraulic properties. In addition to invoice measurements at the beginning and end of water supply it is important to know basic hydraulic variables (discharge, pressure, pressure loss – not only static parameters but also dynamic parameters changing during hours) at important points of water distribution such as water reservoirs, pumping stations and shafts. These measurements are called “distribution measurements”. Monitoring of water reservoirs and pumping stations is a common part of water supply control system, while distribution measurements in shafts do not exist or they are carried out without remote data transfer. At present, measuring and regulating devices, wireless communication as well as data digitalization brings relatively simple solution to monitoring of the entire water supply system for reasonable costs. New options for monitoring of urban water supply system allow provider to manage it in more reliable way.

2 Distribution Measurements in District Meter Area

Urban water supply network usually represent very complicated multi-circuit hydraulic system with several water reservoirs, pumping elements or even system with more water resources. Although the multi-circuit system with the possibility of variant flow directions according to current need is evidently more advantageous, operational experience also shows disadvantages of this network. Its main disadvantage is that the supply system is not transparent. This non-transparency disables quick selection of action in case of emergency, does not allow balance process for specification and localization of water usage and hence water losses. Moreover, it does not allow determination of water usage and demand in urban districts for the purpose of planning the water supply reconstruction or expansion. Many water companies began to develop simple district meter districts of water supply systems.

The division of larger urban water supply systems is mainly hydraulic problem and it is difficult to find solution that would have no adverse effect on existing state. Mathematical model is a useful tool for assessment of particular solutions.

The main objective of dividing the water supply system in districts is to get clearly and easily the information on discharge and pressure conditions not only in districts themselves but also in the entire urban water supply network. The second important goal is to provide the monitoring of water supply system with data transfer from the measurement points to dispatching centre.

The main principles of dividing the supply system in districts are as follows:

- first of all, the proposal of district boundaries and measurement points shall take into account the conditions for construction of measurement point (real possibility of constructing or reconstructing the shaft);
- district boundaries shall be based on existing districts, existing preferential flow paths and shall take into account pressure demands in the districts;
- proposal of districts shall take into consideration the water reservoirs and their capacity;
- boundaries of selected districts should provide alternative solutions of their operation, which means to redirect flows into the significant districts such as the central district of a city or main section of consumer's place.

Several studies on optimization of urban water supply systems aimed at creation of districts, optimization of pressure conditions or evaluation of water losses were conducted at the Department of Sanitary and environmental Engineering within the scientific-research works. The studies were based on the existing conditions verified by mathematical model. The models of water supply systems of Lucenec, Zilina, Poltar, Kysucke Nove Mesto, Cadca and many other cities were designed and calibrated. An importance to re-evaluate the monitoring of network together with pressure conditions is mainly in larger cities with the increased construction of tall buildings such as in the town of Zilina where tall buildings are placed in the areas with low-rise buildings.

Division of network in separate districts is an intervention in its hydraulic conditions, leads to change in flow direction and discharges, flow velocities as well as significant change in pressure conditions. Water supply network consist of the system of pipes that is adapted to the certain hydraulic regime (load) repeating every 24 hours only with slight variations. Every change in the "learned" regime may result in increased number of failures or increased turbidity in pipeline. Therefore, it is very important to create the districts step by step and monitor every change in the system. The same process is applicable for adjustment of the pressure using reduction elements.

2.1 Pressure Conditions

Division of water supply system in districts provides definite flow direction and measurement of water volumes supplied to the district and on the other hand it gives an opportunity to create a space for example for regulation of high pressures. Reduction of pressure in the districts leads to its stabilization and hence in decrease of pressure variations. It is very favourable for technical conditions of pipelines. The usual mistake done by provider is that high pressure is reduced in the section of pipe (throttled pipe) at the inlet to the zone with high pressure. It results in so-called"false" pressure reduction - the pressure is reduced only when the water flows through pipes. In the night hours, when the discharge is minimal, the pressure will rise to its initial high level. Pipes are loaded by increased fluctuation of pressure, which may lead to increased loading of pipes and more frequent occurrence of failures. Such "false" reduction may result in decrease of pressure to zero in case of emergency situations (fire fighting).

Today, intelligent reduction fittings reducing pressure according to pressure demand at the consumer site – discharge at the measurement site or pressure at the consumer site. Such reduction elements are very appropriate support for district distribution measurements.

3 Monitoring of Water Losses in Measurable Districts

At present, pressure and discharge measurements are performed at our department. They are aimed at explaining the effect of changes in pressure on discharge and related water losses in the separately measurable districts. For the purpose of measurement two successive separate districts – municipalities Dlha and Borova – were selected in cooperation with the Bratislava Water Company (figure 1). Pressure and discharges are measured at the inlet to the Dlha in a armature chamber equipped with the pressure-reducing valve. After entering the district, water passes through the consumption site and flows into the access shaft of Borova district, which is supplied by pipes passing through the municipality of Dlha.

The devices recording pressure and discharges of water are installed in both gauging chambers. Discharges and pressures are recorded in data loggers in 10-

minute intervals. Data are saved in the memory of data loggers and transferred in the package ones a day through GSM network to our workplace where they are processed.

Detailed analysis of the night discharges identified water losses in the water supply system (figure 2) and through the detailed analysis of the time course of pressure and discharge in two successive districts the location of higher water losses was computed. Figure 2 shows discharges from distribution measurements before and after repair of failure identified by measurement.

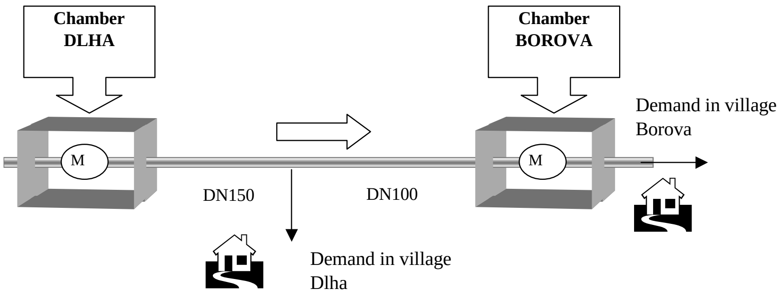

Fig. 1. Measuring districts Dlha and Borova

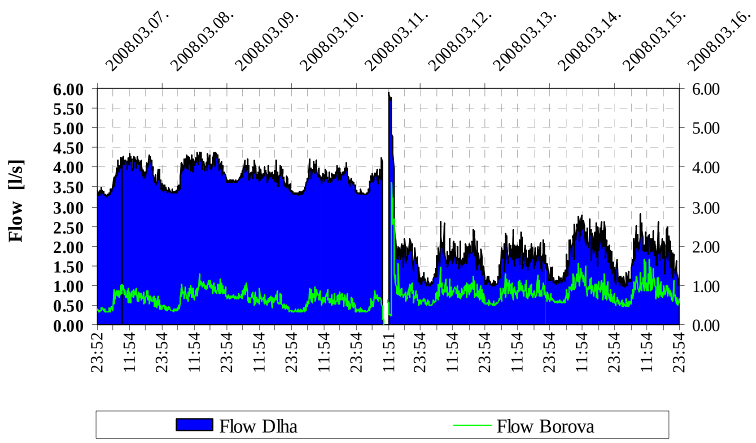

Fig. 2. Continual records of distribution measurements – failure repair

The advantage of this device is that it is able to transmit warning message on emergency situation in the water supply system. Quick detection of failure reduces an overall financial loss. Figure 3 shows the occurrence of failure and its repair.

Figure 4 show that the increased night consumption means not always the failure of supply system.

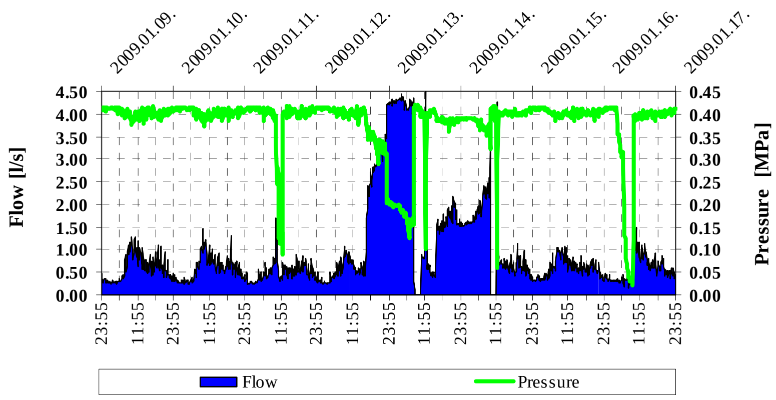

Fig. 3. Continual records of distribution measurements – failure repair

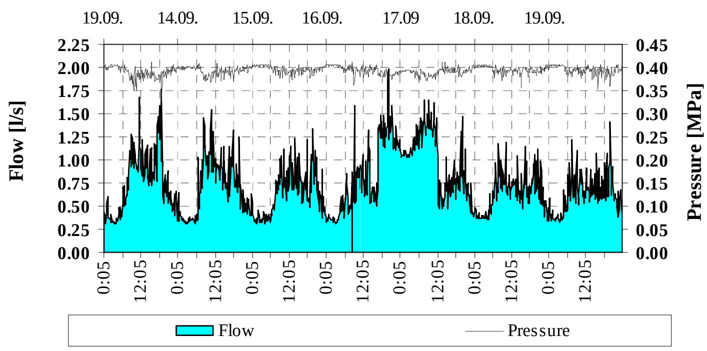

Fig. 4. Continual records of distribution measurements – increased discharge but not failure

This device helps provider to gather information not only on the volume of water usage measured by water meter but also on the time course of pressure during the day, fluctuation of values and minimum night discharges. Based on these data provider knows how the system works during the day when it is overloaded and also is able to estimate the water losses.

4 Conclusions

Urban water supply network is a very complex hydraulic system that is determined by altitudinal and situational distribution of the water consumption site, location water resources, water reservoirs and supply systems.

To provide reliable operation and sufficient water quantity and quality the permanent monitoring of all elements of the water supply system is required. First of all it is necessary to know the actual state of pipelines, their location, properties (DN, material, age) and existing regulation elements (regulating valves, stop valves, etc.). Another important assumption is to provide permanent monitoring of variable system properties. The first step for providing the control of water supply network is to establish the measuring districts.

In the time of advanced information technologies and growing demands on quality of delivered products and services the provider should use modern technologies for recording the data and their remote transfer in order to make the operation of water supply system as efficient as possible. While the water supply dispatching routinely provides online monitoring of water resources, water reservoirs and pumping stations, the data transfer from distribution measurements done in the district shafts is not common. Distribution measurements can detect hidden failures, water thefts (losses) and they can also help to take prompt action in case of emergency situation. The analysis of water supply system leading to its dividing in the districts is important for providing distribution measurements. Investment costs for the construction of distribution district measurements can be recovered in a short time period.

Acknowledgement

The article is supported by the Scientific Grant Agency of the Ministry of Education – VEGA Project No. 1/0854/08 dealt with at the Department of Sanitary and Environmental Engineering, Faculty of Civil Engineering, Slovak University of Technology Bratislava This study is supported by the Science and Technology Agency under the Contract APVT-20-031804.

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A45**

Combined versus Separated Sewer System in Slovakia

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Abstract. The sewer system is the most important system for the basic municipality operation. The history showed us, that various sewer systems were use for the municipality drainage. The view, which one sewer system is suitable for the drainage of the big cities, villages or municipalities depend on many technical, financial, conception factors. The Slovakia is the relatively small country with not more than 2891 municipalities, with the some relatively big cities. The all country inhabitants are not more than 5,4 mil. The factors which has the main influence of the sewer system built depend on the municipalities drainage necessity and on the money availability. The world financial crisis has a serious signification on this chance to sewer system built, because the huge investment for sewer systems is very hardly available.

Keywords: Sewer, design, combined system, separated system

1 Introduction

The background of the basic question: "To build the combined or separated sewer system" is implicated by the serious questions outgoing from the sanitary, social, technical and economical positions. The students in technical school are informed mainly from the sanitary factor. The next one is technical chances to build the system and last one is financial factor. The history showed us, that the decision, which one sewer system building depends on the local conditions. The Slovakia has not more than 10 big cities, where in the history were build the combined sewer system.

Only 1,1 milliard people on the world are connected on the sewer system, from this only 330 million are connected to the biological WWTP. 2,8 milliard people of the world uses the various types of latrines, 2,4 milliard people has no sanitation.

The Central Europe occupy 80 million people, which are no connected to the sewer system, 70 mil are connected, from this 30 mil are connected without the biological stage of waste water treatment. The Central Europe contain about 142 650 municipalities. From this number 109.640 are with inhabitants less than 2000, which represent approx. 30 mil people. The Slovakia region is covered by sewer system in 57,6%. The municipalities less than 2000 are 2512 from total 2891, which represent 1,65 mil people from 5,4 total.

These numbers shows, that the question, what to built, is very complicated. The first goal is to dewater the domestic waste waters; the next one goal is to dewater the rain water. But the environmental in present time accentuate to protect the recipients. The type of used sewer system has the significant influence.

2 Separated Sewer Systems

Is the basic requirement for the inhabitants which want to live in some municipality. The main rule of the separated sewer system is dewatering the domestic waste water from residence to WWTP through the sewer system. The people, which are lived in their residences are not very often informed, which sewer system dewater their waste water to WWTP. This illiteracy cause the problems with storm water dewater, mainly in residence areas, where the people connect their roofs and pavement areas to he sewer system in some cases, which is designated only for the domestic waste waters. People assert, that they pay the waste waters, so they have accrue dewater the roofs and pavement areas from their residence.

This misunderstanding cause the problem mainly with pumping stations on separated sewer system. The rule of water companies is to inform about the rule and type of sewer system, directed to residences. The Slovakia sewer pipes designed for domestic waste waters dewatering use the basic formula for the amount of waste water determining by the (1)

$$Q_{dwf} = I_{no} \cdot q_{wwp} \cdot k_h \tag{1}$$

where: Q_{dwf} – amount of domestic waste waters; I_{no} – number of inhabitants connected to the actual computed point, k_h – maximum hourly coefficient, which depend on the inhabitants number I_{no}.

The Eq. (1) computed only the amount of domestic wastewaters, but it is not suitable for the pipe design. The pipe design depends on the computed amount of waste water Q_{d-dwf}, and this value is twice Q_{dwf}:

$$Q_{d-dwf} = 2 \cdot Q_{dwf} \tag{2}$$

The multiplication factor 2 show, that the pipe will be fill in to the 50% of their capacity. The remains capacity is the reserve, which is used for the annual cleaning the sewer pipe under working arrangements.

3 Combined Sewer Systems

On the other hand with the same goal concerning domestic waste waters and with the goal to dewater the storm waters, is combined sewer system. This system type is not young; it was built in the history in many huge cities, such as Paris, London, New York and many others. The purpose is to dewater in the same pipe rain water together with domestic waste waters.

What is the advantage of this sewer system type? In the history of big cities this system provided the self cleaning service of sewer system using ordinary for dry weather flow respectively for dewatering the domestic waste water. The waste waters were drain directly to recipient, because the historical first problem was to drain waste waters and the idea about the waste water treating was not interested.

New era, we can talk about last hundred years show us, that the non-treated waters cause the various problems in the recipients. The huge recipients, such as big rivers, have the self cleaning ability concerning waste waters. But the population density increasing, together with living standard, causes the enormous amount of pollution increasing. Many rivers lose the self cleaning ability. It was the one reason for waste water treatment plant (WWTP) building.

The combined sewer system, dewatering the waters to the waste water treatment plants, causes the non-uniformity flow to the WWTP. That means, in dry periods the pollution goes directly to the WWTP. In wet period, the municipal waste waters are mixed with the storm waters. And this is the problem, because the WWTP is designed only for the municipality waste water treating and some polluted parts of storm water, mean first flush. The other storm waters needs to separate and send directly to the recipient, because the WWTP capacity and the technology process doesn't allow treat all waste water from the watershed.

This storm water separation is executed through the combined sewer overflow (CSO) structure, which is designed for the specific amount of storm waters separate.

The Slovak big cities uses the combined sewer system for dewater the storm and municipality waste waters.

The basic principle, how to determine the amount of storm and municipality waste waters is based on basic rational formula.

$$Q_{sw} = S\,.\;q_{sw}\,.\psi \tag{3}$$

where: Q_{sw} – amount of storm water, q_{sw} – specific rainfall capacity (similar on specific storm/rainfall intensity) obtained by Eq. (6), ψ – runoff coefficient 0-100%

Specific rainfall intensity q_{sw} represented by Capacity-duration-frequency (CDF) curves are designed for 68 rain stations in Slovakia (Fig. 1), which measure the rainfall intensity in these locations. The CDF curves are used mainly for periodicity/frequency 0.5 or 1.0 depends on locality size. From this curves the method uses the block rains, which is possible to extract from CDF curves.

Formula (3) determines only the storm waters amount. If we want to determine the design amount volume of combined sewer system we use the follow formula:

$$Q_{d-css} = Q_{sw} + Q_{dwf} + Q_b \tag{4}$$

where: Q_{dwf} – represents dry weather flow – municipal waste water, Q_b – ballast waters.

The amount of Q_{dwf} is neglected under the condition, when Q_{dwf} is less than 10% of Q_{sw}. The ballast water is included, because the sewer system is very often untightness.

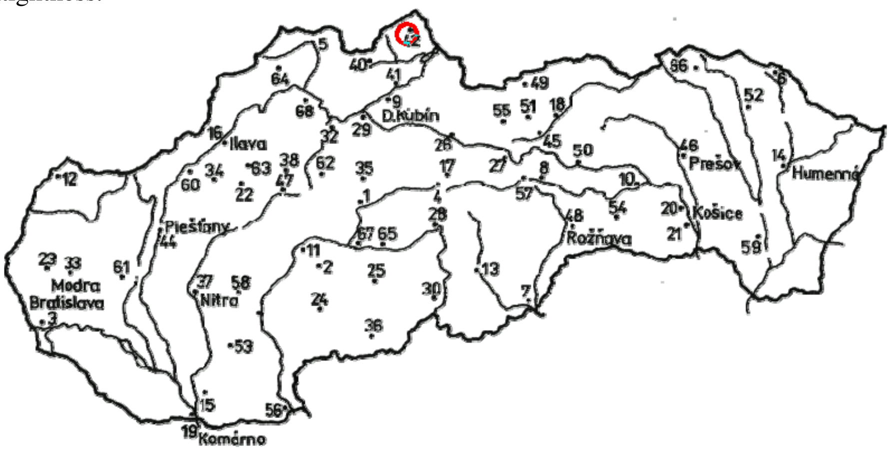

Fig. 1. The rainfall stations map in Slovakia

The formula 4 shows, that in dry weather period, only amount of municipal waste water flow into the sewer system. This flow must be drain into the WWTP.

In the wet weather, when the storm start, the specific amount of waste waters are dewater to the WWTP and remain to the recipient through the CSO structure. The determination, which part will be dewater to the WWTP is not very easy, because on this determination depend the WWTP loading or recipient contamination.

In Slovakia we use the two ways of this waste water amount determination: a) method of the dilution ratio, and b) method of the boundary rain.

The dilution ration method is defined as the ration between domestic waste water and storm water. Usually this ratio is 1:4, but 1:8 in last time. The more dilution mean, that the larger waste water amount will be dewater into the WWTP for treatment process, and it means the greater recipient protection against pollution.

The Boundary rain method outgoing from the watershed size and it is defined by:

$$Q_{boundary} = S.\,\psi\,.\,q_b \tag{5}$$

where: q_b – specific capacity of boundary storm, values from <10-25 l/s/ha>. The choice of this value depends on self-cleaning recipient capability. Generally the large recipient, with the more self-cleaning capability, uses the smaller specific capacity of boundary storm and vice-versa.

The system for design. Many Slovak sewer system were designed and recalculated by the SeWaCAD v2006 computational software system, which can design and recalculate existing sewer systems, combined and separated, by the rational formula. This system is very user friendly and with huge computer capacity. It was designed mostly for designers and the goal of this system is to design the longitudinal profiles of sewer pipes, which is useful for build the sewers. The system works very dynamic, and the hydro-technical computation is directly transferred into the graphical representation. The designer can monitor not only hydro-technical outputs, but actual graphical representation in the same time. This software offers to solve the separated and combined sewer system. Because the combined sewer system design is more complicated when the domestic waste water sewer system, system contain the rainfall tool with the rainfall database from 68 Slovak rainfall stations Figs. 1 and 2.

$$q_{sw} = \frac{K}{t^a + B} \tag{6}$$

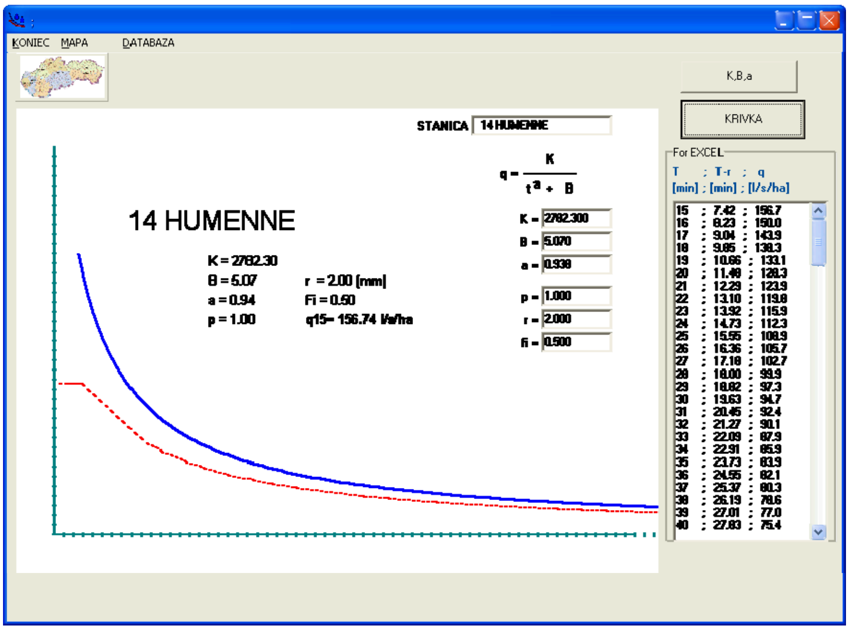

Fig.2. – The CDF curve computed by the SeWaCAD software – example of the CDF curve of the city Humenne

The formula (6) shows the basic principle of the q_{sw} designation, where K and B are the parameters which represents the specific rainfall station, where the measurement data were elaborated, t – represents the time of rainfall duration.

Figs. 3 and 4 show the existing sewer system and indicate the overload of this part of the sewer pipes. The color lines shows the percentage of overload of the combined sewer system (Fig. 4), and the number in brackets shows the value of existing diameter and the other the design diameter of the sewer pipe.

The design of the longitudinal profiles represents very difficult work for designers; because it needs to be careful in the same time with the hydro-technical and graphical computes. Only the repeat computation could bring the requirement results. The system SeWaCAD allows designing not only the hydro-technical computes, but containing the slope generator, which can design the slopes for the designed pipes. Base on slope design, there is possibility of diameter change and vice-versa. The problems occur, when the flat area are solved. In this case we use the condition of minimal slope design for the sewer pipes, which is simply assigned by the Eq. (7).

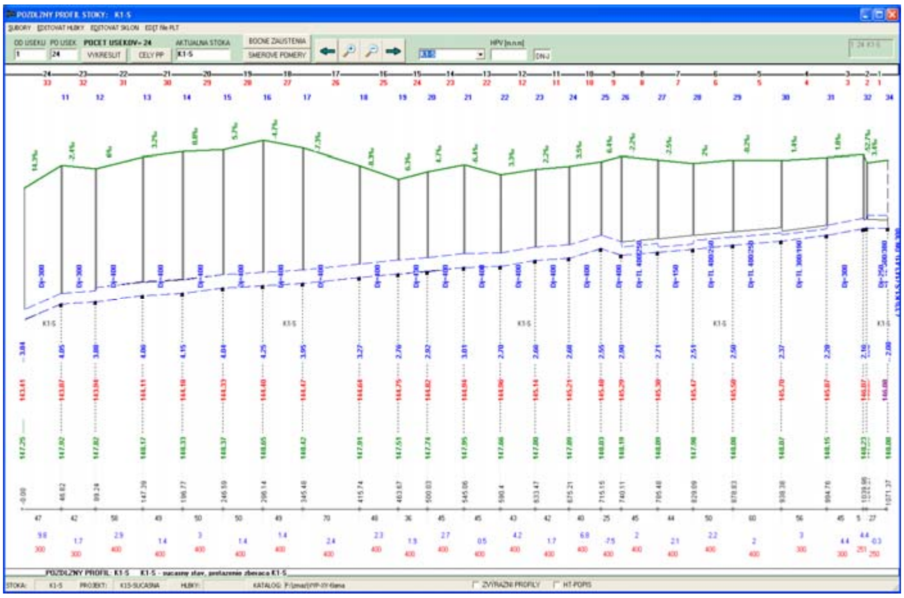

Fig 3. Longitudinal profiles for the sewer system design – example of the existing sewer system

$$i_{min} = \frac{1500}{D} \tag{7}$$

where: i_{min} – minimal sewer pipe slope, D – sewer pipe diameter.

The previous recommendation was set the value 1500 on value 1000. It means that less slope in flat areas were used for the sewer system design. The change was done with respect on the sedimentation function in the sewer pipe. The number 1500 represents the worse sedimentation condition. But this fact has the worse influence on sewer system design, because we achieve the larger excavation in shorter distance. It causes the cost increasing, the more pumping stations designing. The follow factor which has the worse influence on sewer pipes design is the minimal diameter for sewer design use. The recommendation is 250 mm diameter use at his time. The previous years allowed use the 300 mm in diameter for the sewer pipes design. Concerning the minimal slope computation the 250mm has the worse influence on excavation and over-charge the investment costs. But on the other hand, the operational conditions are improved by using the new design conditions.

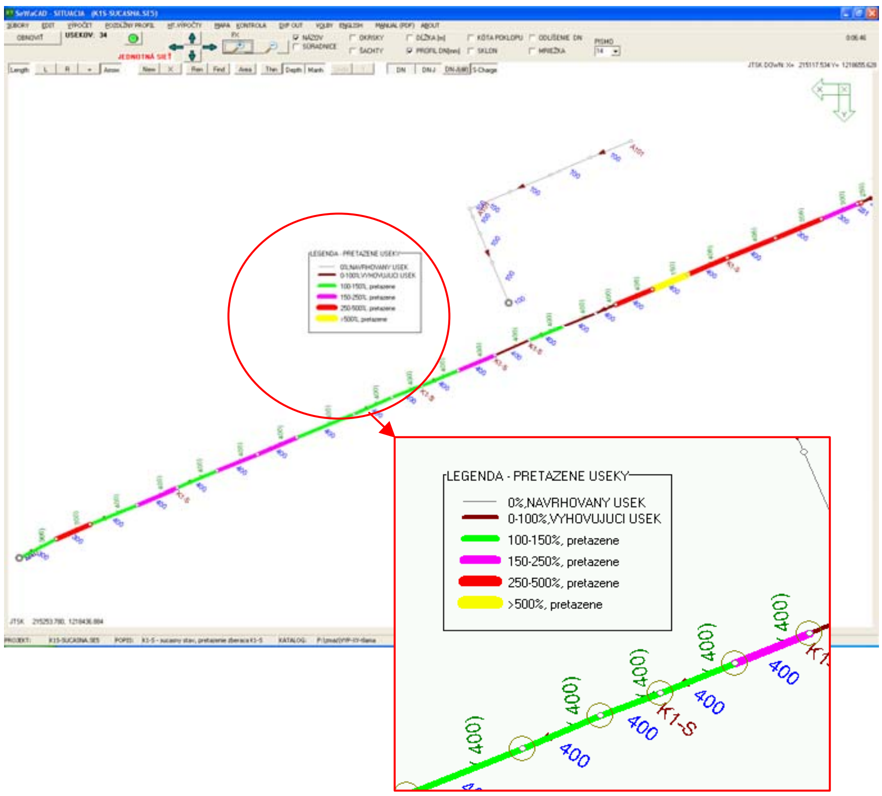

Fig 4. Example of sewer system overloading

4 Conclusions

The choice, which system use for dewatering the water from urban catchments, is not very easy and depends on the municipality size and investment possibilities. Another depending factor is the pollution of the recipient through the CSO structures, which allow saturate the recipient waters with the storm water, which could be contaminated by the pollution of the street, roofs, respectively of the interested catchments. The software which was designed in Slovakia, by the author of this article, SeWaCAD could help with this choice, because it could be very effectively and very shortly give answer on some questions concerning sewer system design and provide the solution how to dewater the waste-waters from the catchments.

Acknowledgement

The article is supported by the Scientific Grant Agency and the Cultural and Educational Grant Agency of the Ministry of Education – VEGA Project No. 1/0854/08 and KEGA Project No. 3/5125/07 and project „Operačný program Výskum a vývoj - Centrum excelentnosti integrálnej protipovodňovej ochrany územia“, kod ITMS 26240120004 dealt with at the Department of Sanitary and Environmental Engineering, Faculty of Civil Engineering, Slovak University of Technology Bratislava.

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A46**

Water Source Protection from Landfills Leachate

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Abstract. The solid waste landfills have become a very difficult problem in the last time. The consideration must be devoted to landfill leachate, which can contaminate the close landfill area. The measurements/monitoring and the accentuation on landfill quality building could bring the positive impact on the surrounding environment. The article focuses the ways of the leachate observation and monitoring on the existing landfills and makes the recommendations for the new one designed and building, with the safety and security occurrence. The liner solid waste landfill system is important too, because it prevents the non-controlled leachate extensions and protects the landfill faults.

Keywords: Leachate, solid waste landfill, water pollution

1 Introduction

Landfills and illegal dumpsites pose a significant risk to human and environmental health. Simply based on the number of sites throughout the country, landfills are one of the largest sources of potential pollution in communities of all types.

By definition [6], a municipal solid waste landfill is a discrete area of land or an excavation that receives household waste, and that is not a land application unit, surface impoundment, injection well, or waste pile, as those terms are defined in law. Household waste includes any solid waste, including garbage, trash, and septic tank waste, derived from houses, apartments, hotels, motels, campgrounds, and picnic grounds. Landfills come in all shapes and sizes and can impact the environment in many different ways.

There are two major sources of contaminants in municipal landfills and dumpsites, leachate and landfill gas. Each is composed of different contaminants and each poses its own set of management burdens for the development. Taken together, they can affect the soils, ground and surface waters, and air in and around the sites of the landfills, many times years after the landfill has been closed.

Leachate is the liquid that results from rain, snow, dew, and natural moisture that percolates through the waste in a landfill or dump. While migrating through the waste, the liquid dissolves salts, picks up organic constituents, and leaches heavy metals, such as iron, mercury, lead, and zinc from cans, batteries, paints, pesticides, cleaning fluids, and inks. The organic strength of landfill leachate can be greater than 20 to 100 times the strength of raw sewage. Landfill leachate is potentially a potent polluter of soil and groundwater. The majority of open dumps and old sanitary landfills do not have liners or proper drainage systems to divert the leachate. Both pose the problem that the leached material could be absorbed into the ground and then possibly move into groundwater, surface water, or aquifer systems [9].

2 Sanitary Landfill and Groundwater Protection

Landfills are engineered areas where waste is placed into the land and sanitary landfill is engineered method of disposing of solid waste on land in a manner that protects the environment (e.g., by spreading the waste in thin layers, compacting it to the smallest practical volume and then covering it with soil at the end of each working day) [10].

No matter how hard we try to avoid generating waste, by waste reduction, re-use, recycling, composting and many other methods of waste pre-treatment prior to landfilling, landfilling will continue and leachate generation is a problem which is here to stay [8].

If there is no base lining the leachate will drain away through any reasonably permeable material, which exists under the dump. Although this material below the dump may do some filtering and further cleansing of the leachate, it can enter the underground strata still in a highly polluting condition [5]. Landfills are developed to solve this problem with the aim of preventing the pollution because this groundwater will be used for drinking water in many parts of the world [7]. The protection of groundwater is a significant factor in sitting all landfills [2]. The geology and hydrogeology of any region have a major bearing on the availability of suitable areas for landfill sites; the level of natural protection for groundwater from contamination by landfill leachate and the design, operation and monitoring of landfills.
The risk to groundwater from the landfilling of waste is mainly influenced by:

The risk to groundwater from the landfilling of waste is mainly influenced by:

- nature of the waste;
- leachate composition;
- volume of leachate generated;
- groundwater vulnerability;
- proximity of a groundwater source;
- value of the groundwater resource;
- landfill design;
- landfill operation and management practices.

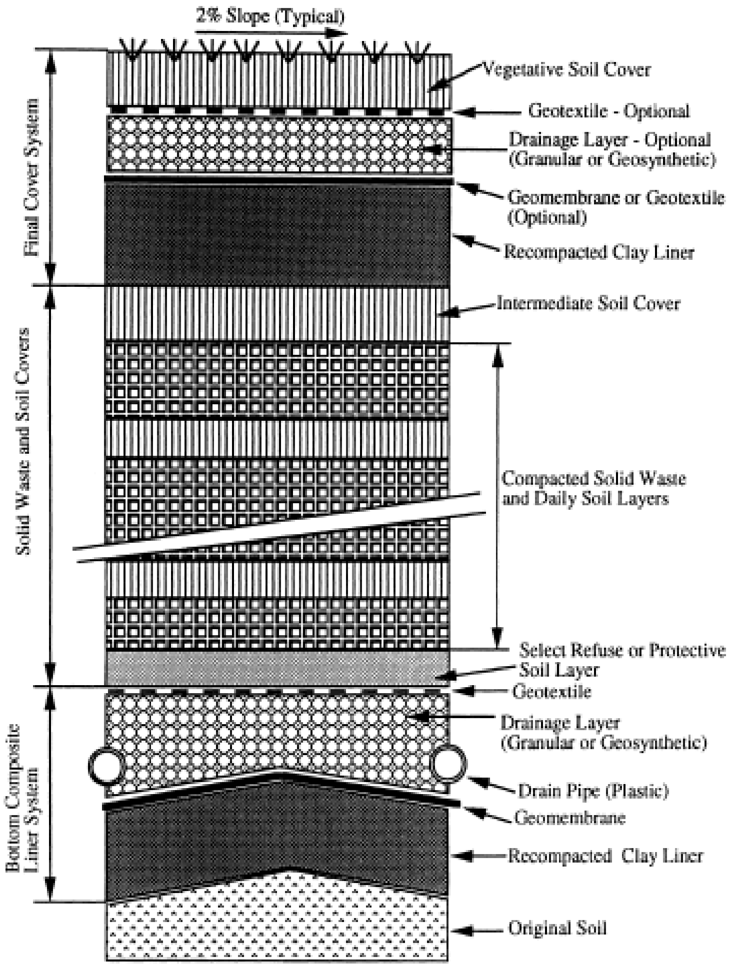

Fig. 1. The typical landfill vertical profile

Best practice in the sitting, design and construction (Fig. 1), and operation of sanitary landfills are [10]:

- be consistent with local land use conditions and zoning codes;
- protect flood plains and wetlands;
- protect against problems caused by unstable geological settings;
- provide for best practices in design, construction, operation and closure;
- minimize impact an air, water and soil quality or otherwise adversely impact public health safety and welfare;
- provide for controlled access to the site;
- provide for use by individuals or public use areas;

- provide means for the measurement by weight or screening of incoming solid waste;
- provide control of run-on and run-off;
- provide prevention of groundwater and surface water contamination;
- provide prevention for air quality contamination;
- provide groundwater, surface water and landfill gas quality monitoring systems;
- provide management and control landfill gas;
- provide recovery and flaming of the landfill gas (where necessary and when economically feasible) and utilization (as an energy sources);
- provide collection, recovery and management of leachate;
- allow efficient and safe operations.

Operation of sanitary landfill should prescribe to the following principles:

- provide controlled access and use only authorized users;
- provide use by individuals at convenience areas, public drop-off areas, or public use areas;
- measure all incoming solid waste by weight and quality;
- conduct random inspection on incoming loads to detect and prevent the disposal of hazardous waste;
- accept only wastes includes in the permit, permit conditions are permit amendments;
- provide training of all on-site personnel and encourage certifications of landfill managers;
- provide use of daily cover (earth or alternative materials),
- provide vector and birds control;
- control run-on and run-off;
- prevent groundwater contamination;
- prevent surface water contamination;
- prevent air quality contamination;
- prevent the migration of landfill gas.

Closure and post closure of sanitary landfills should prescribe to the following principles:

- provide financial assurance for each individual facility for closure and post-closure care, and for identified corrective action;
- continue monitoring to meet permit requirements,
- evaluate the end use of the site in consideration of the potential damage to the final cover system and the proper removal and management of leachate and landfills gas;
- restrict access to monitoring and control systems of the closed facility to authorized personnel;
- note former landfill use in records.

3 The Leachate

One of the most important problems with designing and maintaining a landfill is managing the leachate that is generated when water passes through the waste [4].

Leachate is generated as a result of the percolation of water or other liquid through wastes and the squeezing of the moisture contained in the waste due to its own weight. Thus, leachate can be defined as a liquid that is produced when water or other liquids comes in contact with waste. Leachate is a contaminated liquid containing dissolved and finely suspended solid matter and microbial waste products. The generated leachate will migrate into subsurface water when confining layers beneath the landfill are absent or inadequate.

Leachate production and composition varies relative to the amount of precipitation and the quantity and type of wastes disposed. Good site selection, design and operation assist in minimising the risk of pollution. Calculating the volume and predicting the composition of leachate that will be generated can evaluate the potential of pollution.

3.1 Leachate Production

The volume of leachate depends principally on the area of the landfill, the meteorological and hydrogeological factors and the effectiveness of the capping. It is essential that the volume of leachate generated be kept to a minimum. The design and operation of the landfill should ensure that the ingress of groundwater and surface water is minimized and controlled.

3.2 Leachate Composition

Leachate composition varies due to a number of different factors such as the age and type of waste and operational practices at the site. The leachate consists of many different organic and inorganic compounds that may either dissolve or suspended. The conditions within a landfill vary over time from aerobic to anaerobic thus allowing different chemical reactions to take place. Most landfill leachate has high BOD, COD, ammonia, chloride, sodium, potassium, hardness and boron levels. Ammonia is a contaminant, which may be used as an indicator of contamination, particularly in terms of surface water, as it can be toxic to fish at low concentrations (1 mg/l). Chloride is a mobile constituent, which is often used as an indicator of contamination. The leachate from landfills for non-hazardous waste may produce reducing conditions beneath the landfill, allowing the solution of iron and manganese from the underlying deposits. Leachate from landfill sites for non-hazardous waste often contains complex organic compounds, chlorinated hydrocarbons and metals at concentrations, which pose a threat to groundwater and surface waters. Solvents and other synthetic organic chemicals are a significant hazard, being of environmental significance at very low concentrations and resistant to degradation. Moreover, they may be transformed in some cases into more hazardous compounds.

4 Landfill Liners

Landfill liners are designed and constructed to create a barrier between the waste and the environment and to drain the leachate to collection and treatment facilities. This is done to prevent the uncontrolled release of leachate into the environment. Society produces many different solid wastes that pose different threats to the environment and to community health. Different disposal sites are available for these different types of waste. The potential threat posed by the waste determines the type of liner system required for each type of landfill. Liners may be described as single (also referred to as simple), composite, or double liners [3].

4.1 Single-Liner Systems

A single liner system includes only one liner and a leachate collection system (LCS) above the liner [1]. Single liners (Fig. 2, Box 1) consist of a clay liner, a geosynthetic clay liner, or a geomembrane (specialized plastic sheeting). Single liners are sometimes used in landfills designed to hold construction and demolition debris (C&DD).

Construction and demolition debris results from building and demolition activities and includes concrete, asphalt, shingles, wood, bricks, and glass. These landfills are not constructed to contain paint, liquid tar, municipal garbage, or treated lumber; consequently, single-liner systems are usually adequate to protect the environment.

4.2 Composite-Liner Systems

A composite liner consists of a geomembrane in combination with a clay liner (Fig. 2., Box 2). Composite-liner systems are more effective at limiting leachate migration into the subsoil than either a clay liner or a single geomembrane layer. Composite liners are required in municipal solid waste (MSW) landfills. Municipal solid waste landfills contain waste collected from residential, commercial, and industrial sources. These landfills may also accept C&DD debris, but not hazardous waste.

4.3 Double-Liner Systems

A double liner consists of either two single liners, two composite liners, or a single and a composite liner (Fig. 2, Box 3). The upper (primary) liner usually functions to collect the leachate, while the lower (secondary) liner acts as a leak-detection system and backup to the primary liner. Double-liner systems are used in some municipal solid waste landfills and in all hazardous waste landfills.

Hazardous waste landfills (also referred to as secure landfills) are constructed for the disposal of wastes that once were ignitable, corrosive, reactive, toxic, or are designated as hazardous by the legislations. Hazardous wastes are produced by industrial, commercial, and agricultural activities. Hazardous wastes must be disposed of in hazardous waste landfills. Hazardous waste landfills must have a double liner system with a leachate collection system above the primary composite liner and a leak detection system above the secondary composite liner.

4.4 Liner Components

Clay. To protect the ground water from landfill contaminants, clay liners are constructed as a simple liner, the composite and double liners. Thickness of the compacted clay layers depends on the characteristics of the underlying geology and the type of liner to be installed. The effectiveness of clay liners can be reduced by fractures induced by freeze-thaw cycles, drying out, and the presence of some chemicals.

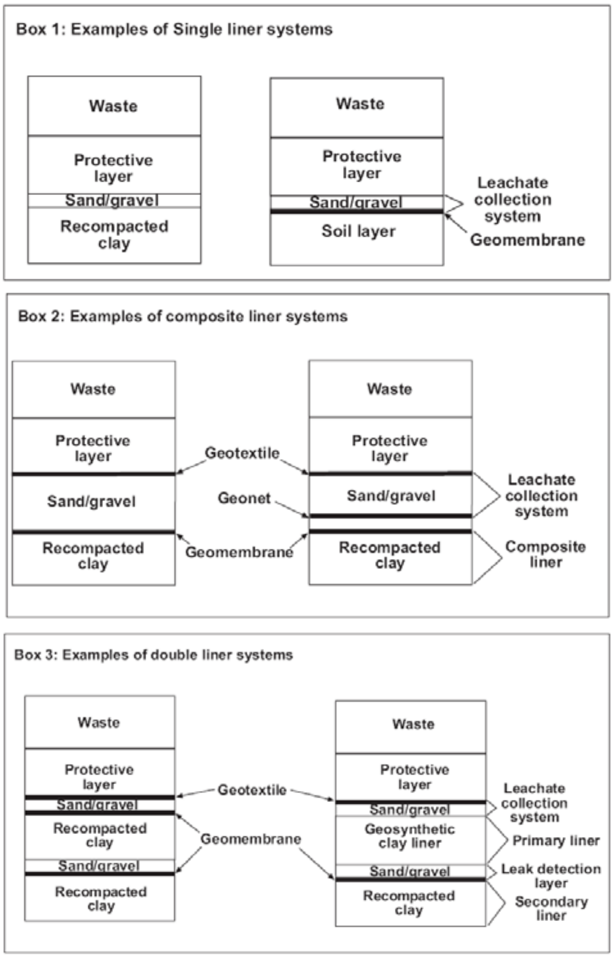

Fig. 2. Examples of liner systems

In theory, 0,5 m of clay is enough to contain the leachate. The reason for the additional clay is to safeguard the environment in the event of some loss of effectiveness in part of the clay layer. The efficiency of clay liners can be maximized by laying the clay down in 2 layers and then compacting each layer with a heavy roller. The efficiency of clay liners is impaired if they are allowed to dry out during placement. Desiccation of the clay during construction results in cracks that reduce the liner efficiency. In addition, clays compacted at low moisture contents are less effective barriers to contaminants than clays compacted at higher moisture contents. Liners that are made of a single type of clay perform better than liners constructed using several different types.

Geomembranes. Geomembranes are also called flexible membrane liners (FML). These liners are constructed from various plastic materials, including polyvinyl chloride (PVC) and high-density polyethylene (HDPE). The preferred material for use in MSW and secure landfills is HDPE. This material is strong, resistant to most chemicals, and is considered to be impermeable to water. Therefore, HDPE minimizes the transfer of leachate from the landfill to the environment. The thickness of geomembranes used in landfill liner construction is regulated by state laws.

Geotextiles. In landfill liners, geotextiles are used to prevent the movement of small soil and refuse particles into the leachate collection layers and to protect geomembranes from punctures. These materials allow the movement of water but trap particles to reduce clogging in the leachate collection system.

Geosynthetic Clay Liners. Geosynthetic clay liners are becoming more common in landfill liner designs. These liners consist of a thin clay layer (four to six millimeters) between two layers of a geotextile. These liners can be installed more quickly than traditional compacted clay liners, and the efficiency of these liners is impacted less by freeze-thaw cycles.

Geonets. A geonet is a plastic net-like drainage blanket, which may be used in landfill liners in place of sand or gravel for the leachate collection layer. Sand and gravel are usually used due to cost considerations, and because geonets are more susceptible to clogging by small particles. This clogging would impair the performance of the leachate collection system. Geonets do, however, convey liquid more rapidly than sand and gravel.

5 Leachate Collection Systems

Integrated into all liner systems is a leachate collection system. This collection system is composed of sand and gravel or a geonet. A geonet is a plastic net-like drainage blanket. In this layer is a series of leachate collection pipes to drain the leachate from the landfill to holding tanks for storage and eventual treatment. In double-liner systems, the upper drainage layer is the leachate collection system, and the lower drainage layer is the leak detection system. The leak detection layer contains a second set of drainage pipes. The presence of leachate in these pipes serves to alert landfill management if the primary liner has a leak. Components of the liner system are protected by a layer that minimizes the potential for materials in the landfill to puncture the liner. This protective layer was traditionally composed of soil, sand, and gravel, but many landfills now use a layer of *soft refuse* instead of soil. Soft refuse consists of paper, organic refuse, shredded tires, and rubber. Individual components of the liner systems are described on the next page.

6 Conclusions

Ground water, infiltrating surface water moving through the solid waste, or moisture contained in the wastes can produce leachate, a solution containing dissolved and suspended solid matter and microbial waste products. The composition of leachate is important in determining its potential effects on the quality of nearby surface water and ground water. While there are many aspects of landfill design that need to be considered, one of the major components that must be addressed is the placement of a liner at the bottom of the landfill. The leachate production and movement, engineering and design may be prevented or minimized to the extent that it will not create a water pollution problem. While many geological, hydrogeological and engineering problems can be minimized through effective landfill designs, a careful sitting process will greatly reduce the risk that may arise during and after operations.

Acknowledgement

The article is supported by the Scientific Grant Agency and the Cultural and Educational Grant Agency of the Ministry of Education – VEGA Project No. 1/0854/08 and KEGA Project No. 3/5125/07 and project „Operational Programme Research and Development – Excellence Centre of Integral Territory Flood Protection (Centrum excelentnosti integrálnej protipovodňovej ochrany územia)", code ITMS 26240120004 dealt with at the Department of Sanitary and Environmental Engineering, Faculty of Civil Engineering, Slovak University of Technology in Bratislava.

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International Symposium on Water Management and Hydraulic Engineering

Ohrid/Macedonia, 1-5 September 2009

Paper: **A47**

Drinking Water Supply in Slovakia

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Abstract. The development of the Slovak public water supplies has fallen behind other European countries. In 2008, approximately 86,3% of the population was connected to public water systems, which supplied them with their drinking water. Paper deals with issues relating to water resources, water utilization, water supply systems. The crucial topics discussed not only by professionals, but also by the general public, include water management and the ongoing process of Water and Sewerage Works (WSWs) privatization. The major objective of the Slovak Republic (SR) as a member of the European Union (EU) includes adopting and implementing EU legislation with respect to the Slovak environment. This is a highly demanding task; in particular for those Slovak authorities and organizations responsible for meeting these requirements. The task is especially formidable since, besides resolving all major current problems, they are also expected to outline the future perspectives of Slovak water management.

Keywords: Groundwater, public water supply, surface water, water protection

1 Introduction

The Slovak Republic (territory 49.014 km^2; population 5,4 million) is situated in the temperate climate zone of the Northern hemisphere with regularly alternating seasons. About 38% of the country is forested. Based on measurements of the average annual air temperature, the warmest part of the country is the area of Štúrovo in the south (10,4^{o}C); while the peaks of the High Tatras in the north, in particular the Lomnický Peak (– 3,7^{o}C), are considered the coldest. The average amount of precipitation is around 760 mm, with the Danubian Lowland being the driest (below 550 mm) and the High Tatras the most humid part of the country (above 2 000 mm).

Since the Slovak Republic is situated on the watershed of the Black Sea (96%) and Baltic Sea (4%), which is the territory where the majority of European rivers rise (excluding the Danube), the regularity of hydrological regime, in particular the high and medium level discharges in the springtime are of great importance (Fig. 1).

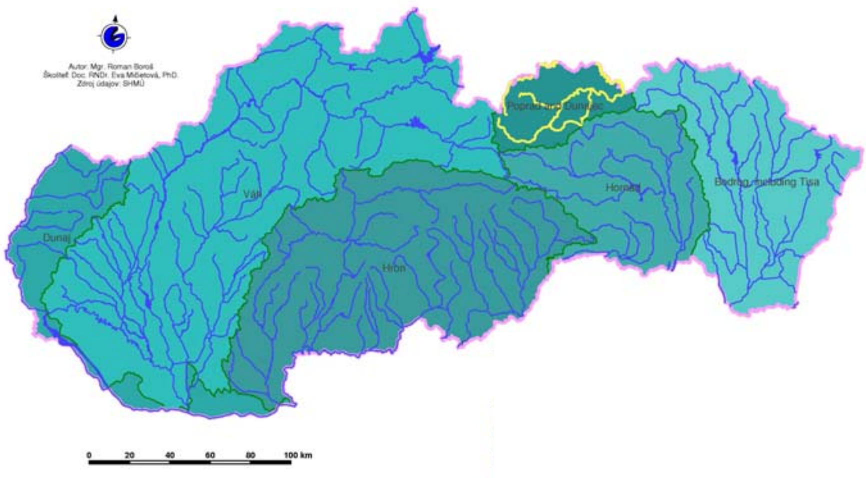

Fig. 1. Slovak river basins

2 Surface Water

At present, there are 54 water reservoirs across Slovakia (with the overall capacity exceeding 1 million m^3) with the gross controllable capacity of 1 890 million m^3. The capacity of these reservoirs allows for the interception of about 14% of the annual mean discharge from our country's territory, as well as the increase of low discharges in dry periods by about 55,5 m^3s^{-1} (above discharge Q355d 80 m^3s^{-1}). Thus the total increased discharge in rivers initiating within the territory of Slovakia reaches approximately 135,5 m^3s^{-1} (Q_{355}d increased by 69%). Water withdrawal, currently amounting to 39,0 m^3s^{-1}, is equal to about 29% of the discharges during dry periods and to 10% of the mean discharge. The water consumption in the territory of SR, varies between 4,8 to 9,0 m^3s^{-1} and decreases during an average year by 1,5 to 2,3%. The previously mentioned water reservoirs include 8 tank reservoirs. Their major purpose is to ensure large-scale drinking water supplies for Northern, Central and Eastern parts of Slovakia.

Data on surface water utilization in Slovakia for 2007 are presented in Table 1, [4].

Table 1. Use of surface water in Slovakia in 2007, in million m^3

Public Supply	Industry	Irrigation	Other agriculture	Total	Discharges
52,884	301,633	15,9	1,354	355,871	1045

3 Groundwater

Ground water in the territory of SR is considered extremely important because it represents the major source of drinking water. Although the hydrogeological conditions of accumulation, circulation and production of ground water are generally favorable, the uneven distribution of ground water resources and the fact that there are some hydrogeological structures practically without useable ground water supplies, represent a disadvantage.

The Eastern Slovak region has significantly lower documented volumes of useable ground water (17% of the total). The remaining 27% represents the resources of the Central Slovak region.

According to the Slovak Hydrometeorology Institute's (SHMI) documentation from December 2006, the natural ground water resources in the territory of Slovakia represent 146,720 m^3s^{-1}, out of which 76,748 $l{\cdot}s^{-1}$, i.e. 52%, are documented usable sources (Ministry of Environment of the Slovak Republic, 2007) [7].

Usable ground water resources are defined as those ground waters that can be withdrawn from the subsurface by technical means, while keeping the natural balance of the environment (Ecological limits).

The established utilizable volumes of ground water are broken down into two basic categories [4, 5].

- Sources and supplies approved by the Committee for Available Groundwater Quantity Classification (AGQC). They are classified into categories A, B, C, C1 and C2 according to the degree of the quality of verification and the level of knowledge of supplies and classification principles (Slovak Technical Standard (STS) 75 7221) [3]. In 2006 the individual categories of the usable ground water supplies were: category A 851,0 $l{\cdot}s^{-1}$, category B 2.126,6 $l{\cdot}s^{-1}$, category C 2.877,9 $l{\cdot}s^{-1}$, category C1 26.584,1 $l{\cdot}s^{-1}$, category C2 12.969,4 $l{\cdot}s^{-1}$and total are 45.409,0 $l{\cdot}s^{-1}$.

- Categories A and B include ground water sources already used as drinking water supplies; categories C1 and C2 include sources ready to be used as drinking water supplies after water treatment.

- Sources and supplies of ground water that have not been approved by AGQC. Data on their volume are documented and broken down into three categories: I, II, III, according to the degree to which they were studied and the reliability of gathered data. In 2006, the amount of useable ground water resources not approved by AGQC was: 31.390,3 $l{\cdot}s^{-1}$. Data on utilization of groundwater in Slovakia in 2006 are presented in Table 2.

Table 2. Use of groundwater in Slovakia in 2007 in [$l{\cdot}s^{-1}$]

Public Supply	Food Produc tion	Other Industry	Agriculture and Live-stock Farm-ing	Cultivation of Plants	Commu-nity Pur-poses	Other	Total Use
8836,1	295,6	852,3	275,8	95	340,2	970,2	11665,2

Source: SHMI

4 Public Water Supply

The total number of inhabitants supplied with drinking water from public water supply network increased in 2006 compared to the previous year by 59,3 thousand to 4.653,4 thousand inhabitants that is 86,3% out of the total number of population of the Slovak Republic. Increase of supplied inhabitants in 2006 represented 1.0 of percentage point. The level of public water supply network development is regionally unbalanced. The largest proportion of supplied population is in the Bratislava Region and higher proportion than the total average is also in Trenčín, Žilina and Trnava Region. The public water supply network development in Banská Bystrica, Košice and Prešov regions falls behind the total average. Much more differentiated condition concerning drinking water supply occurs in particular districts where the proportion of supplied inhabitants is moving up from approximately 60% (Vranov nad Topľou, Sabinov, Bytča, Košice – environs) up to the saturation level (Bratislava, Prievidza, Martin, Banská Bystrica, Partizánske, etc.).

Development of the total number of inhabitants and the number of inhabitants supplied with drinking water is in Table 3.

Table 3. Total population and population supplied with drinking water from the public water supply systems

	1995	1996	1997	1998	1999
Total (number of) population in thousands	5.363,7	5.373,9	5.383,2	5.394,4	5.395,3
Population supplied by WSW, thousands	4.256,8	4.290,4	4.351,6	4.410,1	4.447,8
Proportion (%)	79,4	79,8	80,8	81,8	82,4
	2000	2003	2004	2005	2006
Total (number of) population in thousands	5.400,6	5.380,1	5.384,8	5.386,7	5.390,4
Population supplied by WSW, thousands	4.479,2	4.535,1	4.569,1	4.594,1	4.653,4
Proportion (%)	82,9	84,3	84,9	85,3	86,3

Source: Water Research Institute

In the facilities of water companies, local authorities and other subjects there was produced 334,3 million m^3 of drinking water in 2006 which means the decrease by 3,7 million m^3 compared to 2005. The presented data in Table 4 indicate that the decrease of invoiced water continued – in 2006 it decreased by 7 million m^3 in total. Amount of invoiced water represented 66,5% out of the total amount of water intended for supply.

Since 1990, water consumption in Slovakia has been constantly decreasing. This phenomenon is caused partially by decreasing industrial production, but also by the gradual decrease in public water consumption due to increasing drinking water prices. The specific demand for drinking water has decreased since 1990 by almost 45%; the current consumption value is 89,5 $l \cdot person^{-1} day^{-1}$ [2]. Compared to EU countries, which consume approximately 145,0 $l \cdot person^{-1}.day^{-1}$, the specific water demand in Slovakia decreases year after year. It is alarming situation mostly because the high costs for drinking water lead to construction of own sources of drinking water with the quality far behind the hygienic standards.

The consumption of drinking water is falling deep below the average, leaving us dangerously close to approaching the minimal hygienic standards. This trend also applies to other water use categories, since the demand of industrial and other users – compared to 1990 – has decreased by 30, 2%.

Based on analyses, the specific household water demand should reach the bottom line (80,0 $l·person^{-1}·day^{-1}$) by 2010; and consequently, the overall water use is predicted to decrease to 165 $l·person^{-1}.day^{-1}$. The average specific water demand for households in 2030 is predicted to reach 145 $l·person^{-1}·day^{-1}$; and the overall specific water demand 220 $l·person^{-1}·day^{-1}$. The overall volume of non-metered water has decreased to 114,2 million m^3 - that is 25% of the total water production; more than 83% of that represents losses in the distribution network (27,2% of produced water).

Measures should be adopted and taken to decrease the losses in the piping system to an acceptable rate, corresponding to European standards.

Construction of public water supply network led to an increase in number of technical facilities and structures. Compared to 2005 the total length of water supply systems in Slovakia (water companies, local authorities and other subjects) increased by 638 km up to the total length 26,357 km. Data on water supply and water supply network development are listed in Table 4.

Table 4. Water delivery and trends in development of water supply systems in WSW administration

No.	Indicator	Unit	Year					
			1999	2004	2005	2006	2007	2008 exp.
1	Population supplied by water supply systems (WSS)	Thousand	4.014,4	4.569,1	4.594,1	4.653,4	4.690	4.720
2	Surface water resource capacity	$l.s^{-1}$	29.308	33.855	33.848	33.546	33.500	33.500
3	Distribution network length	km	22.950	25.313	25.719	26.357	26.400	26.700
4	Ground water resource capacity	$l.s^{-1}$	24.396	28.413	27.921	27.713	27.650	27.650
5	Water produced in WSW facilities	$mil.m^3$	402,5	353,2	338,0	334,3	327,4	324,8
	from ground water	$mil.m^3$	336,0	298,6	287,6	280,6	278,0	275,0
6	Total water produced	$mil.m^3$	402,8	356,6	343,1	339,2	331,6	329,1
7	Metered water - total	$mil.m^3$	286,5	241,6	232,0	225,0	224,8	224,1
	- households	$mil.m^3$	185,9	166,2	159,1	152,1	152,2	152,0
8	Non-metered water	$mil.m^3$	116,3	115,0	111,1	114,2	106,8	105,0
	of which: losses in the piping network	$mil.m^3$	96,8	93,2	93,2	94,2	91,9	90,0
9	Specific water consumption	$l.per.day-^1$	126,9	99,7	94,9	89,5	88,9	88,2

Source: Water Research Institute

By the Slovak constitution streams, ground water, mineral water, and natural curative springs are in the ownership of the Slovak Republic. Therefore, in harmony with the needs of society and with the principles of the European Water Chart, the more intensive water utilization and the higher the level of "civilization" of society, the more responsibly and diligently the waters have to be protected, regulated and replenished. Therefore, it is of primary importance that both the surface water and ground water are administered by organizations fully ensuring the direct application of state interests, in particular in the rational utilization of water, water protection and the fair distribution of these irreplaceable natural resources.

The Slovak state water management policy for the future has to be based on the state water management objectives, which include in legislations of Ministry of Soil Management of the SR, Ministry of Environment of the SR and Ministry of Health of the SR, which include in [1]: (i) providing sufficient volume and quality of drinking water for inhabitants and all consumers; (ii) overall treatment of the volume of utilized and polluted water before discharging it back to its natural environment; (iii) achieving a high level of environmental protection within the context of sustainable development; (iv) achieving an adequate level of flood prevention in residential and industrial territorial units and areas of transport infrastructure and intensive agricultural production; (v) achieving of a water resources and facilities protecting against damages caused by drought.

5 Conclusions

The ultimate goal of Slovakia is to reach level in water supplying and connection to sewerage in well developed countries in the European Union. This goes hand in hand with assuming the responsibilities relating to the harmonization of water management policy with that set by the European Union legislation, which is enforced as an inseparable part of the right to a healthy and clean environment.

Acknowledgement

The article is supported by the Scientific Grant Agency and the Cultural and Educational Grant Agency of the Ministry of Education – VEGA Project No. 1/0854/08 and KEGA Project No. 3/5125/07 and project „Operational Programme Research and Development – Excellence Centre of Integral Territory Flood Protection (Centrum excelentnosti integrálnej protipovodňovej ochrany územia)", code ITMS 26240120004 dealt with at the Department of Sanitary and Environmental Engineering, Faculty of Civil Engineering, Slovak University of Technology in Bratislava.

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A53**

Rural Waterworks - Legal and Technical Aspects

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Abstract In the times of former Yugoslavia, as well as in the new Republic of Croatia, a large number of so-called rural waterworks were built. From the legal point of view, such waterworks are illegal, technically they do not meet the standards, and often even the water quality does not meet the prescribed standards either. For the recent 15 years the Republic of Croatia has invested considerable funds in water supply in small communities, including the locations where rural waterworks exist. The paper analyzes the legal, social-psychological and technical aspects regarding the present situation, and the solutions that might be applied to preserve some of the rural waterworks which could, with essential reconstruction, meet the required conditions.

Keywords: Water supplying, rural waterworks, technical standards, legal status

1 Introduction

For the purpose of this paper, the expression ¨rural waterworks¨ shall mean a water supply system supplying more than two households, built by the users and lacking the legal and technical properties of a public water supply system.

In the past, construction of modern water supply systems in small communities was not a priority at the national level, mostly due to acute shortage of funds, and the local population, wanting good quality and secure water supply, used to build rural waterworks with their own forces. Such practice was never prevented, although it meant breaking of a number of laws and regulations. The actual policy even encouraged such projects, sometimes providing financial assistance, but in most cases ignoring and tolerating obvious illegal practice.

The number of such waterworks is large. However, their history and status is not equal. There are a number of water supply systems which have been integrated into legal public water supply systems, but as regards the large majority of others, nothing is happening.

The national water management agency *Hrvatske vode* has commissioned the study "Analysis of small water supply systems on the territory of the Republic of Croatia not included in public water supply systems" [1]. The study registered all waterworks supplying more than 50 users, or delivering more than 10 m^3 per day. Table 1 shows the number of rural waterworks, the number of intakes, daily delivered water quantities and the number of users in each county. The number of rural waterworks is higher in the continental parts of the country where the coverage by public water supply is below the average.

In the Republic of Croatia, public water supply services are available to 84 percent of the population, and Table 1 shows that about 6 percent of the entire population is served from 443 rural waterworks, and the remaining 12 percent are supplied from even minor rural waterworks or on individual basis.

Table 1. Analysis of waterworks in separate counties [1]

County	Number of rural waterworks	Number of intakes	Estimate delivery m^3/day	Number of users
Zagrebacka	54	92	2583	12.251
Krapinsko -zagorska	112	132	5265	39.060
Sisačko – moslavacka	12	19	200	1.330
Karlovacka	22	29	466	2.515
Varazdinska	80	99	3372	18.440
Koprivnicko - krizevacka	8	8	2180	2.693
Bjelovarsko -bilogorska	8	12	1190	13.350
Primorsko -goranska	19	49	17571	41.374
Licko - senjska	6	6	622	2.936
Viroviticko – podravska	12	56	1805	10.750
Pozesko - slavonska	6	6	90	983
Brodsko - posavska	5	8	1090	7.453
Zadarska	2	6	400	2.750
Osjecko - baranjska	29	33	4970	44.469
Sibensko - kninska	4	4	1150	1.570
Vukovarsko - srijemska	28	26	5861	46.886
Splitsko - dalmatinska	1	1	-	3.000
Istarska	6	6	70	440
Dubrovačko-neretvanska	2	2	90	725
Medjimurska	2	2	240	1.461
City of Zagreb	25	36	1670	10.400
TOTAL	443	632	50.885	264.836

Water quality analyses on 538 samples show that even 75% do not meet the standards of the Regulations on sanitary quality of drinking water [1]. Out of this, the major part does not meet the microbiological standards (86%), and 27% of the samples do not meet the physical and chemical standards [2].

The intake of the rural waterworks is usually on the descending spring with a small recharging area and a shallow aquifer, and is therefore susceptible to large yield variations (some even running dry in summer) and pollution (in particular bacteriological). The intake is usually situated on private property, poorly protected or not protected at all. The facilities, pumping stations, reservoirs, pipe networks and other structures and installations were built, in most cases, without design, by improvisation, and following the minimum costs criterion. Pressures in the network are far from uniform, and the capacities do not meet the fire-fighting requirements [3].

Few rural waterworks are equipped with automatic disinfection devices; disinfection is carried out manually, from time to time, and the analyses record periods with high residual chlorine, followed by high concentrations of bacteria [1, 3].

Generally, the technical condition of the waterworks is poor, as well as maintenance and protection of water intakes, reservoirs and pumping stations. Rural waterworks are managed by the users through informal individuals or formal water boards. Most repairs and other works are done by the users themselves, and the water price formed in accordance with current costs is regularly considerably lower than standard tariffs in public water supply. Some rural waterworks have agreements with public utilities (or with municipalities owning such utilities), regarding assistance in periods of water shortage, either by buying water from the utilities (connecting to the water supply network), or by transporting of water to the consumers by tank trucks when the sources run dry [3].

Fig.1. Typical water intake of a rural waterworks (Hum na Sutli, 2008.)

On average, the users of rural waterworks are poor village people living in remote hamlets, far from the influence of larger urban centers. Their number decreases with time, and the age structure and economic status is deteriorating. Due to poor economic status and long years of usage they are not ready to accept changes that are sometimes offered to them through construction of large regional water supply systems.

2 Legal Aspects

According to the Water Act, each person is allowed to use water on his property for his personal use free of charge [4]. Personal use implies using water for drinking, sanitary purposes and similar uses in the household. This does not include water use for irrigation or in the technological process of industrial activities.

If water supply is extended to users beyond the household, this is considered providing of services and treated, in principle, as an activity requiring a number of permits or establishing a utility.

It comes out from the above and from the fact that rural waterworks cover the needs of a major number of households, and none of the waterworks being properly registered according to the existing laws, that all rural waterworks supplying two or more households are illegal and that the services they provide to the users are unauthorized.

Performing of water supply services requires meeting of certain criteria. According to the Water Act: "*The use of water from springs and ground water may be approved only following previous water research works. Water research works are the works and investigations carried out to define the existence, extent, quantity, quality and mobility of ground water resources in a given area*" [4].

Water intakes must be protected by the decision on sanitary protection zones protecting the intakes from intentional or accidental pollution and from other impacts that may be detrimental to sanitary quality of water, or to its yield [4].

Rural waterworks have no concession agreements, and "concession gives the right of use of water and public water estate, or the right to perform industrial and other activities on water and public water estate. Water supply activities are performed by legal entities organized in accordance with the law regulating municipal utilities." [4].

It may be concluded from the above analysis that rural waterworks do not meet any of the key criteria defined by the law, and that they practically represent illegal facilities. It is a paradox that national and local authorities have not reacted to this situation for years. Some of the rural waterworks have been included into the physical planning documents.

In this way the rural waterworks have practically got the right to exist and as such, have entered the mind of the users as inherited property and right.

3 Psychological and Social Aspects

Rural waterworks are not a sporadic phenomenon that may be regarded as incidental. They are the result of the systematic policy of a given period, which must be stopped. The problem could be solved in a rather simple manner by strict enforcement of present laws, but in practice this would be impossible without affecting the social sensitivity towards the users who have, during many years of use, always considered the waterworks as their property, built with their own money.

Confronted with the possibility of connecting to the public water supply system after a number of years, the users of rural waterworks mostly react as follows [3]:

- they refuse to be connected, because the new connection requires investing of a considerable sum of money;
- they do not want to pay considerably higher tariffs for the water supply service than at present;
- they do not see any reason to get connected, being satisfied with status quo;
- many of them argue that water they are using has better organoleptic properties than water from the public water supply system.

Encountered by notions on enforcement of the law, or closing down of "illegal" installations and intakes, they resist and announce radical measures.

Among all reasons given above, which are logical reactions of the users, the most important ones are the price of the connection and the water tariff. With respect to average poor economic potential and high unemployment rate, comparatively small savings may be considered important in the household budgets that lack the funds for other elementary requirements as well.

The problem is how to persuade the users to accept the fact that so far they have been using water illegally and that in future they will, through the care of the society, receive water that is more expensive and, according to their opinion, of doubtful quality.

Obviously, the process must be initiated following the policy of small steps that include education, law application and small concessions to the users, first of all, of economic nature.

The basic point is to inform the users and the public about health hazards and about large risks taken by non-professionals in such an important activity. Within the profession it is necessary to find the optimum solution for each case which will, in additional to technical, also include specific measures to be provided by the politicians, through issuing of corresponding directives and regulations.

4 Conditions for Preservation of Rural Waterworks

Among numerous rural waterworks there certainly are realistic conditions to keep a large number of them to maintain their basic function. The chances are on the side of the rural waterworks that:

- throughout the year have available sufficient water of required quality, or the quality may be achieved by inexpensive treatment;
- have the water intake on public property land;
- have good natural and other conditions for well field protection;
- have comparatively good water supply network and facilities;
- that can meet the required criteria with certain investments, the solution being financially more favorable than bringing water from external network;
- whose users are willing to cede their waterworks to authorized public utilities for management.

The most important among many legal shortcomings is the absence of sanitary responsibility, which must not be left to unqualified individuals or groups. In this sense, there are no alternative solutions. Regardless of the final solution, the first step must be to subject the rural waterworks to supervision by municipal utilities or other firms authorized for the job.

It would be a bad solution for water supply of some communities having rural waterworks to spend large sums from the budget or other sources for water supply projects that are very costly, while there is a realistic possibility to integrate rural waterworks into the category of public water supply systems. Such mistakes have already been made, mainly because there is still no strategy of solving the problem, nor the concept of application of nonconvential small waterworks.

4.1 Ownership Issues

In all cases when the installations and facilities of rural waterworks are situated on land which is the property of the state or the local community, there should be no problems regarding ownership. Where the owners are private persons, the possibilities are buying up of land or long-term agreements of free use or renting of land. All existing installations which will be retained should become the fixed assets of the owner of the waterworks. The owner may be a public utility, or its founder, or a legal entity representing the collective property of the local community. This issue requires passing of special bye-laws.

4.2 Zones of Sanitary Protection

In general, in all rural waterworks there are no defined zones of sanitary protection as required by the law, and subsequent defining of such zones is practically impossible. On one hand, the reason is high costs of research works, and on the other hand the already defined purpose of the land. Further limitation of the purpose of the land in possible zones of sanitary protection would cause resistance of private land owners, in particular if it is agricultural land or land assigned for construction.

It is necessary to define quite precisely the first zones of sanitary protection (areas directly around the water intake), while for the other parts good management must be provided, based on permanent education of the local population.

The risk of failing to define the zones according to the existing regulations is based on historical knowledge on actual works and on regular control of the quality of abstracted water.

4.3 Water Tariff

Statistically, most of the rural waterworks were built in the least economically developed parts of the country. Due to the generally accepted policy of conservation of the village and village population, some parts of the country are even now classified as areas of particular national concern. In this context, the government and the local community should waive the charges paid by towns and more developed parts of the country. This would make the difference of tariffs between the existing rural waterworks and those subsequently legalized acceptable, instead of several times higher, which is the main reason why the local population refuses to hand over their waterworks for management to the local utility.

4.4 Fire Fighting

With regard to the way of living, house construction and sizes of the house lots, it is possible to prove that there are good reasons not to apply present regulations regarding fire fighting from hydrant networks on rural households. Required high pressures on hydrants, large discharges and water quantities for firefighting are the reasons of very high costs of construction of water supply in remote communities. The requirements include minimum water reservoirs volumes of 100 m^3, minimum pipe diameters DN100 mm, minimum pressure in the network of 2,5 bar, and flow capacities above 10 l/s. Such requirements also influence high operation and maintenance costs and are detrimental to water quality, bringing potential hazard of subsequent pollution.

In small communities with freestanding structures, the regulations should define considerably less stringent fire-fighting conditions, relying on firefighting vehicles and possible water or rainwater tanks where vehicles could be replenished.

4.5 Securing of Water Quality

A large number of water intakes in rural waterworks do not respond to drinking water quality criteria according to the present Regulations [2], either constantly or from time to time.

In most cases it is the matter of overstepping the permissible level of bacteriological pollution in comparatively shallow aquifers, and sporadic turbidity. Water disinfection at the intake nowadays does not present a technologically demanding or a costly solution. There is the possibility of incorporating automatic chlorinators or devices for UV disinfection. There are also a number of small capacity devices for removal of iron, manganese, turbidity, suspensions, smell, color… [5].

In some cases, a feasible solution might be the use of double installations, where water for drinking and cooking would be treated by some advanced technology (reverse osmosis, multimedia filters) and transported by one installation, and rain water or some other suitable water would be used for technological purposes.

Modern devices operate mainly automatically, but professional supervision is essential. This function might be taken over by the utility or by some other authorized firm.

In addition to professional guidance and equipment, it is necessary to provide regular control of sanitary quality of water, with the possibility of enforcing of necessary measures, from fines to closing down.

5 Conclusions

The present status of rural waterworks is unsustainable. In addition to constant infringement of basic provisions of the law, there is a constant threat to the users' health.

It is bad practice to construct new public water supply systems in areas where rural waterworks prevail, without simultaneously solving their status. The result is the lack of interest in connecting to new expensive public water supply systems.

After getting the insight into the situation [1], a comprehensive policy is needed to pass additional sub-acts which will deal with small water supply systems in accordance with their status and with the requirements of their respective users, and require the professionals to prepare principled technical criteria for their improvement in the framework of the individual or the integrated water supply system. In this context, each individual case should be analyzed within the given technical and legal framework, and the optimum solution should be offered. The solution, due to clearly difficult political and social-psychological background, should not be imposed from above; a bottom-up process is required which will win over the end users by the policy of small steps, education and necessary material alleviation measures.

Improvement of the status of water supply in rural waterworks is also the opportunity to define, on the national level, the small waterworks system in the legal and technical-technological sense. Such systems, with individual solutions, should be the answer for those areas where conventional network water supply is not the optimum solution.

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A57**

Ohrid Water Supply System–History, Features and Development

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Abstract. The town of Ohrid is in the south-west of Macedonia, on the shore of Lake Ohrid. There has been organized water supply for more than 70 years. The town development has contributed to the development of the water supply system, particularly since Ohrid has become a tourist centre of Macedonia. With realization of various Projects, Ohrid Water Supply System has developed to provide regular water supply for more than 50 000 inhabitants. Ohrid Water Supply System consists of various types of spring with changeable capacity, pipelines of different kind of materials, reservoirs and other facilities, on one hand, and on the other hand, the inconsistent consumption that hugely varies during the seasons (winter-summer), in the range 1:3. This makes Ohrid Water Supply system very particular. Apart from the measures taken for regular water supply, measures are also taken for regular monitoring of water production and consumption, all in order to detect and reduce the water losses.

Keywords: Water supply system, water springs, variations in water production

1 Introduction

Water and water supply have always been and will be a priority for human's life. The human being has always strived to use the water for his needs, because life can not exist without the priceless treasure called water.

The town of Ohrid is on the shore of Lake Ohrid. It is a town with tremendous cultural inheritance, which is mentioned even in ancient history, and today it is the main cultural and tourist centre in Macedonia.

Regarding the water supply, it is specific because many tourists come during the summer, so the total demand for water doubles and at certain weekends increases threefold. On the other hand, the yield of the springs during the summer period is reduced, which requires involving all available capacities of water sources. Besides all specific features and problems, Ohrid is one of the most regular supplied towns in Macedonia.

2 History

On the territory of Ohrid, according to the stories by older citizens, during the time of the Turkish Empire there was a system of pipelines - ceramic pipes, capturing the water from the springs St.Petka and Ponik and distributed it to the sarais and several taps in the town. After that, the water supply was done with wells of from the lake, but all that was individual and not well organized.

The year 1934 is considered as a start of organized water supply in Ohrid, when the water supply system was constructed, consisting of the Captage Biljanini Izvori, a pipeline to the reservoir Kale and the reservoir Kale.

The water supply lines along Dimitar Vlahov Street, Klimentska, Ilindendska, part of 7-mi Noemvri and the old Bazaar also date from this period. Pipes from cast iron were included then and these pipes are used today, as well.

Fig.1. Captage Biljanini Izvori of 1934

Fig. 2. Reservoir Kale of 1934

With the development of the town and the industrial facilities, the water supply system was developed further on. The economic and tourist development of the town was followed by the development of the water supply system. The shortage of water that was present made finding new water springs necessary.

In 1969, Letnica Springs and Biglishta were captaged, the spring Ponik in 1972. A pipeline to the town was constructed which distributed in average 60l/sec in the town. (Letnica springs are carst springs and their yield varies between 40-120 l/sec).

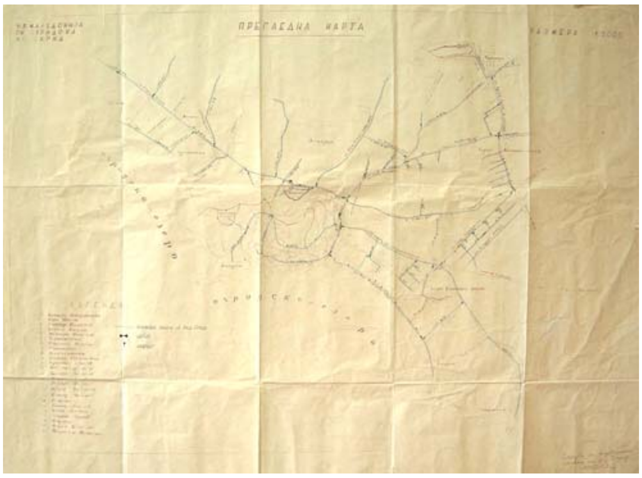

Fig. 3. Water Supply Network in Ohrid 1965-19 66

The town of Ohrid continued developing and slowly it became a tourist and industrial centre. For that purpose, new water quantities were required and in 1978-79 the wells Studenchishta were drilled, with capacity of 2x50 l/sec.

The specific tourism features of the town contributed to search for new water sources that will meet the demands in the summer period. Lake Ohrid was chosen as a source and in 1988 the water supply system from the pump station Metropol was launched, with capacity of 200l/sec, pressure steel pipeline Φ 711 to the reservoir Studenchishta and the reservoir Studenchishta of 3500 м3. In order to remove the plankton in 1998 , microsita Filtomati was placed with holes of 15 microns.

In parallel with finding new spring, the water supply network was extended and water supply rings were formed.

During the last years, the water supply system in Ohrid was reconstructed under the MEAP project, particularly Letnica springs and distribution pipeline, two new wells were drilled in Orman, with the capacity of 2x40 l/sec, two low level reservoirs were constructed with total volume of 2000 m3, two high level reservoirs with total volume of 200 m3, as well as reconstruction and new water supply lines in the network.

3 Water Supply System Ohrid - Present State

Today the water supply system Ohrid provides central water supply for the town Ohrid, the settlements Orman, Racha, Sv.Stefan, Dolno Konsko, Lagadin, Peshtani and the tourist sites along the shore on the eastern part of the lake. The settlements Velgoshti, Trpejca, Ljubanishta and Leskoec are supplied with water from local springs which are given over by the Municipality of Ohrid for management to MJP Proaqua - WU Vodovod Ohrid. The water supply System Ohrid is a complex system, consisted of different types of water springs, as well as several reservoirs located on different height.

The total number of people supplied is 53000, and during the summer period, because of the tourism character of Ohrid and the surrounding, this number rapidly rises and at some weekend it reaches a total number of customers of 120 000.

3.1. Water Springs

Water in Ohrid water supply system is provided from the following:

1. Gravity springs

- springs on gravity (Letnica, Biglishta, Ponik) with average yield of q average/year = 64 l/s (40-140 l/s)

2. Springs with pumping

- with pumping from the wells Studenchishta q_{inst}=115l/s (2x50 l/s and 1x15 l/s new well Bejbunar);
- with pumping from Biljanini Izvori q_{inst}=75 l/s (2x30 l/s and 1x15 l/s);
- with pumping from Orman wells q_{inst}=80 l/sec (2x40 l/s);
- with pumping from Lake Ohrid (only in summer) q_{inst}=200 l/s;
- from local springs for the village Trpejca, only with pumping $q_{average}$ = 4 l/s;
- from local springs for the village Ljubanishta (with gravity $q_{average}$=2l/s and with pumping $q_{average}$ =5 l/s;
- from local springs for the village Leskoec $q_{average}$ = 4l/s.

3.2. Water Reservoirs

In order to balance the hourly inequity in the central water supply system for Ohrid and the local systems, reservoirs have been constructed, with total volume of 7260 m^3.

3.3 Pipelines

Pipelines in the water supply system in Ohrid are made of different type of materials (cast iron 5%, steel 5%, asbestos cement 30%, PVC 36%, PE 14% and zinced 10%), and the total length of the network in the system is approximately 167km.

The pipelines diameter in the water supply network is with profile 2’’to 711mm.

3.4 Water Production

One of the basic preconditions for establishing an optimal water supply is to know the water production. For the purpose, seven flow meters have been installed at the water springs in the central Ohrid water supply system. During the past two years, the data is regularly read once a month and during the last period, we have established a procedure for data reading on a daily basis, as well.

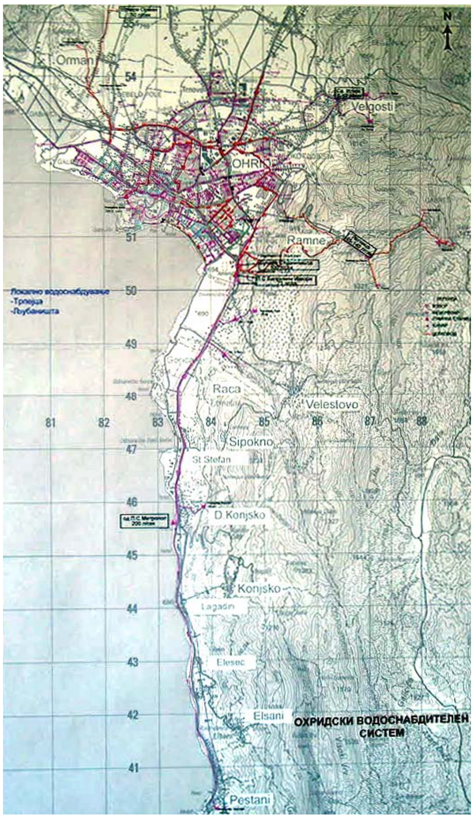

Fig. 4. Ohrid water supply system - today

Obtained data is analyzed and we obtain the results for the monthly disbalance of water production. Thus, the average monthly production in August 2008, as the maximum, compared to the average monthly production in February 2008 as the minimum is in the ratio 2,06:1, and in 2007, the maximum versus the minimum production was in the ration 2,66:1. If we analyze the daily disbalances of the production, this ration varies to 3,5 (4) :1 .

This data is compared with the invoiced amount of consumed water. Since this rate is only a small segment of the water balance (according to IWA) of a water supply system, we are not going to present it here.

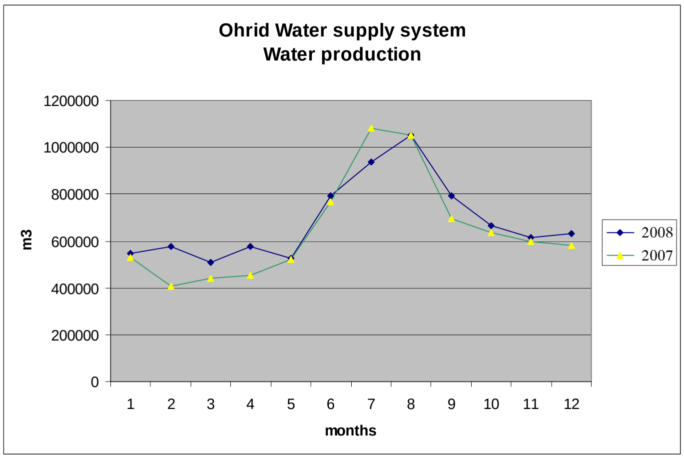

Fig. 5. Water consumption in Ohrid water supply system

While managing a water supply system with huge variations in water production, it is very important to switch on the capacities successively, rationally. Regulation of switching of the individual water springs depends on how full the central water reservoir is. It is particularly characteristic switching on the pump station Metropol. Its peak capacity is in reserve until the summer, when it has to be included, in order to obtain regular and continuous water supply.

4 Conclusions

One of the most important things for each water supply system is to control the excessive water consumption. That means establishing optimal water supply which will provide regular and continuous water supply with fewest controlled water losses.

The next steps to be taken in Ohrid water supply system are as follows:

- replacement of pipelines with small diameters and remedy of bottle necks in the water supply,
- establishment and analysis of pilot areas, where all water losses will be analyzed according to the international standards of IWA ,
- installation of flow meters throughout the water supply network
- establishment of a hydraulic model of the water supply system and introduction of a system with remote control of the entire water supply system,
- finding new gravity springs that will contribute to economical water supply

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A61**

Implementation of Trench-less Methods in Slovak Conditions

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Abstract. The trench-less method in Slovakia expanded after 1989, than foreign technologies allowed to start the new trends, with the new technologies exploitation. So this was milestone. This time together with the world technologies improving could have a significant influence on trench-less systems exploitation. The pipeline condition appeals the reconstruction and rehabilitation, because the Slovak pipelines achieved the lifetime limit. The decision for the trench-less methods using depends on: technical aspects (under streets, crossways, rivers and existing structures), impossibilities of breaking the lively places, environmental aspects (noise, dustiness, traveled city places) cost conditions. Exploitation of trench-less reconstructions is used mostly in high-density urban areas, downtowns, where other methods are useless. The paper presents the actual state of trench-less methods in Slovak republic depends on technical and economical decisions together with environmental. Compare the classical methods with the trench-less.

Keywords: Sewer, reconstruction, rehabilitation, trenchless method

1 Introduction

The new trench-less technologies started in interwar period in world scope, but in Europe in 80th years of last century. These methods were initiated by the camera inspection technologies. The first focus was the sewerage system, considering the pipe diameters and chance of accessibility. The first methods were micro-tunneling, controlled drilling and after the special methods developing. The first Slovak trench-less methods were covering by foreign companies. The present state cover many companies with the various trench-less systems such as Aquadefekt, Ekoline, Inkanal, Maincor, Rehau, Wavin, Zepris etc. The approach of the costumers, which should be use the trench-less methods for theirs pipeline renovation, is very conservative. They used very often the classical verified methods for rehabilitation, and only the complicated, classically impossible reparation, make them to use the new trench-less methods. The unnegligible factor is the costs for the reparation.

2 The Reasons of the Rehabilitation

The pipeline systems build in Slovakia in the last century are in very poor conditions. The construction of pipeline system was very fast in the most of the regional cities. The occurrence was not takes in this era to the quality of these systems, but the priority was only quantity. The expansion of the cities in 50-70 years of last century caused the construction many kilometers of pipes. The fast construction with the non professional approach in this period causes, that the sewer were constructed with very poor quality. There were many problems, to keep the slope concerning the sewers, to make a waterproof weld connection of the pipes, to keep the conditions of firm up over the pipe.

Many pipes, especially sewers were broken during the building, and insufficient of supervision caused the poor quality of execution. The mind of the people, which constructed the sewers, mainly workman, was at low level, and they hoped, that it is impossible to reveal the bad quality of execution.

The new modern approach of appraisal of sewer systems shows with using the TV systems, that the actual conditions of the majority of sewer systems in Slovak cities are in deplorability. The first appraisal of the sewer system by the TV camera started approx. in 90 years of last century.

The advantage against the classic technologies are the minimal intrusion of residents; less probability of the other pipes and cables damaging; less violation of the environment (tree roots, streams); considerable reduction of the time reparation. The non-invasive system for the rehabilitation simplifying the legislative process for the execution, and it has goodly influence for the investor decision.

Historical background. The historical development of sewer system was consists from systematic and unsystematic approach. The first sewer systems were constructed with the goal to dewater small parts of urban areas from waste waters- from inhabitants and dewatering the waters from rain. It was the base for the systematic sewer system, which started as the connections of non-systematic sewer systems. The first goal was easy - dewatering the waste waters directly to the nearest recipient, but this process had to be change, because the contamination of recipient causes the other hygienical and sanitary problems. The one of interesting systematic sewer disposal systems was "Heidelberg barrel system" or the "Kiel exchange bucket system" in Germany. The problem in sewer history was to liquidation the pollution in sewer system. The resolution was WWTP – Waste Water Treatment Plants. We should say, that the actual sewer systems are now actual, but come from last era, from last century. This is the typical for Slovakia. The all sewer systems in Slovak cities were built mainly after the second of world war 1945, but the large cities, capitol of Slovakia Bratislava, the relative big cities in Slovakia Košice and Banská Bystrica has sewer system build before the 2nd of world war. That's why the majority of actual sewer in Slovakia immediately needs reconstruction respectively rehabilitation.

3 The Way of Reconstruction and Rehabilitation Ordinary Using in Slovakia

The Slovakia becomes the country with high level technique standard. No only computers exploitation, but other significant techniques, too. The development in reconstruction and rehabilitation of pipelines we divide into 3 basic groups. The renovation of water pipelines, sewerages, gas pipelines. These 3 fields of the same scope require various technical procedures for achieving the suitable goal. The main divide is base on the type of liquid, which is transfer by pipe. The water pipelines used relatively small diameters for transfer, but this pipe is under pressure, which achieve maximum pressure 0,6 MPa. The similar pipe is gas pipeline, but the difference is in the pressure in pipeline, and the very important is the chance of explosive. We divide the 4 types by the pressure: low-pressure pipes to the 5kPa, moderated-pressure from 5kPa to 0,4 MPa and high-pressure pipes 0,4 – 4 Mpa, very high pressure system 4-10MPa. Base on this dividing, the chance of explosive grows, the reconstruction and rehabilitation becomes complicated. On the other hand the reconstruction of the sewer pipes is significant mainly with the huge diameters of the pipes and the relatively the dept of the sewer location. Sewers are located under all other engineering networks.

The approach of the reconstruction include next systematic steps: developing the preliminary plan of reconstruction, the monitoring of the present state, development of the problems identification system – rehabilitation strategy plan, reconstruction or rehabilitation.

Considering on the various technologies, the competitors fight is not only between contractors, but unwanted and technologically unmotivated fight of trench-less technologies, too. We can divide the exploitation of trench-less technologies as unique in next cases:

- when there is not possibilities from technical aspect use different alternative (realization under streets, crossroads, rivers, existing structures, etc.);
- in some cases, when the excavation can invoke the unwanted marginal impacts (extensive traffic restrictions, restrictions of pedestrian movements, abnormal noisiness or dustiness);
- unwanted changes mainly in travelled urban places etc.;
- the economic aspect, when the repair costs will be markedly lower against using the classical reparation methods namely without the calculation of social marginal necessary costs.

The category of the trench-less technologies we can complete with the basic classifying of trench less-methods:

1. Trench-less technologies for reconstruction and rehabilitation with the removing of old (original) pipes:
   - The disruption of old pipes
   - Drifting of old pipes
   - Pull-out of old pipes

- Force-out of old pipes
- Combination of previous methods

2. Trench-less methods for reconstruction or rehabilitation pipes, but with the conservation of original structure of the pipe with the exploitation of the next synergism, minimal the static aspect;

- trench-less technologies use for reconstruction and rehabilitation of local pipe failures removing;
- creating the new inside surfaces in pipes;
- exploitation of new designing components, which are applied in original pipes.

3. Trench-less technologies for build-up the new lines

- without the soil removing;
- with the soil removing;
- micro-tunneling using;
- directional drilling;
- using the plough.

Trench-less technologies significantly respect the priority if the environmental protection and the nature much more perfectly than using the classical technologies for reconstruction and rehabilitation of the engineering lines structure.

Legislative requirements. The significant influence on reconstruction and rehabilitation in Slovak republic has the legislative, the acts, regulations and directives. The STN EN 752-4 "Drain and sewer systems outside buildings. Hydraulic design and environmental considerations" take consideration on the functional requirements, which define the final state of sewer system. Regulation of MoE SR No. 684/2006 follow the principles of STN EN 752, part 2, which define the functional requirements on sewer systems and define that the sewer system and households must fulfill the basic criteria of functionality**:**

- operation of the pipe system doesn't allow the plugging;
- it must be ensure the protection of public welfare, the work-force health;
- the recipients must be protected against pollution in specified limits;
- flooding periodicity must fulfill the requirement criteria;
- the overload periodicity must fulfill the criteria;
- sewer system and households must not expose the neighbour structures and engineering networks;
- it must be achieve the required lifetime and integrity;
- watertight of sewers and households must fulfill the test requirements;
- it must be prevent against the odor and toxicity;
- it must be ensure the suitable access for the maintenance.

The act No. 442/2002 about the public water-supply and public sewerages changes and complemented with the act 276/2001 about the regulation of network industries as amended of later regulations determine the requirements for the public water supply and sewerages, which must be observe, and which allow or limited the using of the trench-less methods. The exploitation of specific method is limited by other acts, regulations and directives too. The significant are: STN 75 6101: *Sewerages and sewer households*; STN EN 1 2889 – trench-less build and sewer and household testing; STN EN 13380 – *general requirements on components using for renovation and reparation of sewers outside the structures;* STN EN 13566 – *the plastic pipe systems using for reconstruction non-pressure sewer pipes and sewer system* and some similar regulations.

The decision for the using of the specific trench-less method exploitation is selected by the structure, hydraulic and environmental view and the insurance of security and safety in maintainance.

The no small role has the geotechnical investigation, which could recommend improve or constraint the using methods.

The pipe exploration by the CCTV camera is very important before using the specific methods.

The praxis shows, that we cannot define the universal approach for the reconstruction and rehabilitation. This fact allow to rise up the various companies, which are be engaged in this actual problematic. By this, many various methods they used, with the own access of creativity, with the aim to minimize the costs and maximize the effect of the reconstruction with the preservation of adequate benefit of the works. The no-small role has the guarantee of the performing works, the works quality, references of the companies. So from these facts we can assert, that the trench-less exploitation depend on some basic technical aspects and on the marketing policy.

The most widely used trench-less methods. The methods, which are very often used for the rehabilitation, follow the most technical and cost effective trends.
The ZEPRIS company, which repair the sewer, water and gas pipes used for the reconstruction very often the technology **Compact Pipe, Berstlining, GFK-liner and Short-liner**. The next technologies, which are possible to use are: **Microtunneling, Pilot-Pipe, Rolldown, Slipline, Subline, Swageline**

The **Compact pipe** technology is a close-fit lining technique for various damaged utilities, ranging from sewers to gas-, water- and industrial lines. The pipe itself is a continuous string of polyethylene pipe, folded into a C shape along its length. This compact shape is created during manufacturing, when the pipe, while it is still hot, is folded. It is supplied from the manufacturing plant wound on a drum. Through its folded shape, the pipe has become substantially smaller in cross sectional area, allowing it to be easily inserted into the existing pipeline.

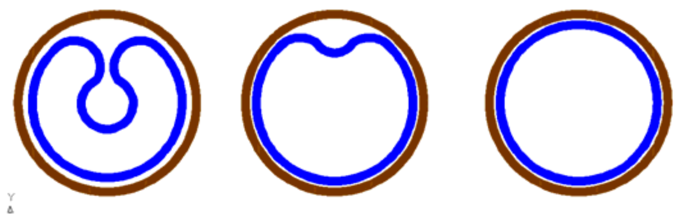

Fig. 1 Principle of Compact Pipe system

The compact-pipe is a rehabilitation technique best suited for inner city areas due to its minimal space requirements. This method allows the replacement of up to 700 m at one time in the nominal diameter range of between DN 100 and DN 500.

Fig.2. Reparation of the sewer system Stupava DN300 by Compact Pipe

The **swagelining** technology makes possible a cost-effective pipe renewal while hardly effecting road traffic. The special feature of swagelining is the larger diameter of the liner compared to the inside diameter of the old line. The return of the liner to its original diameter after having been pulled into the host line ensures its permanent tight fit. How it works? Following cleaning of the old pipe, it is inspected by a TV camera and calibrated. Before being pulled in, the diameter of the liner is reduced by means of a reduction tool, the swagelining die. The applied tension forces are continuously monitored during this process

Fig. 3. C-shape in manhole

When the liner has reached its final position, the winch is slackened and the PE liner presses tightly against the inside wall of the host line - this guarantees a continuous and permanent tight fit. The swagelining technique is suitable for all pipeline diameters ranging from DN 65 to DN 1000.

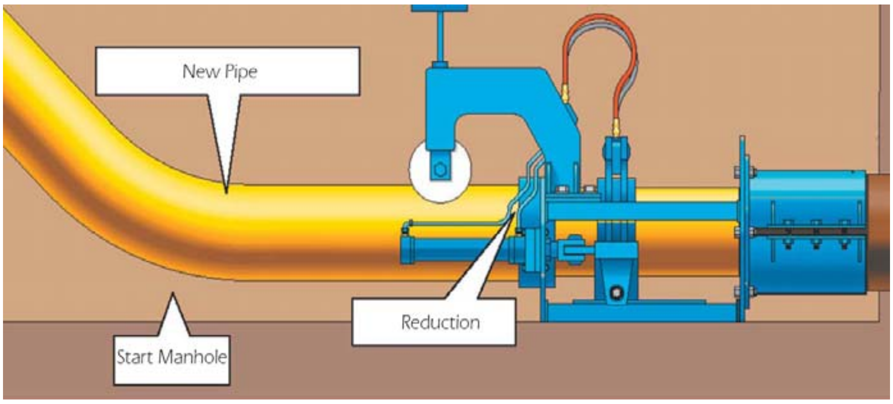

Fig. 4. Swagelining system

The **GFK liner** method is a hose relining technique using a seamless fiberglass textile hose as the liner. Sewers which show damages caused by roots, deposits, closure misalignments, cracks and fragment breaks can be rehabilitated by this method.

How it work?: After the pipe has been cleaned and inspected by means of a camera it is prepared for its rehabilitation by milling and smoothing robots. Then, a winch pulls the folded liner hose via a pit into the damaged pipe. The liner unfolds by means of compressed air and fits to the internal wall of the old line. The hose cures by the application of an airsteam- mixture following a defined temperature curve. The curing process is continuously controlled and reported.

The **short liner** is a method for the trench-less repair of sectional damages in pipelines. Sealing occurs by means of a short hose inserted into the damaged pipe section.

The principle: When the damaged pipeline has been cleaned, the areas in need of repair are localized and precisely measured by video inspection. Subsequently, a packer with a resin-impregnated hose is positioned at the area concerned. At the repair area the circumference of the packer is expanded by compressed air thus pressing the short hose to the internal pipe wall. The excess polyurethane resin escapes and seals cracks and holes. After a short period of time the resin cures and forms a tight bond with the textile hose. The pipe diameter is only reduced minimally by the textile hose. When the curing process has come to the end, the packer is vented and removed out of the line.

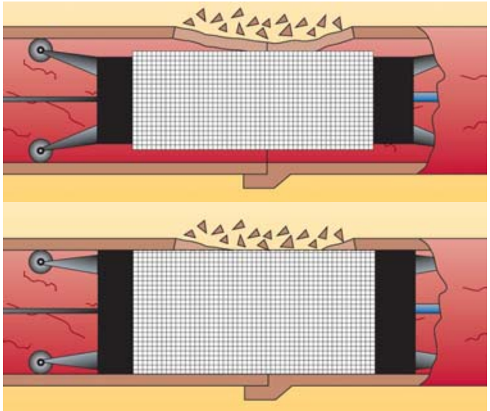

Fig. 5. Short Liner system

Berstlining is a trenchless pipe replacement technology which uses the original line routing, thereby keeping costs for underground engineering to a minimum. A particular advantage of this technique is having the possibility of enlarging the diameter of the line. When we using the berstlining technology, the old pipe is destroyed and the new pipe line is pulled through the existing host line. Old pipes made of stoneware, concrete, cast iron or fiber cement are smashed into pieces and displaced. Old pipelines made of steel or plastic materials as well as defect liners are cut up and expanded. With both of these methods, the old pipe material remains in the soil. Since the old line will no longer be required, preparatory measures such as high pressure cleaning or removal of protruding parts are generally not necessary. The diameter for reparation are from DN100 to DN500. This method was used in 375m reparation of the water pipe DN200 – Solosnica by HDPE 100SDR17 PN10 DN200.

Microtunneling is a high-performance and environmentally friendly alternative to pipeline construction with trenches; it can also be used in the most demanding of circumstances: ground water and difficult geologies are no problem for microtunneling, and it has proven to be a very good method of avoiding obstructions in city centres.

The Pilot Pipe method is a trenchless pipe replacement technique developed by the Pfeiffer group of companies for the potable water and gas piping sector, in which the old pipe is removed completely from the soil and the new one drawn in along the same route. The Pilot Pipe method can be used for replacement of existing pipelines in DN 80 to DN 1600, and consisting of cast iron, steel, asbestos cement, and prestressed or reinforced concrete. The new pipeline may consist of steel, ductile iron, PE or other materials with similar tensile strengths. Unlike other methods available on the market, Pfeiffer‘s can cope with significant changes in ND and installation lengths of 60 to 80 m with no intermediate pits.

Rolldown system: Pipeline conduit relining in the Rolldown technique is an economic solution for inner city areas and for longer distances to be rehabilitated. In the range of between DN 1200 and DN 500, lengths of up to 1000 m do not present a problem.

Slipline allows the rehabilitation of long pipelines at low costs. In this method, standard PE pipes are used as liners and are welded together on site to form one pipeline conduit.

Subline is a pipeline conduit relining technique by means of which long lines and large diameters can be rehabilitated cost-effectively and permanently without difficulties.

4 Conclusions

The trench-less methods are mostly non-invasive methods for the pipe renovation. The decision, which system use, is very often specifying by the problem, by the local conditions and by the economic view. Slovak water and gas companies must do a decision, which system used for the renovation of their pipes. This decision is depend on the trench-less methods knowledge and on the economical evaluation. These facts determine the specific methods exploitation. Follow, the references very often determine the next using of specific method, because the companies need the absolute success of the using methods.

Acknowledgement

The article is supported by the Scientific Grant Agency of the Ministry of Education – VEGA Project No. 1/0854/08 dealt with at the Department of Sanitary and Environmental Engineering, Faculty of Civil Engineering, Slovak University of Technology Bratislava.

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A63**

Slovak Water Resources–Present Conditions

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Abstract. Water supply resources are groundwater and surface water bodies currently used or intended for prospective use. Water used from identified water bodies shall meet relevant qualitative objectives and resulting requirements on water quality and quantity according to its purpose of use. Water resources protection should be viewed as an integrated protection of quality and quantity of substance and ground water, including natural curative springs and minerals waters. For water resources protection the protection zones with limited agricultural use and other activities are designated according to the valid legislation.

Keywords: Groundwater quality, surface water quality, water protection, water resources

1 Introduction

Anthropogenic activities realized in river basins may result in a deterioration of water quality with detrimental effects on the ecosystems. Therefore, the use of such polluted water for drinking purposes, irrigation, industrial use, and fishing or for recreation can be limited and ecological functioning threatened. In the decision making processes of water management authorities is important to have sufficient and reliable information on water quality status.

2 Surface Water Quality

In the year 2007 the national surface water quality monitoring in SR has been divided into 3 categories [5]:

- surveillance monitoring,
- operational monitoring,
- monitoring of protected areas.

This division is in compliance with the Regulation No. 221/2005 on evaluation and monitoring of surface and groundwater, national water register and water balance. Surface water quality was monitored according to the reduced version of the Programme of water status monitoring in 2007 [7]. Some of the sampling sites have been monitored for more (different) purposes, hence the water quality evaluation according to the Slovak national standard 75 7221 and National regulation 296/2005 has been done only for 124 sampling sites [9, 10].

The monitoring frequency of given determinants was different in the period 2006-2007, ranging between 1-24 times per year. Determinants with lower monitoring frequency are:

- biological determinants,
- heavy metals,
- specific organic substances.

The Water Act No. 364/2004 Coll. divides the Slovakia territory into two international river basins Vistula and Danube, which are then divided into river basins of Poprad, Dunajec, further more Dunaj and Morava, Váh (including Malý Dunaj) and Nitra, Hron, Ipeľ and Slaná, Bodrog, Hornád and Bodva [11]. According to this division the assessment of surface water quality in period 2006-2007 has been done.

The results of the laboratory analyses (physical-chemical, chemical, biological, microbiological determinants and on chosen sampling sites also radioactivity determinands) are processed in compliance with the national standard STN 75 7221 „Surface water quality". Validity of this standard was cancelled by the Slovak institute of technical normalization by 01.03.2007.

In this classification of surface water will be changed with Water Framework Directive. In preparation is new classification scheme. This time values are compared with values for surface water in Government Regulation No. 296/2005 Coll. Establishing Qualitative targets for surface Waters and Limit Values of Pollution Indicators of Wastewater and Special Waters.

Requested values in this regulation were fulfilled in 100 % for physical-chemical parameters as: TOC, Sulphate, Magnesium, Calcium, Anionic active surfactants, Cyanides, Copper, Zinc, Chromium, Nickel and some of organic pollutants. Values were exceeding for aluminum and selenium (100 %), also in many monitoring points were exceeding values for AOX and chloroform. Microbiological parameters were frequently exceeding values for fecal streptococci, total coliforms and fecal coliforms bacteria.

Table 1. Amount of monitored surface water quality sampling sites in the year 2007

River basin district	River basin	Number of sampling sites	Monitored river length (km)
I. DUNAJ	Morava	10	325,7
	Dona	10	184,1
II. VÁH	Váh	30	1083,6
	Nitra	9	337,2
III. HRON	Hron	10	358,5
	Ipeľ	10	392,4
	Slaná	5	224,1
IV. BODROG	Bodrog	19	627,3
V. HORNÁD	Hornád	13	494,2
	Bodva	4	127,4
VI. DUNAJEC AND POPRAD	Dunajec	1	16,9
	Poprad	3	142,6
Slovakia summary		124	4314

Trichloroethylene and tetrachloroethylene were not classified, because LOQ limit was higher. Through that in 14 monitoring points values for trichloroethylene were higher than LOQ and limit in Government Regulation No. 296/2005. Classification is in Table 2.

The most polluted of monitored streams are Myjava, Mláka in the Morava river basin, Trnávka, Dolný Dudváh in the Váh river basin, river Nitra, in Nitra river basin, Slatina and Zolná in the Hron river basin, Krivánsky potok, Krtíš in the Ipeľ river basin, Torysa, and Sokoliansky potok in Hornád river basin and Trnávka, in the Bodrog river basin. The reasons of pollution in these streams are industrial point sources and municipal waste waters from cities and villages. It has to be taken into account that tributaries are monitored mainly at this mouth where are demonstrated all anthropogenic influences along the streams (besides these polluted streams are predominantly small rivers which do not have sufficient dilution capacity regarding discharged pollution).

The main objectives of the national surface water quality monitoring programme in the Slovak are as follows:

- characterization of the present state of surface water quality in Slovak Republic;
- identification and quantification of the main problems of water pollution;
- evaluation of trends in surface water quality in Slovak Republic;
- provision of information on water quality for decision making processes of water management authorities, provision of information to public and different international organizations as ICPDR, EEA and OECD;
- controlling meeting the imission criteria for surface water quality based on the Regulation No. 221/2005;
- evaluation of water status against criteria given according to the different water uses;
- reporting to the EU.

Table 2. Evaluation of Monitored Surface Water Quality Parameters under the Regulation 296/2005 for 2006 - 2007

Parameter	Unit	Total number of monitored sampling sites	Number of monitored sampling sites	% meeting the requirements of the Regulation 296/2005
Dissolved Oxygen	mg/l	123	118	96
COD Mn	mg/l	42	40	95
COD Cr	mg/l	114	90	79
Total Organic Carbon	mg/l	22	22	100
BOD with nitrification inhibition	mg/l	98	90	92
Free Ammonia	mg/l	47	47	100
Water Reaction		123	114	93
Water Temperature	°C	123	118	96
Dissolved Solids	Mg/l	68	64	94
Total Iron	mg/l	37	32	86
Total Manganese	mg/l	37	33	89
Ammonium Nitrogen	mg/l	121	106	88
Nitrite Nitrogen	mg/l	121	44	36
Nitrate Nitrogen	mg/l	121	114	94
Organic Nitrogen	mg/l	57	54	95
Total Phosphorus	mg/l	89	76	85
Total Nitrogen	mg/l	123	118	96
Dissolved solids - ignited	mg/l	52	47	90
Chlorides	mg/l	109	105	96
Sulphates	mg/l	109	109	100
Calcium	mg/l	104	104	100
Magnesium	mg/l	104	104	100
Fluorides	mg/l	1	1	100
Phenols - volatile	mg/l	71	68	96
Anion Tensides	mg/l	41	41	100
Non-polar extractable substances -UV	mg/l	74	53	72
Total Cyanides	mg/l	16	16	100
Active chlorine	mg/l	32	17	53
Mercury	µg/l	26	22	85
Cadmium	µg/l	20	20	100
Lead	µg/l	20	19	95
Arsenic	µg/l	17	16	94
Copper	µg/l	25	25	100
Total chrome	µg/l	16	16	100

Parameter	Unit	Total number of monitored sampling sites	Number of monitored sampling sites	% meeting the requirements of the Regulation 296/2005
Nickel	µg/l	16	16	100
Zinc	µg/l	19	13	68
Selenium	µg/l	1	1	100
Aluminium	µg/l	11		0
Saprobic Index of Bioseston		57	48	84
Coliform Bacteria	KTJ/ml	76	26	34
Thermotolerant Coliform Bacteria	KTJ/ml	70	17	24
Faecal Streptococci	KTJ/ml	52	9	17
Chlorophyll	µg/l	51	43	84
Saprobic index of microflora growth		1	1	100
Producers in 1 ml(aut.org.)	number/1ml	32	25	78
Abundance of phytoplankton	number/1ml	11	8	73
Total volume alpha activity	mBq/l	26	25	96
Total volume beta activity	mBq/l	29	27	93
Radium 226	mBq/l	3	3	100
Tritium	Bq/l	13	13	100
Absorbed organic halogens	µg/l	30	3	10
Pentachlorophenol	µg/l	14	14	100
Benzene	µg/l	47	47	100
Toluene	µg/l	32	32	100
Chlorobenzene	µg/l	1	1	100
1,3-Dichlorobenzene	µg/l	4	4	100
1,4-Dichlorobenzene	µg/l	4	4	100
1,2-Dichlorobenzene	µg/l	4	4	100
Sum Xylene	µg/l	32	32	100
Chloroform	µg/l	44		0
1,2-Dichloroethane	µg/l	41	38	93
Tetrachloromethane	µg/l	36	_	
1,1,2-Trichloroethylene	µg/l	36		0
1,1,2,2-Tetrachloroethylene	µg/l	29	29	100
Cis 1,2 - dichloroethene	µg/l	29	21	72
Benzo(a)pyrene	µg/l	57	57	100
Fluoranthene	µg/l	57	54	95
Naphthalene	µg/l	57	57	100
Hexachlorobenzene	µg/l	52	52	100
Lindane	µg/l	54	54	100

Parameter	Unit	Total number of monitored sampling sites	Number of monitored sampling sites	% meeting the requirements of the Regulation 296/2005
1,2,4-trichlorobenzene	µg/l	46	45	98

3 Groundwater Quality

Basic evaluation unit of groundwater balance is a hydrogeological region with its subsequent classification into sub-regions. According to valid hydrogeological regionalization the territory of Slovakia was divided into 141 hydrogeological regions. Based on water management balance data, natural resources of Slovakia are amounted to 146.7 $m^3 \cdot s^{-1}$. Groundwater resources represent amount of 76,830 $l \cdot s^{-1}$, i.e. more than 52% of natural resources. The Committee for Available Groundwater Quantity Classification approved 45,149 $l \cdot s^{-1}$, representing 58.8% of available groundwater amount and 30.8% of natural groundwater resources.

The total available groundwater amount represents the sum of available resources approved by the Committee for Available Groundwater Quantity Classification and supplies not approved by the Committee which are determined based on volumes documented from hydro-geological researches and surveys.

Total available groundwater resources as of January 1,2008:

- approved: 45,149 $l \cdot s^{-1}$
- not approved: 31,681 $l \cdot s^{-1}$
- total: 76,830 $l \cdot s^{-1}$

Compared to the previous year there was recorded an increase in available groundwater resources by 82 $l \cdot s^{-1}$ (0.11%) in 2007. According to documented available groundwater resources it can be stated that current and also expected water demand is well assured [1].

Groundwater is preferentially intended for drinking water supply under the Water Act Groundwater abstraction has been following the downward trends in Slovakia since 1990. In 2007 used groundwater resources amounted to 11, 366 $l \cdot s^{-1}$ that represents a decrease by 299.2 $l \cdot s^{-1}$ (2.6%) compared to 2006. Data on groundwater abstraction are included in the Slovak Hydro-meteorological Institute (SHMI) Water Abstraction Register. In 2007 the SHMI Water Abstraction Register listed 5468 resources of Slovakia.

In the assessment of ground water use in Slovakia according to the purpose it can be stated that there was the decrease of water consumption in public drinking water supply of the inhabitants in animal production, social needs and other use. On the contrary, abstractions increased globally in industry and plant production.

From the viewpoint of water management use the ratio of usable amounts and abstractions varies in individual hydrogeological regions.

Monitoring of ground water quality is represented by the systematic monitoring and assessment of ground water quality condition according to the requirements of the Ministry of Environment of the Slovak Republic. In terms of this legislation the Ministry of Environment of the Slovak Republic provides locating and assessing the condition of ground water through the SHMI. Systematic ground water quality monitoring within the national monitoring programme has been provided by the SHMI since 1982.

Monitoring programmes experienced the changes in 2006. In line with the strategy for the implementation of the Water Framework Directive in the Slovak Republic the Programme of Water Condition Monitoring for 2007 was prepared. This document included the requirements to collect all the information on water condition necessary to be reported to the European Commission in required quality.

By the year 2006 the monitoring objects were distributed into 26 important water management areas (alluvial sediments, mezozoic and neovulcanic structures). Afterwards in line with the requirements of WFD the territory of the Slovak Republic was not divided into important water management areas anymore for the purpose of monitoring. Since 2007 this division has been made on the basis of bordering the ground water bodies. Monitoring of ground water chemical condition was divided into [4]:

- Basic monitoring
- Operational monitoring

Within the *basic monitoring* all ground water bodies were covered by at least one abstraction site. In 2007 the ground water quality was monitored in 130 objects of basic monitoring. These objects are either a part of state monitoring network of the SHMI or the springs not affected by point pollution sources. Ground water samples were taken once in the autumn for selected group of parameters (with the exception of boundary monitoring object where the samples were taken 3 times).

Operational monitoring was done in all ground water bodies assessed as risk because of reaching not good chemical condition. Monitoring network was enlarged by adding 34 piezometric wells in the territory of Žitný ostrov where the levels 1 – 3 are monitored, what is 84 levels altogether. The region of Žitný ostrov represents a separate part of the SHMI monitoring network because it plays an important role in the whole process of monitoring of water quality changes in Slovakia since this region is the most significant drinking water resource in our territory. In the region of Žitný ostrov the samples were taken 4 times a year for basic monitoring and twice a year for additional monitoring in spring and autumn periods (extreme groundwater levels). To meet the requirements of the Directive no. 91/676/EHS related to water protection against pollution caused by nitrates from agricultural sources, the pollution caused by nitrogenous substances was monitored in 116 objects in vulnerable territories in Slovakia within operational monitoring in 2007 [6]. Next in 2007 within operational monitoring 218 objects were monitored because there is an assumption of potential infiltration of pollution into ground water from potential pollution source or related group. The frequency of sampling was twice a year in 155 quaternary objects, four times a year in 32 pre-quaternary karst objects and once a year in 31 pre-quaternary objects.

The results of laboratory analyses were assessed under the Regulation of the Slovak Government No.354/2006 Coll. on Drinking Water Requirements and Drinking Water Quality Control [8]. The assessment is performed using comparison of measured and limits values of all analyzed indicators. The results will be published in the annual report "Groundwater Quality in Slovakia for 2007" and Biennial Report "Žitný ostrov Groundwater Quality for 2007 - 2008" [2,3].

In the objects of *basic monitoring* the limit values of concentrations in Slovakia, except for the "Žitný ostrov" region, were defined under the Slovak legislative. Recommended value of the percentage of water saturation by oxygen specified in the terrain was reached in 54% of samples. Values pH were in the interval of limit values with the exception of 4 samples, conductivity exceeded the indicating value 3 times out of the total number of 132 specifications. Within the basic monitoring objects of ground water there are the issues of unfavorable oxidation-reduction conditions becoming essential which is being pointed out by the most frequently exceeded acceptable concentrations of the total Fe (31 times), Mn (31 times) and NH_4^+ (8 times). Besides these parameters there was sporadic exceeding in case of Cl^-, SO_4^{2-} and NO_3^-. Increased concentrations of the following trace elements were recorded: Al (25 times), As (4 times), Pb (2 twice) a Sb (once). Pollution caused by specific organic substances has only local impact; majority of specific organic substances was specified under the detection limit. Limit values in this group were exceeded only in the one structure.

Ground water contains relatively small amount of oxygen which is confirmed also by the fact that the recommended percentage of oxygen saturation in water was reached only in 26% of samples. The values of conductivity measured in the field exceeded the indicating value 55 times out of the total number of 467 specifications; ph with the exception of 20 samples was in the interval of limit values. Mn and total Fe are the most frequently exceeded parameters which mean that unfavorable situation of oxidation-reduction conditions is ongoing. Besides these parameters the exceeded limit values of Cl^- and SO_4^{2-} indicate the impact of anthropogenic pollution on ground water quality. Land use pattern (agricultural areas) is reflected into increased contents of oxidized and reduced forms of nitrogen in ground water. Ammonium ions NH_4^+ (70 times) and NO_3^- (47 times) participated mostly in that exceeding. In the objects of operational monitoring the acceptable value specified by the Regulation was exceeded by 5 trace elements (Al, As, Sb, Ni and Hg) in 2007. Increased contents of Al (49 times) and As (26 times) were mostly recorded. The presence of specific organic substances in ground water is the indicator of human activity impact. Wider scale of specific organic substances was recorded. Exceeding the limit values was most frequently recorded in parameters from the group of polyaromatic hydrocarbons (phenanthren, fluoranthene, benz[a]pyrene, pyrene) and group of volatile aromatic hydrocarbons (1,3 – dichlorobenzene, 1,4 – dichlorobenzene, 1,2 – dichlorobenzene). Sporadically the limit values in the group of pesticides and volatile aliphatic hydrocarbons were exceeded.

The purpose of the monitoring programme indicates that monitoring objects of the basic monitoring are situated in the areas not affected by human activity, therefore ground water show better quality in comparison with the objects of operational monitoring designed to catch the impact of significant sources of ground water pollution.

4 Conclusions

The water resources protection in Slovakia is considered as an integrated protection of groundwater and surface water quality and quantity, including springs and mineral waters. Quantitative protection is based on accumulation ability and management of particular region with respect to abstracted or pumped water. This is the reason why the limit for surface water use is determined by so-called ecological limit (MW_{eko}), which has no effect on a habitat in river basin.

One of the key roles of water protection in terms of water quality is to resolve the problems relating to sources of pollution. Pollution sources, which have a negative impact on water quality, are broken down into two categories based on the type and severity of their impact: *point* sources of pollution and *non-point* sources of pollution

Acknowledgement

The article is supported by the Scientific Grant Agency and the Cultural and Educational Grant Agency of the Ministry of Education – VEGA Project No. 1/0854/08 and KEGA Project No. 3/5125/07, and the Project: „Operational Programme Research and Development – Excellence Centre of Integral Territory Flood Protection (Centrum excelentnosti integrálnej protipovodňovej ochrany územia)“, code ITMS 26240120004 dealt with at the Department of Sanitary and Environmental Engineering, Faculty of Civil Engineering, Slovak University of Technology in Bratislava.

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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A71**

Natural Curative Waters of Slovakia in Spa

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Abstract. Slovakia is one of the countries which is rich in amount of cold and thermal natural curative waters. These waters belong under the special waters by the Slovak Water Act. They are used mostly for the healing processes in spas and they are also filled into consumer's packages in the natural mineral waters bottling factory. The article talks about the places where natural curative waters of Slovakia appear; also it deals with the valid legislation for the processes from its approval to its usage. The natural mineral waters filled for the consumers, distributed by the commercial market, come under the Ministry of Economy of the Slovak Republic regulation too, but these waters must be accredited by the European standards. The Ministry of Health of the Slovak Republic defines the protective zones for providing the water protection. They are undergoing the revision in every location where the natural curative sources and natural mineral waters are used in connection with the nowadays valid legislative rules.

Keywords: Curative waters, mineral resources, balneology, state-of-art operation

1 Introduction

Treatment on the base of water is known a long time ago. The first knowledge is kept from ancient Greece. Perhaps the greatest development of balneology was reached in Roman Empire. The spas were built very luxury because the main part of social life was held there. The fall of Roman Empire leaded to decline of balneology in Europe for a long time. At the beginning of 16th Century starts the renaissance of spas. In the locations of mineral waters the old spas are renewed and the new ones are erected in the whole Europe. In Slovakia the spa Piestany and Trencianske Teplice were originated at that time. Since the half of 16th Century the drinking cures and curative muds started to be applied. At this time other spas in our area were originated – Bojnice, Turcianske Teplice, Bardejov, Herlany and others.

For the cognition of spas the first legal steps of the state are important for us, as the provision of Maria Theresa from the year 1763 about the inventory of spa and mineral water in Austro-Hungarian Empire. The inventory contains the description of 128 locations of mineral waters in Slovakia. At the end of 18^{th} Century flourished the spas Trencianske Teplice, Piestany, Turcianske Teplice, Sliac, Bardejovske Kupele, Rajecke Teplice, Vysne Ruzbachy and others [1].

2 European Balneology

In Europe are at present about 1800 spa locations with thousands of balneo clinics, hotels and pensions oriented mainly on spa guests. Generally we can talk about 3 regions. In the first one are the countries which are not rich in natural curative waters as Sweden, Norway, Finland, Denmark, Holland, Great Britain and Ireland. They are under the influence of American short-time stays in relaxing centres. The client takes the bath with additives, various oriental curative processes, massages, meditations and training to body building. The cures in the whole region of Central and South Europe are oriented to utilisation of thermal mineral waters and climate cures where there is a long-lasting tradition of their using. In spa there is the continuous medicine background, the rapid development of the new product is registered, so called thalassotherapy that use the curative effects of sea water, seaside climate, sea mud and sea algae. In France is this branch of balneotherapy more popular that classic balneotherapy.

Among the balneal leaders are Germany, Italy, but the Czech and Slovak Republics as well if we take the classic balneotherapy into account. The special status has Iceland that owns the biggest amount of the thermal springs, so it has presumptions to be the balneal leader. The biggest concentration of the spas with traditionally highly-developed balneology is in Germany where about 250 balneal enterprises and clinics are in operation. In Austria there are about 200 spas, in Czech Republic about 70 spas, following by Poland, Hungary and Slovakia. The coastal states as Estonia, Latvia, Lithuania and Poland combine the traditional balneotherapy with centres of thalassotherapy [3].

The common European programmes for balneal stays are originated on trans-boundary health care. European Association of Spas elaborated catalogue of criteria for the standards of quality for balneal products. The spas must be first of all approved in its county and to have the concluded contracts with insurance companies. The advantage is to own the quality certificate Europespa-med. The Slovak spas have more positives – they own the superior curative waters and qualitative health care, gradually they improve their visual appearance and their offers for free time activities.

Natural curative resources and natural mineral waters. According to the Law No. 364/2004 Z.z. – Water Act, in § 3 section 4 ground waters are preferably intended for drinking water supply. Then in section 5 are determined waters, which are approved as natural curative resources and as natural resources of mineral table waters, according to special provision, later only „special waters", to which is this law applied only if it is expressly stated.

In Slovakia, there are about 1,500 natural mineral resources in evidence. In total, the Ministry of Health now monitors 156 objects. 105 from them are approved as natural curative or natural mineral resource. Besides that there are monitored other 53 resources which are not approved.

Mineral water is ground water with authentic origin accumulated in natural environment, springing on surface from one or more natural or artificial ascent ways. This water differs from other ground water especially [6]:

a) with its origin;

b) content of trace elements;

c) content and character of total dissolved solid substances exceeding 1 000 $mg.l^{-1}$ or content of dissolved gases exceeding 1 000 $mg.l^{-1}$ of carbon dioxide or at least 1 $mg.l^{-1}$ hydrosulphide; or

d) minimal temperature in spring 20^0C.

The Law of the Slovak National Council No. 538/2005 Z. z. on natural curative waters, natural curative spas, balneal sites and natural mineral waters, and on change and amendment of some laws, so called balneal law is the first valid separate legislative rule for Slovak curative spa. It came to force on January 1, 2006, some of its parts on March 1, 2006. The part of the law is 6 generally binding rule of law. According to the Slovak Constitution, natural curative sources and natural mineral waters as a part of ground waters are owned by state. They are neither part nor accessories of land. The law states conditions, under which natural curative water and natural mineral water after exploitation from natural curative resource or natural mineral resource becomes the property of a physical or legal person.

In order to secure supervising over keeping rules stated in this law, and according provisions the Ministry of Health of the Slovak Republic established Inspectorate of Spas and Springs (IKŽ). The State Balneal Commission (ŠKK) is according to the law No. 538/2005 Z. z. established at the Ministry of Health of SR. It deals with natural mineral waters, natural curative waters, natural curative spas and climate conditions suitable for curing and it is a management body, which decides on the first level.

The Inspectorate of Spas and Springs was founded at the Ministry of Health on 1.1.1958 in Prague with scope for the whole Czechoslovak Republic. Since 1967 it is a part of Ministry of Health in Bratislava. In its 40 year history the Inspectorate solved many tasks of legislation, organizational, check and other nature. The Inspectorate participated in preparation of laws, provisions, directives, statements and other rules connected with its field of activity.

Approval of natural curative waters and recognition of natural mineral waters is defined in balneal act in §5-9. The mineral water can be **approved *as natural curative water***, if during at least *five years* there were proven its curative effects in balneal practice and it fulfils requirements stated by generally binding rule of law, issued by the Ministry of Health. Curative effects can be considered as proven also if such effects were verified in long term balneal practice when water with similar physical and chemical properties was used. The procedure of approving of natural curative water and natural mineral water concerns not only waters from Slovak resource but also waters imported to Slovakia as an EU member from third countries. The Law states conditions according to which it is possible to recognize mineral water as natural mineral water.

As the ***Natural mineral water*** can be ***declared*** only the water from resource which was at least *three years* monitored, and during this period it was proven the stability of all decisive indicators, its nutrition properties were not changed, and it fulfils requirements stated by generally binding rule of law, issued by the Ministry of Health. If the resource user cannot due to technical reasons use the natural mineral water from resource to which it was issued the approval to use, and he/she applies for approval of natural mineral water with the same chemical composition from new resource from the same hydrogeological aquifer, the condition in previous sentence is considered as fulfilled.

3 Use of Natural Curative Resources and Natural Mineral Resources

It is possible to approve use of natural curative resource and natural mineral resource only if the water from resource was already approved. The resource can be used only after the decision on approval was issued, and after receiving the certificate.

Natural curative resources should be *preferably used for curing* and can be used only in the scope of approval. Requirements on obtaining, processing, filling, marking and putting on a market of natural mineral waters in consumer packing are settled by the law No. 152/1995 Z. z. on food in sounding of later rules.

Use of natural curing resources and natural mineral waters is checked by monitoring system which set and defined in § 2 section 14 of the law. At present, 39 localities in Slovakia are monitored, and 36 of them use local information system (LIS) IKŽ, which secures transfer of data to Central Monitoring System (CIS) IKŽ.

For the benefit of protection of natural curative resources and natural mineral resources the law states the duty to determine **protection zones.** Protection zones are determined on 2 levels according to expertise and professional background data elaborated by competent person. Borders of protection zones, procedure of treatment, kinds of forbidden activities, scope of protection measures, and way of their change and cancellation state generally binding rule of law. Regulation also states requirements of proposal of determination of protection zones.

Natural mineral water is microbiologically clean ground water springing on surface from one or more natural or artificial ascending ways, which fulfils qualitative requirements according to special regulation and was approved according to balneal law as suitable for use as nutritive and for production of packed natural mineral waters. It differs from common drinking water by characteristic original content of minerals, trace elements or their parts, as well as physiological effect and its original state. Natural mineral waters in consumer package are equipped with description of all properties determined by certified laboratory. Present filling plants of natural mineral waters in Slovakia are listed in the Table 1.

Table 1. List of natural mineral waters approved by the Ministry of Health SR

Commercial name	Name of the spring	Locality of exploitation
Budiš	B-5 B-6	Budiš
Fatra	BJ-2	Martin - Záturčie
Maštinská Ave	HM-1 ST-1	Maštinec
Ľubovnianka	LZ-6 (Veronika)	Nová Ľubovňa
Gemerka Maxia	HVŠ-1 ŠB-12	Tornaľa
Baldovská	BV-1 B-4A	Baldovce
Odyseus	S-1(Cifrovaný)	Lipovce
Salvator	S-2 (Salvator)	Lipovce
Slatina	BB-2	Slatina
Čerínska minerálka	ČAM-1	Čačín
Mitická	MP-1	Trenčianske Mitice
Kláštorná	KM-1	Kláštor pod Znievom
Matúšov prameň	CC-1	Lúka

4 Balneology in Slovakia

Balneal law introduced division of health facilities that provide balneal care to **„natural curative spas“**(PLK) and **„balneal medical institution“**(KL). The law introduced new system of approving operation of these health facilities.

Table 2. Indicatory list for balneal care [8]

Indications of patients after 18 year age		Indications of patients under 18 year age	
I.	Oncological deseases	XXI.	Oncological deseases
II.	Deseases of circulation system	XXII.	Deseases of circulation system
III.	Deseases of digestive organs	XXIII.	Deseases of digestive organs
IV.	Deseases of metabolism and endocrine glands	XXIV.	Deseases of metabolism and endocrine glands
V.	Non-tuberculosis deseases of respiratory organs	XXV.	Non-tuberculosis deseases of respiratory organs
VI.	Nerve deseases	XXVI.	Nerve deseases
VII.	Deseases of locomotive organs	XXVII.	Deseases of locomotive organs
VIII.	Deseases of kidneys and urinary tract	XXVIII.	Deseases of kidneys and urinary tract
IX.	Mental deseases	XXIX.	Gynaecological deseases
X.	Skin deseases	XXX.	Skin deseases
XI.	Gynaecological deseases		
XII.	Occupational deseases		

Table. 3. State-of-art operation of natural curative spas and balneal medical institutions

Č.	type	Operator: indications (according to law No. 661/2007 Z.z., appendix 6)
1.	PLK	Bardejovské Kúpele a.s.: I., II., III., IV.,V., VIII., XII., XXI., XXII., XXIII., XXIV., XXV., XXIX.
2.	KL	MV SR Družba, Bardejovské Kúpele: I., II., III., IV., V. ,VIII., XII.
3.	PLK	Kúpele Bojnice a.s.: VI., VII.
4.	PLK	Kúpele Brusno a.s.: II., III., IV., XII.
5.	PLK	Prírodné jódové kúpele Číž, a.s.: II., IV., VI., VII., XII., XXVI., XXVII.,
6.	PLK	Kúpele Dudince a.s.: II., VI., VII., XII., XXVII.
7.	KL	Slovthermae, Kúpele Diamant Dudince, š.p: II., VI., VII., XII.
8.	PLK	Wellness Kováčová, s.r.o.: III., IV. (AKS), VI., VII., VIII., XI.(AKS), XXVI. (AKS),, XXVII.
9.	KL	Špecializovaný liečebný ústav Marína, š.p., Kováčová: I., VI., VII., XXI., XXVI., XXVII.
10.	PLK	Liptovské liečebné kúpele a.s. Lúčky: I., VI., II., XI., XII.
11.	PLK	Kúpele Nimnica a.s.: I., II., III., IV., V., VI, (okrem 5,6), VII., XI., XII. (okrem 2)
12.	PLK	Slovenské liečebné kúpele Piešťany a.s.: VI., VII., XII., XXVI., XXVII.
13.	KL	Vojenské zdravotnícke zariadenia, a.s., Piešťany: VI., VII., XXII.
14.	PLK	Slovenské liečebné kúpele Rajecké Teplice a.s.: VI., VII., XII.
15.	PLK	Kúpele Sliač a.s.: I., II., XI.
16.	PLK	Liečebné termálne kúpele a.s., Sklené Teplice: VI., VII., XII.
17.	PLK	Slovenské liečebné kúpele Piešťany a.s. – Smrdáky: VII., X., XII., XXX.
18.	PLK	Slovenské liečebné kúpele a.s., Trenčianske Teplice: VI., VII., X., XII.
19.	KL	KLÚ MV SR ARCO, Trenčianske Teplice: VI., VII.
20.	PLK	Slovenské liečebné kúpele Turčianske Teplice, a.s.: I., III., VI., VII., VIII., XI., XII., XXI., XXIII., XXVI., XXVII., XXVIII., XXIX.
21.	PLK	Kúpele Vyšné Ružbachy a.s.: I., II., III., IV., VIII., IX., XI., XII.
22.	PLK	Kúpele Lučivná, a.s.: XXV.
23.	KL	KRÚ MV SR BYSTRÁ, Liptovský Ján: I., IV., V., XI., XII.
24.	PLK	Kúpele Štós, a.s.: I., IV., V., IX,. XI., XII., XXI., XXIV., XXV.
25.	PLK	Kúpele Štrbské Pleso, a.s.: V:
26.	PLK	Kúpele Horný Smokovec s.r.o.: XXV.
27.	PLK	Kúpele Nový Smokovec, a.s.: IV., V., IX., XII.
28.	KL	Sanatórium Dr. Guhra n. o., Tatranská Polianka: IV., V.
29.	KL	TATRASAN, s.r.o., Nový Smokovec: V.
30.	PLK	Vojenské zdravotnícke zariadenia, a.s., Tatranské Matliare: I., IV., V., XI., XII.
31.	KL	Sanatórium Tatranská Kotlina n. o.: V.

Legend:

PLK Natural curative spas (more than one health facility)

KL Balneal medical institution (independent health care institution)

KS Ambulant balneal care [8]

Table 2 presents list of indications, for which it is possible to organize balneal cure paid from public resources. Table 3 shows actual state of operation of natural curative spas and balneal medical institutions according to valid approvals for operation [8].

All mentioned balneal institutions are entitled to include in their offer the term „curative“, since they have approved resource of natural curative water. On the other hand, none of aquapark in Slovakia has not approved source of water, which is used in pools. The structure of clients in spas changed considerably. Only 40% of clients are sent by Slovak health insurance companies. About 40% of clients are foreigners, the rest are home self-payers.

6 Conclusions

The European Union prepares a prospective plan of balneology, which should be worked out by particular member states in their own programs. Qualified estimations say, that every second European in 2050 will suffer from depressions, 12% will have diabetes, the progress of oncology diseases will not stop, cardiovascular diseases will remain widespread as well as locomotive organ problems.

However, those problems can appear in 40-years old people under condition of present living style and lack of movement. Spas in future will be mostly oriented to prevention not only for common diagnoses, but stays in spa will be oriented to programs against drugs, alcohol and smoking.

Slovak balneology is a part of our heath system like in most of European countries, and it is comparable with countries with the most developed balneology. Just high quality health service attracts foreign guests to Slovak spas. We should keep and improve this image and quality of Slovak balneology.

Acknowledgement

The article is supported by the Scientific Grant Agency and the Cultural and Educational Grant Agency of the Ministry of Education – VEGA Project No. 1/0854/08, and KEGA Project No. 3/5125/07 and project „Operational Programme Research and Development – Excellence Centre of Integral Territory Flood Protection (Centrum excelentnosti integrálnej protipovodňovej ochrany územia)“, code ITMS 26240120004 dealt with at the Department of Sanitary and Environmental Engineering, Faculty of Civil Engineering, Slovak University of Technology in Bratislava.

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6. Vyhláška MZ SR č. 100/2006 Z.z., ktorou sa ustanovujú požiadavky na prírodnú liečivú vodu a prírodnú minerálnu vodu, podrobnosti o balneologickom posudku, rozdelenie, rozsah sledovania a obsah analýz prírodných liečivých vôd a prírodných minerálnych vôd a ich produktov
7. Vyhláška MZ SR č. 101/2006 Z.z., ktorou sa ustanovuje minimálne materiálno-technické a personálne vybavenie prírodných liečebných kúpeľov a kúpeľných liečební a ustanovujú indikácie podľa prírodných liečivých vôd a klimatických podmienok vhodných na liečenie
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International Symposium on Water Management and Hydraulic Engineering
Ohrid/Macedonia, 1-5 September 2009
Paper: **A104**

Axial Dispersion in Pressurized Water Distribution Networks–A Review

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Abstract. A review of recent findings on axial dispersion in pressurized water distribution networks is presented. Solutions, with axial dispersion via a two-dimensional advection-diffusion-reaction model and without dispersion using a one-dimensional advection-reaction model, are compared. Axial dispersion turns out to be an important transport process in laminar and transitional flows. The effects of stochastic water demand on solute spreading are discussed. Expressions for the time-averaged rates of axial dispersion are presented. An Eulerian-Lagrangian scheme is combined with a network numerical Green's Function technique, applied to a network with stochastic water demands and then compared against the EPANET model and field observations. Both models achieve similar results at locations where turbulent flows prevail. However, the dispersion model provides better agreement with field observations at locations where laminar flows dominate.

Keywords: Drinking water quality modeling, water distribution networks, dispersion, contaminant transport, chlorine decay

1 Introduction

Because of physical, chemical and biological processes, water quality deteriorates in a water distribution system during transport between the points of treatment and consumption. Since it is not feasible to monitor water quality throughout a network, nor is it possible to sample all possible operation scenarios, water-quality engineers and system designers often employ computer-based mathematical models to predict the spatio-temporal distribution of constituents in water distribution networks. Such models can be used to analyze water quality degradation problems, assess alternative operational and control strategies for improving and maintaining water quality, design water-quality-sampling programs, optimize disinfection processes and evaluate the water quality aspects of distribution-network improvement projects.

Axial (or longitudinal) dispersion of fluid flowing through a pipe is defined as a mass transport process in which a solute spreads in the axial direction while moving downstream due to a non-uniform velocity distribution over the pipe's cross-section. Most distribution network water quality models including the widely used, public-domain EPANET [28] and commercial codes [37] apply a one-dimensional advection and reaction scheme (1D-AR) that neglects axial dispersion. These transport models perform satisfactorily when used for analyzing pipes in which turbulent flows dominate, but they tend to be less reliable when used for pipes in low flow zones, such as dead-end zones, which are common in municipal water-distribution systems. Depending on the time of day, low-flow conditions may predominate in 20-50% of a water distribution network having pronounced diurnal demand patterns [7].

In the presence of laminar flow, axial dispersion can be an important factor when predicting water quality. For example, the travel time of water through a distribution system is often assumed as nominal hydraulic residence time (volume/flow) or estimated as the residence time using a water-quality model like EPANET. In either case, the computed travel time does not take dispersion into account. However, the fact is that a part of a reactive disinfectant will move further along the supply pipe under conditions of advection-dispersion (A-D) transport than it does under pure advection transport conditions. Similarly, under A-D transport some fraction of the disinfectant will *not* move as far along the supply pipe as expected in pure advection. These differences in residence times could have ramifications when the supply system requires expeditious and accurate disinfection. In addition, when dispersive transport is neglected, utilities may over-dose the disinfectant to achieve certain residual concentrations at remote points in distribution network. Using dispersion transport model, on the other hand, may help utilities to fine-tune their disinfectant doses.

For these reasons, the next generation of network water quality models should account for axial dispersion. This paper presents the summary of recent research on the subject of axial dispersion in water distribution networks.

2 Background

Classical works on axial dispersion, conducted five decades ago (Taylor's theory [29]), laid the foundations that have been examined ever since and applied to chemical and industrial processes for laminar and turbulent flows. However, these theoretical and experimental findings have not been integrated into water quality models for pressurized water distribution systems. Transport of a conservative chemical tracer moving in steady laminar flow through a pipe is described by the two-dimensional advection-diffusion equation:

$$\frac{\partial C}{\partial t} = D\left(\frac{\partial^2 C}{\partial r^2} + \frac{1}{r}\frac{\partial C}{\partial r} + \frac{\partial^2 C}{\partial x^2}\right) - 2U\left(1 - \frac{r^2}{a^2}\right)\frac{\partial C}{\partial x} - KC \tag{1}$$

where: $C=C(r, x, t)$ is the solute concentration at any point in the cross section, D is the molecular diffusion coefficient (diffusivity) of the solute in water, U is the mean velocity in the axial direction, K is the first-order reaction rate constant for bulk decay (no wall decay is considered), a is the pipe radius, r is the radial position, x is the axial position, and t is time.

According to the Taylor's classical theory for dispersion, Eq. (1) can be simplified to the one-dimensional advection-dispersion equation, provided a certain initialization period has elapsed:

$$\frac{\partial C}{\partial t} + U\frac{\partial C}{\partial x} = E\frac{\partial^2 C}{\partial x^2} - KC \tag{2}$$

where: C is now the average concentration in the cross section and E is the axial dispersion coefficient, considered as constant. A dispersion process with a constant dispersion coefficient is known as *steady dispersion*. Expressed in dimensionless time T, the initialization condition is $0<T<0.5$, where $T= Dt/a^2$. Because of the small value of the diffusivity (D=10^{-5} cm^2/s for water) and the constantly changing flow in water distribution networks, the dispersion process is always in the initialization period, i.e., it is *unsteady dispersion,* and Taylor's theory is not valid. Accordingly, Gill and Sankarasubramanian (G/S, [14]) extended Taylor's theory and obtained an exact dimensionless expression for the unsteady instantaneous rate of dispersion in steady laminar flow.

A few previous studies of mass transport in single pipes under steady flow conditions indicate that axial dispersion is an important factor in laminar flow zones. Using analytical solutions of one-dimensional models in a steady flow, Axworthy and Karney [4] studied water quality under low-velocity and high-dispersion flow in a water distribution system. They concluded that the advective transport model with a continuous step input of solute significantly underestimates the nodal concentrations obtained by solving the advection-dispersion equation under low flow velocity. Their research implies that water quality models should incorporate dispersive transport in low-flow pipes.

Starting with a two-dimensional model and laboratory experiments that stressed the influence of dispersion, Ozdemir and Ger [22, 23] used a single equation to express bulk decay, radial diffusion and the subsequent pipe-wall reaction of chlorine and then converted the equation into a one-dimensional model. In subsequent studies, Ozdemir and Ucak [24] and Ucak and Ozdemir [36] embedded that equation into a computer program designed to analyze dynamic water quality in drinking water distribution networks.

In formulating a model for steady-state dispersion, Biswas *et al.* [6] argued that radial diffusion is the only important dispersion mechanism for chlorine concentration decay, and they compared it to field data.

One of the first attempts to numerically model the dispersion in water distribution networks is presented by Islam and Chaudhry [15]. However, their work did not consider unsteady dispersion and dispersion between pipes at network junctions.

Basha and Malaeb [5] applied an Eulerian – Lagrangian method for simulating the advection-dispersion-reaction process of constituent transport in water networks. In their study, the dispersion term in the governing equation is approximated using finite differences, and the resulting first-order partial differential equation is integrated using the method of characteristics. However, their model neglects the dispersion effects present between pipes at network junctions, which may lead to inaccurate results in networks where such effects are significant.

3 Recent Advances

3.1 Dispersion in Steady and Random Intermittent Laminar Flow

Since the exact G/S expression [14] for the time dependent dispersion coefficient is somewhat cumbersome to use, Lee [16] derived the following, well-fit approximation to the G/S equation with a correlation that exceeds 99.9%:

$$E(t) = E^{*}\left[1 - \exp(-\frac{t}{\tau_0})\right] \tag{3}$$

where: $\tau_0 = a^2/16D$ is a Lagrangian time scale reflecting molecular diffusivity D across the pipe radius a. Lee [16] also presents an expression for estimating the time-averaged dispersion coefficient in steady flows as

$$\overline{E(T)} = \beta(T)\, E_T \tag{4}$$

where: $\beta(T) = 1 - \frac{1-\exp(-16T)}{16T}$ and E_T is the Taylor dispersion coefficient in a steady flow [29].

Equation (2) is restricted to steady flow; however, flow in a water-distribution system fluctuates over time due to changing consumer demands. Because of the sporadic and stochastic nature of water demands, the flow (besides laminar) in water distribution pipes at the network periphery is often intermittent [7, 8, 11], giving rise to an even stronger dispersion transport. As a starting point (Buchberger *et al.* [8]), analyzed dispersion in intermittent laminar flow under the assumption of achieving an instantaneous laminar velocity profile during the periods when flow occurs, and concluded that the value of the time-averaged dispersion coefficient in intermittent laminar flow is larger than that given by Taylor's formulae for dispersion in steady laminar flow. Later, using principles from the linear systems theory Lee [16] studied unsteady dispersion in random, intermittent laminar flow and derived explicit expressions for the instantaneous rate of dispersion $E(t)$ resulting from an arbitrary sequence of laminar flow pulses. The study reveals that $E(t)$ is the sum of two factors: the dispersion memory derived from previous pulses and the nonlinear excitation derived from the current pulse. During periods of stagnation, active dispersion ceases and the memory of previous dispersion decays exponentially.

Based on the Eq. (4) (Li *et al.* [18]) estimated the spatially averaged dispersion coefficient using the travel time in a pipe with laminar flow as:

$$E_i\left(t^n\right)=\frac{\left[aU_i\left(t^n\right)\right]^2}{48D}\left\{1-\left(\frac{\tau_0}{t_i^r}\right)\left[1-\exp\left(-\frac{t_i^r}{\tau_0}\right)\right]\right\} \tag{5}$$

where: t_n is current time at time step n; $U_i\left(t^n\right)$ is averaged flow velocity in pipe i within time step n; t_i^r is the travel time in pipe i with flow velocity $U_i\left(t^n\right)$.

Buchberger and Lee [9] proposed a simple analytical method for estimating dispersion in steady and unsteady laminar flow with no initialization time. Using a Lagrangian dispersion model to simulate water quality and the dispersion predicted by that analytical method, they suggested a method for integrating dispersive transport phenomena into water-quality models.

3.2 Assessment of the Importance of Dispersion in Water Distribution Network Modeling

Using analytical solutions for a non-conservative solute, Lee [16] suggested that dispersion is important in laminar flows when the dimensionless group, $\mathrm{KE}/\mathrm{U}^2 > 0.11$ and proved that the advection-reaction (AR) model significantly under-predicts the non-conservative constituent concentration.

Through theoretical and numerical approaches, Li and co-workers [17, 20] analyzed the relative importance of the three basic mechanisms (advection, dispersion and reaction) with respect to the overall transport process in laminar flow. The significance of longitudinal dispersion in conjunction with a step input of reactive solutes in steady flows was first investigated analytically and compared theoretically in order to obtain the relative importance of the terms in one-dimensional governing equation under different conditions.

To further study the conditions under which dispersion becomes important for accurate water quality modeling, numerical solutions for a two-dimensional advection-diffusion-reaction (2D-ADR) model and a 1D-AR model were then used to investigate and to corroborate dispersive behaviors with unsteady flows or unsteady solute source profiles [17, 20]. The differences between the water quality results obtained by the 2D-ADR model and those obtained by the 1D-AR model were identified. It was demonstrated that mass dispersion is an important factor of a conservative solute with an instantaneous injection source or a sinusoidal injection source, but it may not be important to the regions not affected by the concentration front for the linearly increasing source and the step injection source. These results are governed by the potential driver of dispersion, the second-order derivatives in Eq. (2), which are greater than zero for instantaneous injection and sinusoidal injection but approach zero for regions unaffected by concentration front of the other two injection sources.

The effects produced by time scales, pipe sizes, flow velocities, water-demand pulse, arrival rate and the time history of flows on unsteady dispersion were also analyzed [17, 20]. It was concluded that the importance of dispersion in water quality modeling increases as pipe diameter increases for laminar dominated flows. Dispersion in water quality modeling plays a more important role for conservative contaminants. With an increase in reaction rate, dispersion becomes relatively less important for unsteady solute sources. Flow patterns and time scale do not appear to produce any obvious effect on the importance of dispersion. However, the time-averaged dispersion decreases greatly with the occasional bursts of turbulent or transitional flows, which most likely occur with small time scales. Therefore, a large time scale may lead to an overestimation of the dispersion rate because it masks the turbulent or transitional flow regimes in low flow zones

3.3 Numerical Methods

Numerical solutions for the 2D-ADR Eq. (1) in steady and random intermittent and unsteady laminar flow are presented by Buchberger and Li [10], Lee [16], Li [20] and Li *et al.* [17]. A Lagrangian-Eulerian numerical scheme was implemented by splitting Eq. (1) into two equations, one for molecular diffusion and another for advection. Each advected point retains its radial position but is translated downstream by an amount equal to the travel distance, which is computed as the product of the flow velocity and the time step. Radial and axial diffusion are computed with an Eulerian approach. In intermittent flow, during a busy period, only radial diffusion is considered using a Crank-Nicholson scheme, because axial diffusion is negligible compared to advection. During idle times, both axial and radial diffusion are simulated by applying an unconditionally stable alternating-direction implicit scheme and using two half steps to advance one full time step.

Because large water distribution systems often contain hundreds of dead-end stems serving thousands of consumers, it is impractical to use a 2-D transport model to simulate water quality in all the branching pipes of a municipal network considering the massive computational cost. Consequently, most network water quality models simulate solute transport with pipe hydraulics based on 1-D flow. Furthermore, to promote compatibility with existing models, the approximation for unsteady laminar dispersion is merged with a 1-D transport model in order to obtain an equivalent dispersion coefficient E for intermittent unsteady laminar flow [16], [17], [20].

In a piping network, Eq. (2) applies for each pipe, and the following boundary conditions hold at the network nodes:

a) At some nodes, seen as constituent sources, the concentration C is prescribed.

b) Mixing at the network nodes: a complete mixing has long been assumed in the network models, although recent works clearly demonstrated that this assumption needs to be revised [3], [25]. Recent studies [3], [25] show that pipes with different flow and constituent concentration may convey flow to a four way junction including a cross, a double-tee, or a tee-wye combination. Constituents are *mixed* at the node and new concentration values are obtained based on the ratios of flow rates in upstream and downstream pipes.

c) Mass conservation at the network nodes: Tzatchkov *et al.* [32], by generalizing the derived Eq. (2) to include the case of a junction at which several pipes meet, obtained the following nodal equivalent of Eq. (2):

$$\sum_{j=1}^{m}\left(\frac{dx_j}{2}A_j\right)\frac{\partial C}{\partial t}=\sum_{j=1}^{m}\left(A_j D_j\frac{\partial C}{\partial x}+Q_j C-K_j C A_j\frac{dx_j}{2}\right)-q_i C \tag{6}$$

where: m = number of pipes connected to the node; A_j = cross sectional area of pipe j; Q_j = flow rate in pipe j; D_j = dispersion coefficient of pipe j; K_j = first order decay constant of pipe j; and q_i = flow rate abstracted at the node. For the cases in which two pipes of equal characteristics meet at a node and q_i=0, Eq. (6) reduces to Eq. (2) if dx_j and dt tend to be infinitesimally small.

The numerical solution of the 1D-ADR equation in networks poses three main problems:

a) Boundary conditions at those nodes that are common to several domains have to be formulated and considered;

b) The direct application of the numerical schemes produces large, non-banded, asymmetric and unstructured systems of equations to be solved, especially when the network is large;

c) The difficulty increases when advection dominates over dispersion. Sharp concentration gradients are expected in this case, and an extremely fine discretization would be needed if Eulerian methods were to be applied. Because of the small values of the dispersion coefficient, contaminant transport in water distribution networks falls exactly in this category of advection-dominated problems.

To solve the advection-dispersion-reaction equation in pipe networks, Tzatchkov *et al.* [32] developed a numerical solution based on the domain decomposition strategy used to efficiently solve the resulting finite difference equations. A two-stage Eulerian-Lagrangian numerical scheme is applied [1]. Eq. (2) is split into two parts (an advective part and a dispersive part) and then numerically solved in each time step in two stages. In the first (Lagrangian) stage, the advective (or advective-reactive) part is solved for each pipe. The backward method of characteristics is used. In the Eulerian stage, an implicit numerical scheme is used, leading to a system of linear equations. To achieve computational efficiency in solving this system, a special technique that employs numerically computed Green's functions within each pipe is proposed [2]. In each pipe, the sought solution is represented by the superposition of three numerically obtained auxiliary solutions: a homogeneous (zero boundary conditions) solution and two Green function solutions (one for each reach end) multiplied by the unknown values of the constituent concentration at the two reach ends. To obtain the Green functions that correspond to each reach end, a unit value for the concentration is imposed at one boundary and a value of zero at the other, and the resulting tridiagonal system is numerically solved. The fluxes at each of the pipe ends are expressed in terms of the values of the concentration at each end, and continuity balance relations are used to construct a system of linear equations for finding the values of the unknown quantities at the network nodes. Thus, the large system of equations that represents the discretized network is decomposed exactly into three easy-to-solve tridiagonal systems for each pipe, and one low-order system to represent the concentration at the pipe junctions. This method can be applied to any type of network, branched or looped, and to advection-dominated and dispersion-dominated transport phenomena in order to handle a broad range of flow velocities that can be met in real distribution networks. More details can be found in [1], [30], [31], [32], [34], [35].

Tzatchkov *et al.* [33] extended the proposed advection-dispersion-reaction numerical model to include dispersion in intermittent laminar flow and variation of the dispersion coefficient during the initialization period. Li [20], Li *et al.* [18] and Li *et al.* [19] extended the method proposed by Tzatchkov *et al.* [32] to create a more complete computer model, ADRNET. The EPANET toolkit network hydraulic functions are incorporated and used as the hydraulic engine in order to simulate the extended period pipe flows in ADRNET based on time-averaged stochastic demands. The improvements made to the model are as follows: a) for better computational stability, a fully implicit difference format has been adopted to replace the Crank-Nicholson format; b) to achieve a more reasonable representation of network conditions, improved techniques for estimating the spatially-averaged dispersion coefficient have been utilized based on Lee' work [16]; and c) to more accurately simulate network water quality, stochastic water demands have been incorporated and examined.

3.4 Experimental and CFD Work

Cutter [12] conducted dispersion coefficient estimation experiments with a 15 cm diameter pipe. His results showed, as expected, that the dispersion coefficient increases with an increasing Reynolds number for Re≤2400 in laminar flow zone but decreases linearly with an increasing Reynolds number (2400<Re<4000) in transitional flow zone. In a turbulent flow zone, however, dispersion coefficients maintain a much lower level compared to those in laminar and transitional flow zones.

Romero-Gomez *et al.* [26] conducted a series of experiments in an effort to compare the empirical results of an axial dispersion of a non-reactive tracer in a pipe under laminar and transitional flow conditions with the modeling outcomes obtained from EPANET, Computational Fluid Dynamics (CFD), and the 1D Advection-Dispersion (AD) model. The experimental setup was constructed at the Water Distribution Network Laboratory at the Water Village, an experimental facility at the University of Arizona, Tucson, AZ, U.S.A., and consisted of a 10 m-long PVC pipe with a 15.3 mm inner diameter (1/2 inch nominal diameter), mounted on a metal scaffolding. Tap water, pumped from a storage tank into the pipe, constituted the main water source, whereas a micro-pump was used to inject water taken from a beaker and containing a tracer (sodium chloride). The flow rate was both controlled and monitored by means of turbine type flow-rate sensors. The flow rate was controlled using pump controllers and the needle valve at the downstream end of the pipe. Tracer concentration was monitored with electrical conductivity sensors. Two 4-ring potentiometric electrical conductivity probes and transmitters were placed at the upstream and downstream monitoring locations, 7.84 m apart, to measure tracer concentrations. Flow rate and concentration were observed in real time and recorded every second using a data logger.

The CFD simulations of the species transport in the pipe were carried out using FLUENT [13]. Two-dimensional, axi-symmetric, unsteady-state simulations were performed. The geometric extents of the circular pipe were H = 0.008 m x L = 1.6 m, both numbers representing a 2D computational domain axi-symmetric about the x-axis. Therefore, a 100D-long pipe was simulated. A quadrilateral mesh was defined with 30,480 cells, of which 10,160 cells (8x1270) belonged to the boundary layer region that was defined at the pipe walls. The following boundary types were set at the domain edges: velocity inlet (left), outflow (right), wall (top), and axis (bottom). The material was set as a mixture of water and sodium chloride. The boundary conditions (BCs) at the inlet were obtained from the experimental readings of flow rate (used for velocity BC) and upstream concentration (used for species BC). Because the upstream concentrations are transient, a time dependent profile was created for the latter. The species transport equation, which was added to the solver, was taken to be uncoupled from the flow calculations.

CFD simulation results for laminar flows were in excellent agreement with the experimental data. Two distinct characteristics were observed: (i) the experimental and CFD-simulated maximum concentration at the downstream location is lower than those based on the "plug flow" for all the cases, and (ii) the downstream breakthrough time of the "plug flow" profile is always delayed as compared to the experimental and CFD-simulated time and this difference becomes shorter with higher Reynolds numbers. Thus, axial dispersion of a solute can be an important transport process in laminar and transitional flow regimes. The actual rate of dispersion can be estimated by using the method of moments and experimental tracer data obtained under any flow regime. The magnitude of the dispersion coefficient dropped quickly when the flow leaves the laminar regime and enters the transition regime, consistent with available theory and previous experimental results by Cutter [12].

3.5 Field Validation and Applications

Tzatchkov *et al.* [32] and Li *et al.* [19] applied advection-dispersion-reaction models to simulate the fluoride and chlorine transport in the Cherry Hill Brushy Plains service area network, for which a series of field measurements was carried out by the EPA in order to compare the observed concentration with the predictions made by the EPANET model [27]. In those network pipes with medium and high flow velocities, the two models give similar results. In pipes with low flow velocities the measured concentration evolution is more closely represented by the proposed model than by the EPANET model [30], [32], [34]. The proposed model that considers dispersion appears to provide a substantial improvement over predictions by EPANET using advection-reaction model in low pipe flow zones of the network.

Nilsson [21] used numerical Monte Carlo experiments to simulate a deliberate bio-chemical assault on the Cherry Hill Brushy Plains water distribution system. The attack was modeled as a steady 6-hour injection delivering 3600 g of a conservative contaminant to a single node on the main line. Advection, dispersion and reaction were considered. The migration of the contaminant plume was tracked for 55 hours throughout the pipe network and the cumulative mass dose was computed at five target nodes. Combining the EPANET solver with a stochastic water demand generator the exercise was repeated for 1000 independent trials to establish a distribution of consumer dose exposures at the target nodes. A battery of simulation experiments was then performed to investigate how changes in system storage and demand patterns affect the baseline nodal dose loadings. Results for this case study show that the nodal dose distribution was extremely sensitive to the assumed system's operating conditions. When comparing different network configurations, the degree of system storage dominated the overall response. A relatively minor variability in water demands can lead to a broad range in the cumulative dose received at a particular node. In the case study, advective-dispersive transport did not have an appreciable effect on the total contaminant dose delivered to the target nodes. This could be due to the relatively small size of the network, the prolonged length of the attack, and/or the conservative contaminant modeled.

5 Conclusions

Solute dispersion is an important component of network water quality simulation, and it should be incorporated into the next generation of distribution network water quality models. The work presented in this paper should improve our fundamental understanding of solute transport and enhance our ability to model and predict water quality in municipal distribution systems. Improved water quality models capable of achieving accurate spatio-temporal axial dispersion patterns will be critical to efforts aimed at optimizing water quality sensor placement, assessing models for early warning systems, and generating the exposure information needed for quantitative risk assessment. Progress has been made, but the spatial evolution process of dispersion requires further research in order to determine the mass dispersion coefficient in network pipes under unsteady flow.

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