Ninth International Symposium on Water Management and Hydraulic Engineering

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Edited by

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Content

Theme III **Design and Construction Works**

Theme IV **Sanitary Engineering and Sustainable Water Use**

Theme V **Surface and Groundwater Resources** (Including Floods and Droughts)

Preface

Preserving the quality of our water resources and serving the water requirements of the society are the main challenges of water managers and hydraulic engineers. To ensure that these goals are achieved in the short term as well as in the long term perspective, an integrated approach is needed. This approach has to consider economic, social and environmental objectives and has to be based on an active participation of the concerned people. It considers both, surface and ground waters and it is based on a basin wide approach. Some of these concepts are already part of the EU-framework directive on water and are now being implemented in the EU countries.

This conference tries to contribute to these goals by inviting experts from Central Europe, from countries which are already since a decade and even longer the members of the EU as well as countries which have joined recently the union and there are also countries, especially from the South-East Europe, which are not yet member states. The main idea is to exchange experiences in the field of water management and hydraulic engineering. This is especially important in Europe where major river basins are shared by several countries, obviously demanding for the development of a shared vision and an integrated approach. Hopefully, this exchange of ideas and the improvement in the professional contacts among the participants will result in the mid term in improved concepts and implementation plans.

This conference is the ninth event in the series of similar conferences which were started in 1976, as a bilateral activity between the faculties of the universities of Gdansk (Poland) and Zagreb (Croatia). Since 1998, participants from the Slovak University of Technology, University of St. Cyril and Methodius from Skoplje (Macedonia) and the BOKU-University of Natural Resources and Applied Life Sciences in Vienna (Austria) contribute regularly to this two-annual conference series.

Now, it is for the first time, that the former Department of Water Resources Management, Hydrology and Hydraulic Engineering (IWHW), which is now integrated into the Department of Water-Atmosphere-Environment at BOKU-University of Natural Resources and Applied Life Sciences is organising this conference from 4th to 7th of September 2005 in Ottenstein, in the Northern part of Austria. About 60 papers were received, from which about the half will be orally presented at the conference. In the proceedings, all the papers are listed in alphabetic order within specific topics. During the conference, the presentation of the papers will be organised in six thematic items which refer to

I. Integrated Water Resources Management
II. Hydraulic Engineering and Environmental Impacts
III. Design and Construction Works
IV. Sanitary Engineering and Sustainable Water Use
V. Surface and Groundwater Resources (Including Floods and Droughts)
VI. River Restoration Projects (Strategies and Experiences).

To manage all the tasks related to this conference, a scientific committee consisting of:

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was set up. Further, the support by Mrs. U. Stowasser (AQA GmbH, Vienna) and sponsoring by EVN Naturkraft GmbH & Co KG is highly appreciated.

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

Editors

H.P. Nachtnebel and C. J. Jugovic

Theme I

Integrated Water Resources Management

International Symposium on WATER MANAGEMENT and HYDRAULIC ENGINEERING
Ottenstein/Austria • 4th–7th September 2005
Paper №: **I.01**

The Role of Hydro Power Plants in Poland's Economy

Stefan Bednarczyk
Piotr Ksiazek

Abstract: After World War II, Poland emerged as a state in new territory and completely different economic and social conditions. It had a considerable influence on economy, including the development of hydropower engineering.
In the years 1946-1953 less than 300 hydro power plants, out of about 6200 ones, were rebuilt and actuated. However, they were small ones, as in 1954 there were only two big hydro power plants in Poland.
In the period 1956-1981 the state followed a restrictive policy on small hydro power plant development. Considerable financial outlays were made on building big hydro power plants. Construction costs were mainly covered by power engineering, yet industry, public utilities, flood control, forestry and recreation benefited from it.
Apart from classical hydro power plants, three big pumped-storage power plants were built in 1970 - 1982. Two of those plants play a significant role in the national energy system by covering morning and evening peak, as well as regulating frequency, emergency reserves and compensation work. The need for building a pumped-storage hydro plant resulted from Poland's energy production based on coal and lignite and unsuccessful plans of developing nuclear power.
Favorable conditions of developing small hydro power plants were created by the state after 1989.
Keywords: power, plants, hydro, Poland, pumped-storage

Introduction

After World War II, Poland emerged as a state reconstituted in new territory, and political, economic and social conditions were determined by post-war changes. The country's borders were moved west, and its overall area decreased by about 20%. As a result, relocation of the population followed. In the years 1945-1947 6 million people, that is 25% of the population, were removed from former Polish territory and resettled. The country was completely ravaged. Over half of towns were in ruins, and the war caused considerable damage to industry. Furthermore, Western and Northern Territories, former German lands incorporated into Poland after World War II, were stripped of machines and equipment. In addition, a great number of watermills, sawmills and electric power stations were totally destroyed. In 1947 565 hydro power plants, out of 6240 ones, were in operation in the Western and Northern Territory, and about 65 in the central part of Poland, with a capacity of 25 MW (the average capacity was 14 kW). The situation in Poland was incomparable with any other country which fought in World War II.

The state's energy policy in 1946 - 1956

In 1946 almost all the branches of industry were nationalized, including more than 90% of hydro power plants. The economy came under the control of the government, with the exception of 2/3 of privately owned family farms. The state's energy policy was aimed at assembling, rebuilding and developing the existing thermal and hydro plants, which was indispensable to meet the growing demand. A considerable progress was made, as it is illustrated in table No.1.

Table No.1

Year	1946	1947	1949	1953	1955	1956
Electric energy production in TWh	5,71	6,61	8,3	13,68	17,75	19,5

In that period investments were made in thermal energy. However, bigger hydro power plants with a few hundred kW capacity, including two major ones: Rożnów (50 MW) and Dychów (75MW), were rebuilt. Due to lack of replacement parts as well as wear and tear of electric power generation machines in the post-war decade, operating many small old hydro power plants was abandoned.

Hydropower engineering as an essential element of water conservation plan in 1957 - 1989

At the turn of the 1950s and 1960s, analysis of hydropower engineering in Poland was made, and prospects of hydroenergy development assessed.

Poland has meagre water resources. The flowing water reserves are estimated at the level of 1500-1800 m^3 per person. As it is a flat country, possibilities of hydroenergy development turned out to be faint. It was estimated that building water power plants would produce the energy of 12 billion kWh (12 TWh) : 4, 3 TWh in the Lower Vistula, about 3,5 TWh in mountainous and submountainous rivers, and the remaining ones in the other parts of the country accounting for 4,2 TWh. Yet, hydrotechnical construction costs to generate such electric energy appeared to be extremely high. Meanwhile, demand of industry, public utility companies and railways for electric energy turned ot to be much higher than the existing hydroenergy potential. Therefore, the state authorities put a lot of organizational and financial emphasis mainly on developing thermal power stations because of easy availability of coal in Poland. However, the role of hydroenergy was covering of peak power demands. That is why, priority was given to designing and building several storage reservoirs and the Lower Vistula development was undertaken. On the basis of those projects, 22 storage reservoirs with a capacity of 1020 mil m^3, apart from 6 existing ones of 195 mil m^3 capacity, were built on the Vistula and its tributaries. In the Oder river basin 12 reservoirs of 226 mil m^3 capacity were built in addition to 11 existing ones with a capacity of 257mil m^3. The projects of building huge water reservoirs were drawn up, including 9 dams on the Vistula and its tributaries, which could form reservoirs of 710mil m^3, and on the Oder tributaries of about 450mil m^3.

There were projects concerning the Vistula development because its hydroenergy potential is 1/3 of overall potential. Moreover, plans were made for building a waterway connecting Poland with the Soviet Union and later on with German waterways. If those projects were carried out, it could be assumed that there would not be dangerous floods in the river Vistula basin and they would be effectively limited in the Oder basin. However, in order to eliminate them completely, additional reservoirs and polders should be built. Yet, it is unfeasible owing to a dense infrastructure network. Unfortunately, those plans were not put into effect due to two main reasons:

1) in the 1970s financial resources envisaged to be spent on hydroenergy infrastructure were allocated for building pumped-storage power stations

2) considerable socio-economic disturbances in the 1980s made financing building expensive hydrotechnical plants impossible.

Material and social benefits from building reservoirs on Polish rivers

Hydroenergy development projects concerning Polish rivers brought in substantial benefits. 28 power plants, out of 128 ones, are storage and substorage power plants with a total capacity of 745 MW (5 biggest ones have a capacity of 584 MW).

The remaining power plants were built before 1939 and their total capacity amounts to approximately 140MW.

The incurred costs of reservoir infrastructure, with the exception of Niedzica, were reimbursed a long time ago from electric energy sale. At present electricity boards, which were granted property rights by the state, reap large profits.

Table No. 2

Power plant	River	Capacity (MW)
Solina	San	200
Włocławek	Wisła	162
Niedzica	Dunajec	92,5
Dychów	Bóbr	79,5
Rożnów	Dunajec	50

Energy and industrial storage reservoirs, built mainly on Carpathian tributaries of the Vistula and on the Vistula itself, contributed to flood relief a few times. They protected towns and housing estates in the Vistula valley from flooding at least three times (1958, 1977, 1997). The reservoirs on the Oder tributaries in the Sudeten effectively prevented floods several times. Yet, in July 1997, due to excessive precipitation, heavy flood inundated the region. The damage caused by the flood was enormous because of insufficient quantity and capacity of storage reservoirs and inadequate preparation of reservoirs for taking the flood water. Rivers and storage reservoirs supply the majority of Polish towns and almost all industry with water (Kraków, Łódż, Nysa and others). Storage reservoirs contribute to flow leveling, swell controlling and providing water to lower parts of channels when rainfall is very low, which effectively results in both supplying industry with water and limiting ecological damage to the river and its valley.

Building several energy reservoirs brought immense material and non-material advantages for local communities. Big dams, power plans and reservoirs are tourist attractions. Plenty of leisure and holiday centers were built at Zegrzyński Reservoir (the Dębe Hydro Power Plant), Solina and Koronowo Lakes and other ones.

Numerous housing estates and summer houses were build by well-off citizens of big cities. It resulted in developing local communities, creating new jobs and generating additional income. In the vicinity of storage reservoirs the level of underground water increased, which affected agriculture and forestry. Koronowski Lake and power plant derivation brought about stabilization of underground waters in the area of 3000 hectares. It contributed to increasing crops and timber growth in forests from 30% to 50%.

The construction and role of pumped-storage power stations

The assumptions concerning hydroenergy development from the 50s of the previous century became outdated in the mid-60s. Demand for electric energy increased significantly due to rapid development of undustry, communication, urban sprawl and population growth. It considerably influenced changes in energy policy and led to thermal energy development, which is shown in table No. 3

Table No. 3

Year	1960	1965	1970	1975	1980	1985	1989
Electric energy production (TWh)	29,31	43,80	64,53	97,2	121,9	137,71	145,47

The share of hydropower engineering constantly decreased to the level of 2% at the end of the 70s. Meanwhile, in the mid-60s peak power shortage occurred, which was exacerbated at the end of the 70s. In such a situation hydroenergy investments comprised building three new pumped-storage power plants subsequently, the data of which are presented in table No. 4.

Table No. 4

Power plant	Capacity	Year of actuating
Żydowo	150	1970
Żar-Porąbka	500	1980
Żarnowiec	716	1982

The above mentioned power plants were earmarked for taking excess energy in the night and covering morning and evening peak, as well as regulating frequency, emergency reserves and compensation work. Nowadays Żarnowiec and Żar-Porąbka play a significant role in the national energy system.

The influence of political changes on Polish hydroenergy

At the turn of 1988/90 radical changes in state economy took place. In the second half of the 1990s, after some turbulence, privatization of electricity boards followed. Some of them were taken over by foreign capital. At the same time many sectors of industry, mainly heavy industry, decreased production or simply closed down. In consequence, demand for electric energy declined and its production fell as it is shown in table No. 5

Table No. 5

Year	1990	1992	1994	1996	1998	2000	2002	2004
Total production (TWh)	136,3	132,7	135,3	143,2	142,8	145,2	144,1	154,1

The state and electricity boards stopped building new power plants, but in the second half of the 1990s some renovation work was carried out. It is characteristic that on the one hand electricity boards derive considerable financial profit from hydro power plants, and on the other hand, with few exceptions, oppose new investments in hydroenergy, which are so advantageous to community.

In 1981 and 2000 the government made some crucial decisions enabling individuals and private companies to invest in hydroenergy. In the years 1981-1990 about 100 small hydro power plants were rebuilt and actuated, and another 250 ones in 2000.It is worth mentioning that a small power plant in Poland is defined as a plant which has a capacity of less than 5,0MW. It is estimated that at present there are about 400 private hydro power plants. State and private small river hydro plants constantly increase production, which is presented in table No. 6.

Table No. 6

Year	1990	1992	1994	1996	1998	2000	2002	2004
Production of river hydroplants (GWh)	1472	1617	1783	2063	2532	2332	2767	2889

However, it is a small amount in relation to the commitments taken on by Poland's government in the European Union.

Conclusions and comments

1. It is noted with satisfaction that in the last two decades many small hydro power plants were revitalized and about 100 were built, which was thoroughly approved by society. They have a positive influence on the environment and integrate local communities.

2. It is to be regarded as unfortunate that the government abandoned building big hydroenergy plants with storage reservoirs, which are the most effective method of preventing floods. In addition, they regulate water levels in rivers, which has an advantageous influence on the environment, economy and society. It is caused not only by state budget deficit but also wrong approach to that matter.

3. Private Polish and foreign companies look for making a considerable profit easily and quickly. For that reason, they are neither interested in improving water conservation nor in investing in hydroenergy at present and in the future.

4. Hydroenergy development of the Lower Vistula is still a very controversial and disputable issue and stirs up emotions. There is so much opposition to it that one cannot expect the problems of the Lower Vistula to be resolved soon.

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Paper №: I.02

Regime Analysis of the River Sava Reach

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Neven Kuspilic

Abstract: The river Sava has large importance in water management of the Republic of Croatia. In the recent period, great attention has been given to the maintenance and navigability setting of the river, which is in conjunction with Croatian international obligations resulted from AGN agreement. The river Sava is navigable on its major reach, and future river regulation actions should contribute to the necessity of undisturbed inland navigation. On the other hand, regulation actions may not exceed the scope of the natural river characteristics. Morphological analysis according to the regime theory gives an overview of the river natural and evaluated project condition. The idea is to examine hydraulical and geometrical characteristics of the river channel. Such analysis was carried out for Jamena-Samac reach, with evident channel stability level.
Keywords: regime theory, inland navigation, river regulation, morphological analysis

Introduction

The river Sava is the largest Croatian river and has high importance in water management of the state. Croatian international obligations resulted from AGN agreement, have given large attention to the maintenance and navigability setting of the river. Current navigability condition is satisfactory on major Sava reach, but there are short reaches with insufficient channel depth and radius. As a part of inland waterway project, one of the proposed regulation actions is local instream dredging operation. Future depth corrections should contribute to the necessity of undisturbed navigability, but may not exceed the scope of the natural river characteristics.

Due to the variation of geomorphological conditions along alluvial stream, there are reaches with lower and with higher morphological changes in longterm sense. Stable river reaches have accomplished dynamic stability, which implicates longterm steady state of its geometry and sediment transport. Channels that maintain a stable average form, engineers describe as “in regime”.

This paper gives stability analysis of 106.5 km long reach from Jamena to Samac, based on empirical regime concept. Natural river characteristics were examined and compared with evaluated future condition after local dredging operation. Analysed river reach from Jamena to Samac is shown in Figure 1.

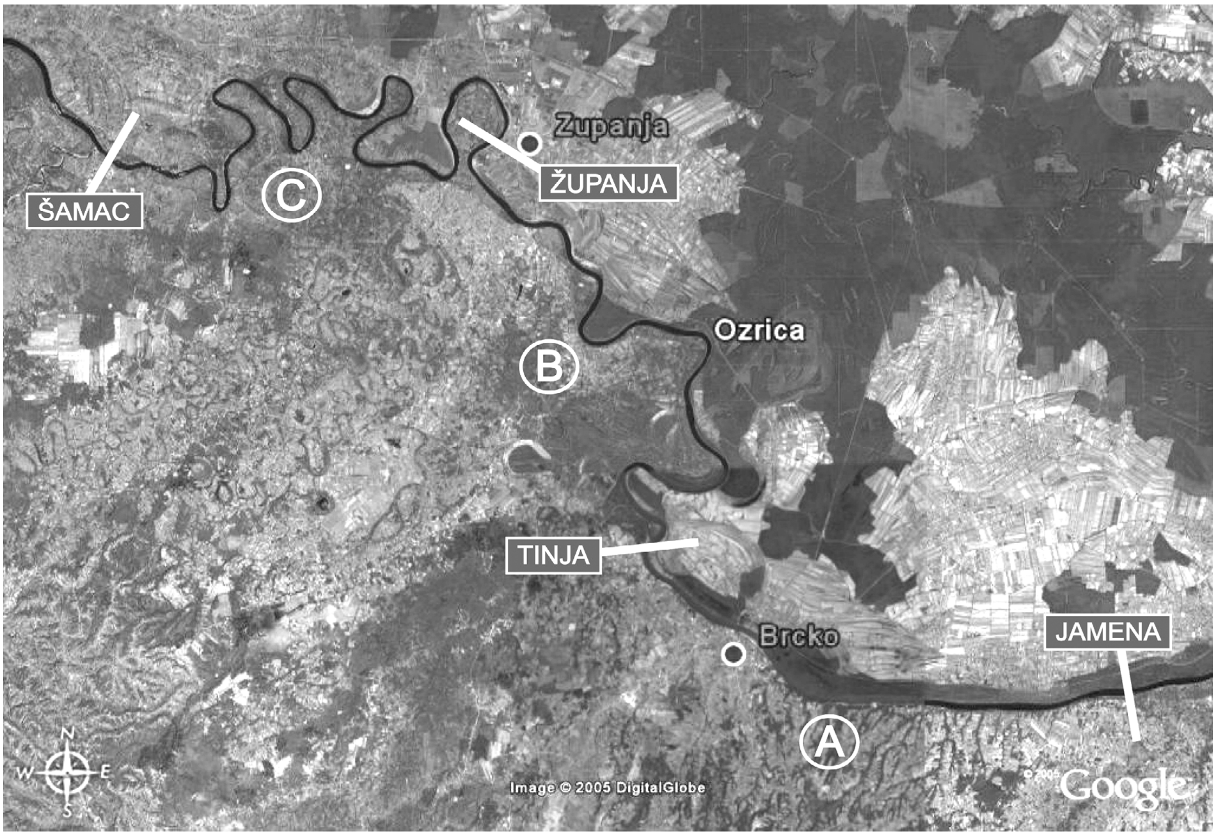

Figure 1 Analyzed river Sava reach from Jamena to Samac, length 105.6 km, flow is left to right.

Empirical regime concept

When hydraulic engineers describe the geometry of a river channel, they define the type of river (straight, meandering, braided, etc.), and the mean values of width (B), depth (d) and slope (S).

Empirical regime theory was first developed by British engineers working in the Indian subcontinent and was derived in a response to the problem of designing large irrigation canal systems. Consider a very long, straight open-channel excavated in a cohessionless alluvium; and suppose that at starting time a constant flow rate begins to flow in it. It is assumed that the flow is nearly bankfull, and that is transporting sediment. Experiment shows that, the flow rate will not “accept” the initial channel shape: it will gradually deform it, so as to establish a certain definite channel "of its own", which is referred to as the regime channel. Hydraulic geometry parameters: width, depth and slope of such regime channels were then related to the discharge.

More recently regime theory has been applied to natural rivers. Ignoring plan geometry, an alluvial river can adjust its hydraulic geometry to achieve a stable condition in which it can transport a certain amount of water and sediment. Thus, it has three degrees of freedom and the problem is to establish relationships that determine these three quantities of width, depth and slope.

Stability of planform

River planform may be classified according to channel pattern or channel type. Usually, there are three major channel patterns: strait (or sinuous), meandering, and braided. The braided channels have steeper slopes while meandering channel have smaller slopes. It is believed that several apparent thresholds exist between different river patterns.

To distinguish different pattern areas, river investigators used relationship between bankfull discharge (Q_b, [m^3/s]) and channel slope (S_0). Using data of sand-bed and gravel-bed streams, Leopold and Wolman (1957) obtained relation for the threshold separating meandering and steeper braided streams as:

$$S_0 = 0.0125 Q_b^{-0.44} \quad 3.1$$

Based on empirical investigation, Lane (1957) obtained the threshold equation for meandering channel as:

$$S_0 < 0.0007 Q_b^{-0.25} \quad 3.2$$

The analysed river Sava reach from Jamena to Samac is a part of the transfer zone (middle part) of the river Sava. Usually, the river channel in this part has the most stable geometry and its configuration is easy to define. Based on the river pattern, the whole reach can be divided into three sub-reaches A, B and C, as shown in Figure 1. Channel slope for the current and project condition was calculated among 520 cross-sections of the reach. After bankfull discharge had been adopted (see chapter 4.), slope threshold values were calculated and compared to the channel slope (Table 1.).

Table 1 Average channel slope for current and project condition.

	Bankfull discharge (Q_b) [m^3/s]	**Average channel slope (S_0) ×10^{-5}**		**Threshold channel slope ×10^{-5}**	
		Current	**Project**	**Leopold**	**Lane**
Reach A	2180	4.18	3.86	42.5	10.2
Reach B	2117	1.51	1.71	43.0	10.3
Reach C	1442	6.81	6.12	50.9	11.4

In order to define planform stability more sophisticated approach is required than just a view at the channel slope change. Future dredging works would make only local changes of the instream depth, leaving all other channel characteristics unchanged. Thus, in this case, insight only in channel slope change could be satisfactory for the analysis of planform stability.

If the river slope is close to a critical value, a small change in slope may lead to a large change in channel pattern. Channel slope data shows that natural channel slope on all reaches is beyond threshold values. Therefore, based on aerial overview and threshold values it can be concluded that the river Sava channel on Jamena-Samac reach has meandering pattern in generally stable condition.

Difference in channel slope between current and evaluated project condition is less then 12%. According to this magnitude of change and compared to threshold values it can be concluded that there would be no significant disturbances of natural channel pattern.

Channel-forming discharge

By contrast to the irrigation canals, alluvial river is unconstrained in developing its own geometry and has all its boundaries as a free surface. The range of discharge for irrigation canals is limited, so there is little inherent difficulty in deciding the discharge to be used in the regime relations. Natural rivers have a wide range of discharges and thus it is more difficult to know which discharge should be used in the morphological analysis.

There is no universally agreed method of determining the channel-forming (dominant) discharge for alluvial rivers. For downstream changes in channel geometry, the bankfull discharge is usually used as the channel-forming discharge. This simplified approach is justified in view of the fact that lower discharges, which move less sediment, contribute less to the channel formation. Discharge above a bankfull stage is largely absorbed by the broad flood plain and therefore it generally has less effect on the channel shape.

Leopold et al. (1964) found that the bankfull stage has a return period averaging 1.5 yr. Other investigators mainly found that bankfull discharge does not have a common recurrence, but that is usually greater than the mean annual discharge. Using 233 sets of data, Williams (1978) obtained the following equation for the bankfull discharge:

$$Q_b = 4.0\, A_f^{1.21}\, S^{0.28} \quad [\mathrm{m^3/s}] \tag{4.1}$$

Decision of the bankfull discharge value (Q_b) was made after results of hydraulic HEC-RAS model. Soundings of 520 cross-sections were made in 2003, as a part of inland navigation project. Hydrological data were used from the National Meteorological and Hydrological Service (DHMZ). Calibration of the model for mean annual discharge was made according to the measured data of water levels on the gauging station Jamena, Zupanja and Samac. For the gauging station Zupanja, continuous long-time measurements of water level and discharges exists, so a reliable discharge rating curve could have been established. Equations for the discharge rating curve are shown in Table 2. Datum for the gauging station Zupanja is H_0 = 76.28 [m.a.s.].

Table 2 Discharge rating curve for the gauging station Zupanja (DHMZ, 2001).

Water elevation [cm]	Discharge [m^3/s]
-100 <= H <= 100	$Q = 86{,}375(H + 2)^{1{,}326} + 100$
100 < H <= 1000	$Q = 57{,}336(H + 2{,}7)^{1{,}609}$

For bankfull discharges, Manning's roughness values where used from model results for the mean annual discharge. Several cases of bankfull discharges were examined for each of the three reaches. Since stages of the left and right bank are rapidly changing along the river reach, bankfull discharge value was resolved after water-surface elevation at each cross-section had been examined. Water-surface elevation at one cross-section of the reach A is shown in the Figure 2.

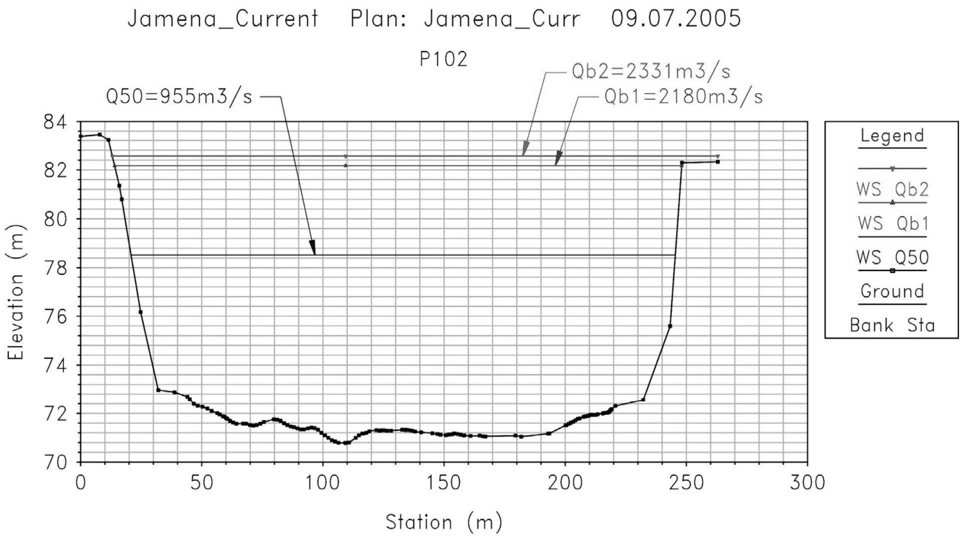

Figure 2 Water-surface elevation for several discharges on the cross-section P102 (Reach A).

For analyzed river Sava reach, mean annual and bankfull discharges were considered and compared. Adopted bankfull discharges are shown in Table 3. together with calculated values according to the Williams equation (4.1).

Table 3 Mean annual and bankfull discharges for Jamena-Samac reach.

Discharge	Mean annual (Q_{50})		Bankfull (Q_b)		Williams	
	[m^3/s]	×10^4 [ft^3/s]	[m^3/s]	×10^4 [ft^3/s]	[m^3/s]	×10^4 [ft^3/s]
Reach A	955	3.37	2180	7.70	2649	9.36
Reach B	927	3.27	2117	7.48	2520	8.90
Reach C	927	3.28	1442	5.09	2332	8.24

Bankfull discharges are also compared with data of Schumm and Carlson (after Chang, 1998) and plotted in Figure 3.

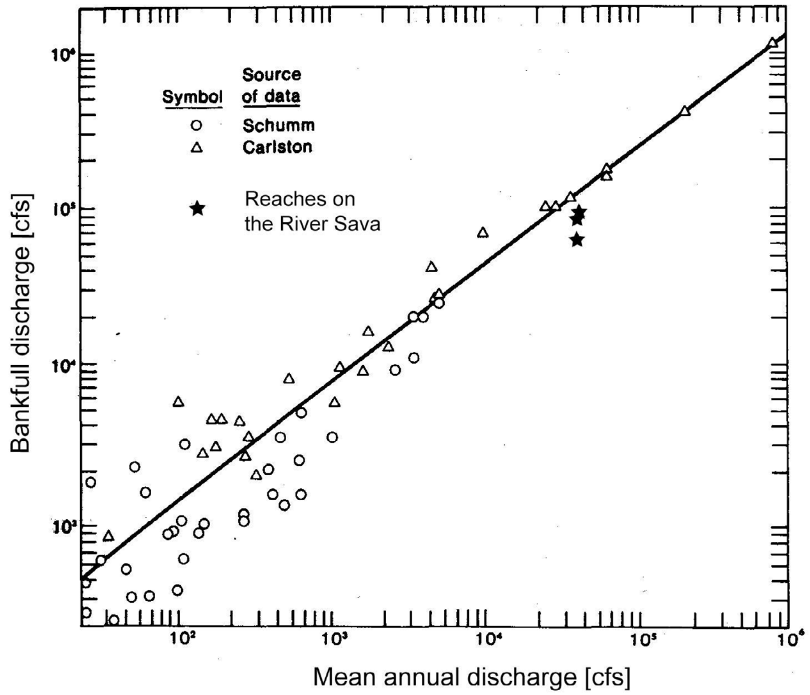

Figure 3 Relationship between bankfull discharge and mean annual discharge (after Chang 1998).

It can be seen that Williams equation gives about 20% higher values for reaches A and B, and 60% higher value for reach C. Comparison of adopted bankfull data to mean annual data also demonstrates that a bankfull discharge of the reach C has the largest difference.

Hydraulic geometry

Hydraulic geometry of river cross section is referred to surface width *B*, mean depth *d* and mean velocity *v*. The size of the channel (hydraulic geometry) is determined by the water flow through it and is changing with the discharge. Earlier studies by Leopold and Maddock (1953) and others have produced empirical relationship of gravel streams as a function of discharge. General form of the equations was:

$$B = a_1 \cdot Q^{b_1} \qquad d = a_2 \cdot Q^{b_2} \qquad v = a_3 \cdot Q^{b_3}$$

where a_1, a_2, a_3, b_1, b_2, b_3 are numerical constants. Since $Q = B\,d\,v$, it follows that $a_1 \cdot a_2 \cdot a_3 = 1$, and $b_1 + b_2 + b_3 = 1$.

The variations of *B*, *d* and *v* with *Q* depend on the cross-sectional geometry and are affected by the river pattern. For this reason, the numerical constants in the foregoing equations are varying from one stream to the other. Nevertheless, respective exponents among regime investigators fall into these ranges:

b_1: 0.39-0.60 b_2: 0.29-0.40 b_3: 0.09-0.28

For analysed river Sava reach, hydraulic geometry values for current and project condition were obtained from HEC-RAS model results. For the adopted bankfull discharges, calculated mean geometry values are shown in Table 4.

Table 4 Mean hydraulic geometry values for reaches A, B and C for current and project condition.

	Discharge	**Current condition**				**Project condition**			
	Q_b	**B_{av}**	**d_{av}**	**v_{av}**	**S_{av}**	**B_{av}**	**d_{av}**	**v_{av}**	**S_{av}**
	[m^3/s]	**[m]**	**[m]**	**[m]**	**[m/m]**	**[m]**	**[m]**	**[m]**	**[m/m]**
Reach A	2180	290	8.96	0.86	2.21E-05	290	8.90	0.87	2.28E-05
Reach B	2117	283	9.07	0.86	1.99E-05	281	9.02	0.87	2.07E-05
Reach C	1442	249	7.97	0.77	4.81E-05	245	7.85	0.79	4.91E-05

Based on comparison of depth changes, it can be seen that the dredging works would decrease depth on reaches A and C for 10%, and would decrease mean depth on reach B for 0.5%. Decrease of depth on reaches A, B and C is followed with increase in mean velocity, but with small magnitude. Bankfull width would preserve its current condition.

Three bankfull discharge values were insufficient to establish a reliable relation of hydraulic geometry and discharge. Therefore, values of exponents for width (b_1) and depth (b_2) should have been adopted from other regime river data. Values of channel width and depth for current river condition are compared to data from several gravel-bed channels after Calow and Petts (1992), as shown in Figure 4.

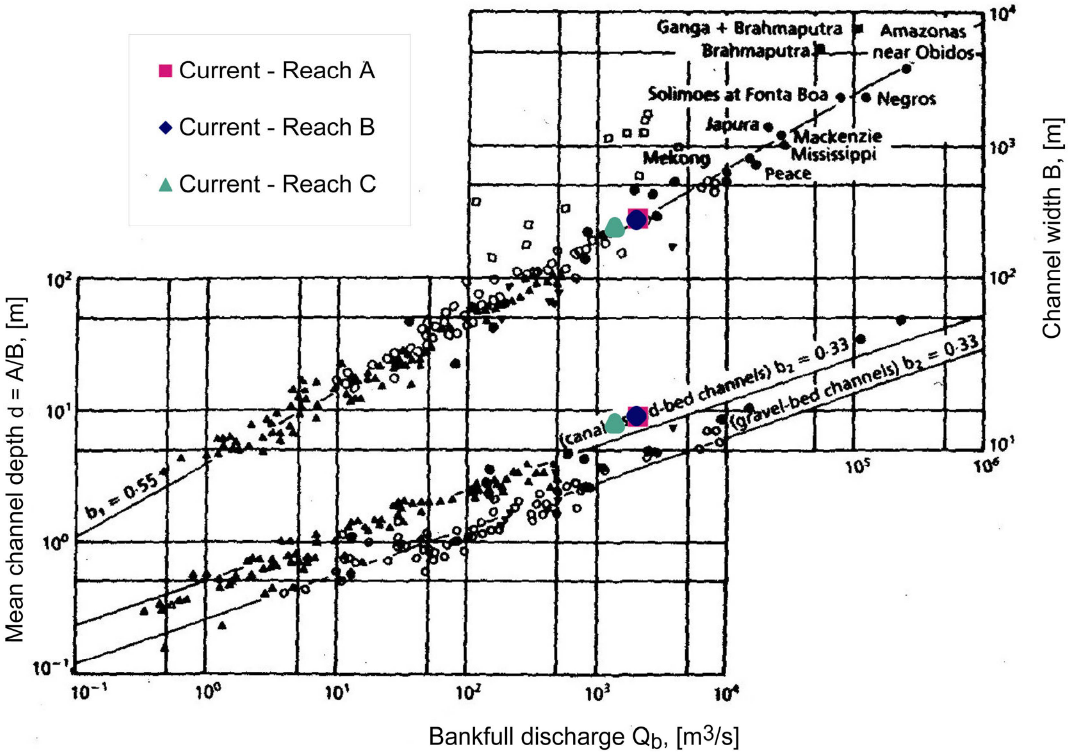

Figure 4 Scale relations for channel width and depth versus flow for current Sava condition compared to the data of Calow and Petts (1992).

It can be seen that Sava mean channel widths are in good correlation to other regime channels. But channel depth for all reaches is approximately 3.0 meters higher. Higher channel depths are in conjunction with conducted instream gravel mining in the last several years. Comparison with other gravel-bed rivers implicates that hydraulic geometry exponents for width b_1=0.55 and depth b_2=0.33 for Jamena-Samac reach can be adopted.

Summary

According to the aerial photographs and analysis of channel slope, Jamena-Zupanja reach has a stable meandering planform. In the analysis, bankfull discharge is adopted as a channel forming discharge after results of hydraulical model. Relation of mean annual to bankfull discharge, as well as Williams equation, implicates that sub-reach C has less stable form than other reaches, which is in conjunction to formation of barns in bends on this reach.

The regime method relies on available data and attempts to establish relationships that determine three quantities of width, depth and slope. Compared with other gravel-bed streams hydraulic geometry exponents for width b_1=0.55 and depth b_2=0.33 for analysed reach could be adopted.

As the channel slope of current and project condition is beyond the threshold values, it can be concluded that evaluated project condition would not make large disturbances of the river natural pattern.

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Charakteristics of the Lakes in Radunia River Catchment According to the EU Water Framework Directive

Teresa Jarzębińska

Abstract: Radunia River is the tributary of Motława which flows into Vistula River. The length of Radunia is 104.5 km and the catchment area 861 km^2. Catchment area of Radunia River is characterized by many lakes and rivers. Essential parts of the catchment consist of agricultural land (52%) and forests (38%). The main functions of the catchment area are agriculture, tourism, and recreation.

There are 68 lakes in Radunia River catchment. Their total area is 34.4 km^2, corresponding to 4% of the whole basin. Many lakes have been physically modified over the years. In 1908 - 1937 8 hydraulic power plants were constructed and put into operation on Radunia River. To increase the energy potential of Radunia River 3 control weirs were installed. Their aim was to control the outflow from the lakes. Apart from the production of electric energy, the water of Radunia River and its tributaries were widely used for propelling of turbines in the mills. Recently appeared a very important and new user of water from Radunia River i.e. water intake for the City of Gdańsk.

According to the EU WFD Radunia River catchment was divided in to 12 water bodies. In the paper the main problems of the water resources management in the catchment are presented. The typology, reference conditions and establishment of objectives for the lakes are also described.

Keywords: EU WFD, water bodies, typology of the lakes, water resources management

1. Introduction

The Water Framework Directive of the European Union (EU WFD) was passed on 23 October 2000 and entered into force on 22 December 2000. Three years later, i.e. on 22 December 2003, Member States should implement this Directive in their national legislation. Poland entered into the EU on 1 May 2004. From this moment we are also obliged to adapt our laws to the recommendations of the EU WFD. In the same time started the Grant No KBN Nr PBZ–KBN–061/T07/2001 of the Committee of the Scientific Research "Methodical basis of the national project of the integrated development of the water resources management in Poland". In the frame of the study it was decided to carry out pilot projects in a number of selected Polish river basins. One of them was the Radunia River catchment. This project was carried out by the Institute of Hydro-engineering of the Polish Academy of the Sciences and Gdańsk University of Technology.

The paper present the results of the studies concerning the characteristics and typology of the lakes existing in this catchment.

2. Radunia River

The source of Radunia River is at the elevation 165 m above sea level few kilometers south from Lake Stężyckie. Then it flows through the chain of 7 lakes of glacial origin. These are: L. Stężyckie, L. Raduńskie Górne, L. Raduńskie Dolne, L. Kłodno, L. Brodno Małe, L. Brodno Duże and L. Ostrzyckie. Water surface elevation of first three lakes is controlled by the weir Chmielonek. Next 3 lakes are controlled by the weir Brodnica Dolna and finally the outflow of Radunia from L. Ostrzyckie is controlled by weir Ostrzyce. Water level variation in these lakes ranges from 162.30 m to 159.37 m. Total water surface area of these lakes is 33 km^2 and their volume about $200\times10^6 m^3$ which gives important retention capacity and thus secures the uniform discharge of the river over the whole year. Downstream from the control weir Ostrzyce, Radunia flows through a small lake Trzebno. Along the next 13 km the river has mountain character (slope 2.2‰) and then flows into reservoir Rutki created by the dam. The volume of the reservoir is $0.3\times10^6 m^3$ and operational head 12.0 m. Average discharge in this cross-section is 3.2 m^3/s and maximum discharge 18.8 m^3/s. Further downstream (3.5 km) is the mill Żukowo operating on the head 2.75 m. In the km 41.0 there is another regulating weir Lniska with the head 2.50 m. In km 31.0 there is reservoir Łapino of the volume 1.55×10^6 m^3 and hydraulic power plant using the head of 13.8 m. Power scheme Bielkowo consists of a small control weir, diversion channel, 2 reservoirs, concrete power shaft, steel penstock and power plant operating on the head of 44.25 m. This high head was obtained by the utilization of river section of high slope forming a large loop and water diversion. Immediately downstream there is the largest reservoir Straszyn (3.4×10^6 m^3) with hydraulic power plant and at present water supply for the city of Gdańsk. Next there is a chain of three run-off-river power plants with reservoirs: Prędzieszyn, Kuźnice and Juszkowo. Control weir (km 12.0) Pruszcz Gdański begins the Radunia Channel which initially flows parallel to the Old Radunia River. The last hydraulic power-plant Pruszcz Gdański is already situated on Radunia Channel. The Old Radunia River flows through Żuławy Gdańskie and discharges into Motława River. Radunia Channel which runs at the foot of high plateau joins Motława River in the center of Gdańsk. It was constructed in XIV century to supply water for mills and defensive moats in Gdańsk. Channel discharge amounts to 7 m^3/s. The Old Radunia River flows within flood dykes, has no tributaries and has practically transition character for flood discharges. Total length of Radunia River from its source to the discharge of the Old Radunia into Motława River is 104.5 km.

S – sources of Radunia River

1 – L. Stężyckie
2 – L. Raduńskie Górne
3 – L. Raduńskie Dolne
4 – weir Chmielonek
5 – L. Kłodno
6 – L. Brodno Małe
7 – L. Brodno Duże
8 – weir Brodnica Dolna
9 – L. Ostrzyckie
10 – weir Ostrzyce
11 – L. Trzebno
12 – power plant Rutki
13 – weir Żukowo
14 –River Mała Supina
15 –River Strzelenka
16 – weir Lniska
17 – power plant Łapino
18 – weir Kolbudy
19 – River Reknica
20 – power plant Bielkowo
21 – power plant Straszyn
22 – power plant Prędzieszyn
23 – power plant Kuźnice
24 – power plant Juszkowo
25 – power plant Pruszcz Gdański
26 – Radunia Channel
27 – Radunia River
28 – River Motława
29 – Martwa Vistula

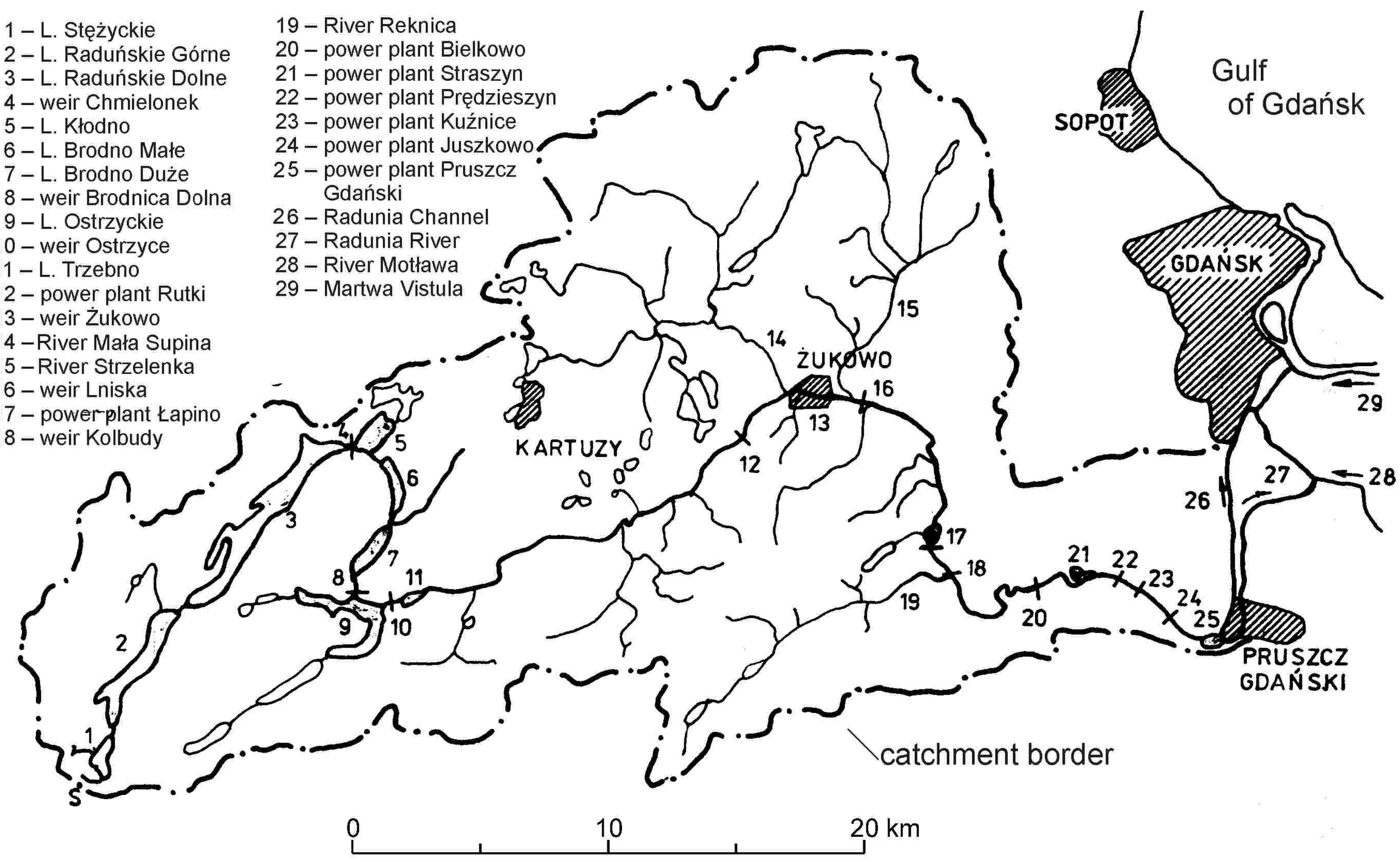

Fig.1 Radunia River and its catchment

3. Catchment of Radunia River

Total catchment area of Radunia River is 861 km^2 which includes also the catchment of Old Radunia River from Pruszcz Gdański to the inflow into Motława. This area amounts to 24.8 km^2. The catchment of Radunia Channel is 52 km^2. If we take into account that the river channel of Old Radunia serves only for conveying flood discharges and the main river bed is Radunia Channel then total catchment area will decrease to 836 km^2.

The elevations in the catchment vary from -0.3 m below sea level at the area of depression near the outflow of Radunia to Motława up to 328 m (Wieżyca Mountain) which is situated at the watershed of Radunia and Wierzyca Rivers. The density of river network over the whole catchment is uniform and may be regarded as average. In its upper course Radunia flows through several lakes of tunnel valley type of glacial origin. In the catchment of Radunia River there are 68 lakes of the total water surface of 33 km2 which amounts to nearly 4% of the catchment. These lakes vary considerably in size ranging from 1 to 735 ha. In the lower course of Radunia River there are 5 artificial reservoirs used mainly for hydro energy. Their total water surface is 196 ha. The main tributaries of Radunia are: Mała Supina (km 48), Strzelenka (Strzelniczka) (km 44), and Reknica (km 29).

In the longitudinal cross-section of Radunia the following three sections can be distinguished.

In the initial upper course Radunia flows through a chain of glacial origin lakes. Along this course of 30 km it is difficult to distinguish the river channel and therefore it is assumed that the real beginning of the river is in the km 73 (cross-section Ostrzyce) at the elevation 158 m. It may be assumed that Radunia River in its upper course has typically lowland character.

From Ostrzyce Radunia flows through high plateau of Pojezierze Pomorskie. Downstream of Goręczyno (water gauge) there is the section of consecutive narrowings and widenings of the valley. Within wide valley sections the channel slope is mild (0.2‰) and the river has lowland character. In the regions of gorges Radunia becomes mountain river. In some places it has a character of a torrent where the slopes reach even the values of 4 to 7‰.

Downstream from Kolbudy the slope of Radunia River becomes milder. In natural conditions it amounted to 1.6‰. Along this section several weirs and hydraulic power plants were constructed which cause the decrease of natural slope within the backwater.

The average elevation of the catchment is 164 m above sea level. The geological structure of Radunia catchment bears typical character of several glacial periods.

The catchment area of Radunia River is utilized in the following way: agricultural lands - 52%, forests - 32%, and waters - 4%. Inhabitated areas do not exceed 1% of the catchment area. The main functions of the area are: agriculture, tourism, and recreation. The terrains close to Gdańsk are used for housing schemes, services and small industry. The eastern part is the base of the development of Gdańsk, the middle part serves mainly for agriculture and the western part is dominated by tourism and recreation. Especially there are very good conditions for sailing and all kinds of water recreation.

4. Lakes in the Radunia River catchment

Lakes are the basic hydrographic elements in Radunia catchment and are the main form of surface waters. There are 68 natural water reservoirs, whose surface exceeds 1 ha. Total surface area of the lakes amounts to 3274 ha, which constitutes 4% of the Radunia catchment. The highest lake concentration is in the western, part of the catchment. Lakes occupy there 10.5% of the catchment, which is the highest lake indicator in Poland. Small amount of lakes is in the eastern part of the catchment.

Lakes in Radunia catchment are characterized with great variation of types, size and relation to the hydrographic system. Position and morphometry of the lakes in Radunia catchment depend on their variable genesis. Most of them have glacial origin.

Few lakes (Raduńskie, Ostrzyckie) have been physically modified over the years as the result of changes of the water levels for the water resources management purposes.

Lakes situated in the Radunia catchment show considerable variation in the trophy, chemical and sanitary state. More than half of the lakes are eutrophic reservoirs. They are shallow, have unfavorable oxygen conditions and well developed microflora. They belong to the II class of water quality.

5. Typology of the lakes

According to the WFD Radunia River catchment is situated in ecoregion 14; Central plains. Differentiation of the lake types is done using the typology System A (altitude, mean depth, surface area and geology). This typology gives potentially 13 different types of lakes in Poland. In practice in the catchment of Radunia River only 4 types of the lakes exist. The typology is presented in tables 1-3.

Table 1 Altitude typology of the lakes

Type	Aititude [m asl.]	Number of the lakes	%
high	> 800	0	0
mid-altitude	200-800	8	12
lowland	< 200	60	88
	Total:	68	100

The geological typology was done taking into account the contents of calcium compounds (25 mg Ca/l). There are 16 lakes examined with the surface area higher than 50 ha. 14 from them have the calcareous character. Additionally it has been decided to use System B to determine the optional factors for assessment of the residence time and mixing characteristics of the water. 11 lakes have the Schindler coefficient higher than 2, and 10 of them are stratificated.

Table 2 Size typology of the lakes

Surface area [km^2]	Number of the lakes	%
< 0.5[x)]	52	76
0.5-1	10	15
1-10	6	8
10-100	0	0
> 100	0	0
Total:	68	100

[x)] –proposed typology in Annex II of WFD gives the value 0.5 km^2 as the lower limit of the lake surface. In this paper, because of significant differentiation of lake surface, the group of lakes with smaller surface area was distinguished.

Table 3 Depth typology of the lakes

Mean depth [m]	Number of the lakes	%
< 3	12	18
3-15	29	43
> 15	1	2
Lack of data	26	38
Total:	68	100

The typology has the abiotical character. Due to the lack of knowledge it is not possible to establish reference conditions for the lakes. We can estimate the physical and chemical characteristics of the lakes only. Water of the lakes in Radunia River catchment is characterized by a good physico-chemical quality and can be accounted to II class of water quality. This was caused by the presence of chemical and organic compounds. The remaining indicators account lakes water to the I class. The biological quality of water secures the requirements of the II class. Water flowing out from Lake Ostrzyckie is accounted to the III class because of high concentration of chlorophyl "a". Its increase is observed in March, July and August during intensive development of plant organisms. In relation to the previous years most water quality indicators remained on the same level or even improved. Current status of the lakes is the following: from 16 lakes, 10 have the good and 6 the moderate conditions. The main reasons for failure to meet objective in the future are phosphorus and nitrogen loadings from agriculture.

Summary

The study of the typology and reference conditions for the surface water bodies in Poland, according to the WFD, is still in progress, therefore the conclusions can have only very general character.

1. Differentiation of the lake types in Radunia River catchment is carried out using the typology System A. Additionally two optional factors from System B are in use.

2. The typology has the abiotical character. Due to the lack of knowledge it is not possible to establish at present current ecological status, reference conditions and future objectives for the lakes.
3. In general water quality of the lakes (physical, chemical and biological) can be regarded as good.
4. From the preliminary assessment carried out it can be concluded that the majority of the lakes will fail to meet the high ecological status in 2015. But the identification of the need for further characterisation, including environmental monitoring, will be the basis for planning the programme of actions in the future.

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The Public Participation in the Water Resources Management on the Expert System Basis

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Boris Beraković

Abstract: An example of the public participation in the water resources management on the expert system basis is presented in this paper. The expert system is expected to improve water resources management by including people in water resources management activities. Involvement of the public in water resources management is a constant and time consuming process that should begin by educating the public about the importance of water resources, introducing public to the basic structure of water management processes and their own role in the process. Based on this idea an expert system was generated to create a connection and achieve better collaboration between public and institutions responsible for water management. Depending on the problem related to water and the location at which the problem occurs, the expert system guides the interested party (individual or a representative of a group like local community, club, association etc.) to the responsible institution and even to the exact department in the institution. Another advantage of this system is the possibility for the public of submitting notifications about spotted water problems and directly forwarding them to the relevant institutions. This approach could improve the recognition of problems in time, which is one of the initial and important, steps in planning the water resources management. An expert system like this could be, with the appropriate computer interface, placed at water management institutions web sites. It could be useful to individuals, directorate and municipal bodies, and other interested parties for directing their inquiries to the right institution. This example is an idea of how expert systems can be used in water management with the goal of improving the integration of public in water management process.

Keywords: public participation, water management

Introduction

Water management is a set of permanent complex activities that implies water use, flood protection and water conservation and includes planning and implementation of water management strategies.

The objective of water management is to satisfy human needs related to water, including fulfilling environmental, social and economic criteria.

In the water management strategy planning process, the first step is to recognise the problems related to water. Institutions competent for water management should carefully and continuously observe processes that are occurring in society and that are related to water, in order to make optimum planning of appropriate water management strategies. Including public interested in water use, flood protection, water conservation and other activities in connection with water in the water management planning leads to acceptable, optimum projects. Some large and important projects have failed because the public was not included in the planning process or was included too late (in the final phase of the project).

Public participation in water management process is possible only if the public is previously educated about water, its importance for life and further human development, the problems related to water and water management, the institutions competent for water management and its own role and potential contribution to the water management process. Educating public about the above-mentioned issues should begin with education in elementary schools. In the document of UN Agenda 21 [1] the importance of involving public in society development processes, environmental conservation and sustainable development, with the accent on involving women and children is highlighted. According to EU Water Frame Directive, involving public in making River Basin Management Plans is compulsory [2].

In Croatia the water management is in the competence of Water Management Directorate, within the Ministry of Agriculture, Forestry Water Management, and the state-owned company Croatian Waters, but the distribution of drinking water and the waste water disposal is in the competence of Communal Societies. Also, the use of water power is in the competence of Croatian Energy Directorate which manages water on the base of concessions that make of them institutions competent for water management in relation to the public (consumers). From the aspect of water quality, monitoring analyses are continuously performed by Croatian Waters, but also by Public Health Institutes and, in the case of water supply and sewage water disposal by Communal Societies also. There also exist other institutions related to diverse water management processes: Ministry of Sea, Tourism, Transport and Development, Ministry of Environmental Protection, Physical Planning and Construction and others.

Due to the numerous institutions related to water management and their internal complex structure, determining the appropriate institution to contact for a specific problem related to water is not always an easy and simple task. Therefore it would be useful and appropriate to provide public help whereby, on the base of the problem, the person/costumer would be directed to the appropriate institution and even the right department or person.

Contribution of people in recognition and solving the problems related to water could be achieved using expert systems, which guide the person/costumer to relevant points in the process of solving the problems on the base of relevant information. Such expert system could be integrated on the web pages of the institutions responsible for the water management issues. The notification or request about the recognised problem related to water could be directly forwarded to the right department with the possibility for the interested party to insert all personal information and data, that gives to the notification a bigger weight, or can be made anonymously. In the latter case, the problem can be treated as an indication and should be verified on the field.

Expert system basics

Expert systems are computational tools that mimic or at least try to mimic the human thinking process and the human capability for solving problems and decision making by using knowledge bases and rules from a desired domain. They are appropriate for solving badly or poorly structured problems such as water management problems, because their solution can be based on heuristic and empirical knowledge [3]. They are capable of incorporating mathematical models, empirical knowledge and expert judgements, engineering intuition, heuristic rules and needed information, in order to provide a useful advice to the user that can help him make the best decision related to the problem.

According to [4], there are two types of expert systems: knowledge based expert systems (KBES) and expert systems based on neural networks (NN). Often a combination of knowledge based expert systems and neural networks, called the hybrid expert system or expert network, is used. The knowledge based expert system can be the rule based, the rule inducing or the case based expert system.

Expert systems consist of six subsystems [5]: knowledge acquisition, knowledge base, working memory, inference engine, explanation system and user interface, while three “persons” are involved in design, development, maintenance and work of the expert system: user(s) – person that will use the expert system via users interface, expert(s) – expert person or other sources of knowledge (such as literature: books, journals, Internet etc.) from which expert knowledge for the knowledge base is acquired, and knowledge engineer(s) – person that generate the expert system on the base of the acquired knowledge and later on maintain, update and include new knowledge (Figure 1.). The knowledge engineer and the expert can even be the same person.

The inference engine makes conclusion based on the knowledge base and applied IF-THAN rules.

Expert systems can be designed using a programming language or an expert shell.

Programming languages used for the expert system design are Lisp, Prolog, C and Pascal, and nowadays C++, Java, Visual Basic and others [4,5,6]. For the design of expert systems using programming languages, it is necessary to have programming skills and to know the basic principles and rules of programming.

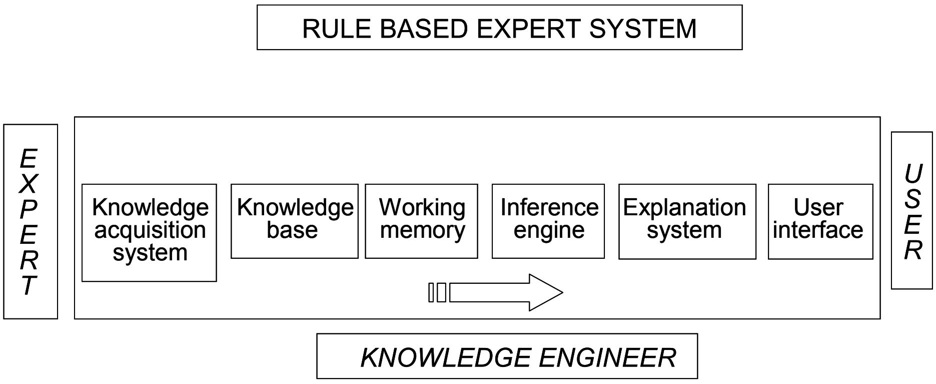

Figure 1 Expert system scheme

Expert shells are tools that enable people that do not have a good knowledge of programming to design expert systems. They are developed by eliminating knowledge bases from expert systems used for specific problems or fields, creating the so called "empty" expert systems – expert shells. Embedding new knowledge base in these expert shells using the old principles for inference and explanation, the new expert system can be designed.

Nowadays there is a wide range of different expert shells on the market that can include different types of expert systems (rule based, rule inducing, case based expert systems and/or neural networks). Some of them are free such as: BABYLON, ES, GEST, CLIPS, RT-Expert for DOS Personal Edition, while the others are for sale: Aion Development System (ADS) ART Enterprise, Doctus KBS, Expert choice, Expert edge, EXSYS, Insight 2+, KEE, Knowledge kraft, M.4, Nexpert Object, OPS83, Personal Consultant, Personal Expert, RT-Expert, XpertRule and many others [6].

The use of expert systems in water management is diffused all over the world in: water quality control [7,8], drinking or sewage water treatments [9,10,11], development and management of water supply, sewage, irrigation, drainage and food protection systems [12,13,14,15], water management strategy planning [16] etc.

In this paper the expert shell Xpert Rule KBS (Attar Software Ltd.) was used for the design of the expert system.

Expert system development

Developing expert systems can be explained by the following procedure [5]:

1) Statement of the problem to be solved
2) Searching for human expert or the equivalent data or experience
3) Design of the expert system
4) Selection of the degree of participation of the user
5) Selection of the development tool (programming language or shell)
6) Development of the expert system prototype

7) Prototype verification and refinement
8) Maintenance and updating

The knowledge required for creating the knowledge base can be acquired from experts (by interviews, questionnaires etc.), from the literature where it can be stored (books, manuals, articles, laws, etc.) or from internet. The knowledge base necessary for building the expert system that should direct the public to a competent institution on the base of the stated problem related to water and the location at which the problem occurs is, in this paper, mostly derived from the authors' knowledge extended by the knowledge from literature and internet.

Problems related to water can be assigned to three categories:

- Problems related to the use of water and water goods
- Problems related to water protection and conservation
- Problems related to flood and erosion protection

The problems related to the use of water and water goods can be: inadequate pressure in water supply systems, bad quality of water in water supply systems, public interest in water use for irrigation, interest in building and use of hydroelectric plants, interest in exploitation of sand and gravel etc.

The problems occurring in relation to water protection and conservation can be: recognition of river or sea pollution, recognition of points where the nontreated waste water or waste material is emitted in water resources (rivers, lakes, seas), recognition of wild waste disposals in water protection zones, recognition of leakage form septic tanks etc.

The problems related to flooding, flooding protection and erosion protection can be: periodical recognition of flooding the areas from rivers, periodical recognition of flooding due to inadequate rainwater drainage, river embankment ruining, bad maintenance of drainage systems, icefloods etc. and recognition of erosion processes.

This is a rather rough overview of problems that the public is capable to recognize and which, if recognized at the right time, can help in improving the quality of planning the water management strategies.

In the first problem solving step, as well as in later water management strategy implementation control, the public has a considerable role. The public is the local inhabitants living in the water management area (watershed), the inhabitants that live outside the watershed but have advantages from it, and all other inhabitants. The public is: individuals, local, region and state authorities that should represent the constituents' interests, other representatives, nongovernmental organizations, different groups, clubs, societies etc.

The institutions responsible for water management in Croatia are Water Management Directorate, within the Ministry of Agriculture, Forestry and Water Management, and Croatian Waters, but also Communal Societies, Public Health Institutes, Ministry of Sea, Tourism, Transport and Development, Ministry of Environmental Protection, Physical Planning and Construction and others. The competent institutions are numerous and also complex in their inner structure.

Within the Water management Directorate there are inspections at the state and region levels, while Croatian Water are structured on the state level, water region and watershed areas levels and by categories: use of water, water protection and conservation and flood and erosion protection. Communal societies are organized on administrative unit basis (cities or municipalities) and can include both drinking water supply systems and sewerage systems or just one of them.

In this paper an overview of basic problems related to water is given. Expert systems are capable of upgrading and updating so it is possible to expand the set of problems related to water and to include other institution related to the water management process creating a wider and even better expert system.

Expert system in use

The user/the public uses the developed expert system through the user interface shown in figures 2 to 8. Figures from 2 to 8 show the path of the user from the recognized problem, in "water use" water management area, "inadequate (too low or too high) pressure in water supply system", and on the base of the region "Primorsko-goranska županija" and the city "Rijeka" at which the problem is recognized, to the competent water management institution related to the problem. That is Communal Society - Vodovod i kanalizacija d.o.o. Rijeka, and the exact department and person to contact (address, telephone, fax, e-mail).

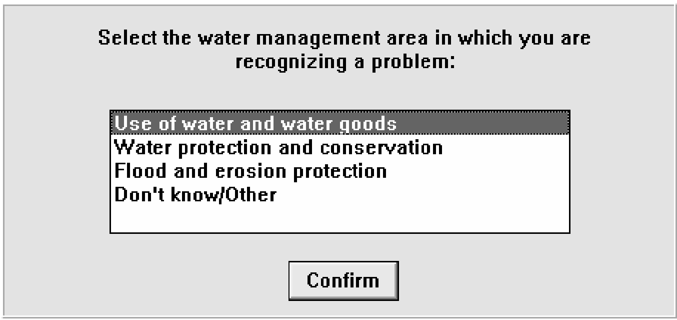

Figure 2 Selecting the water management area in which a problem is recognized

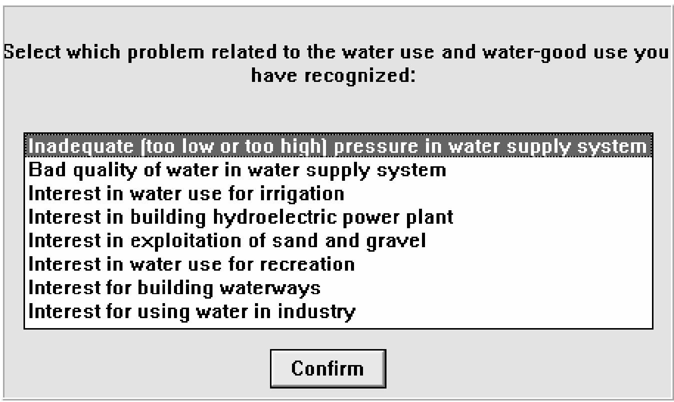

Figure 3 Selecting the problem related to water or water-goods use

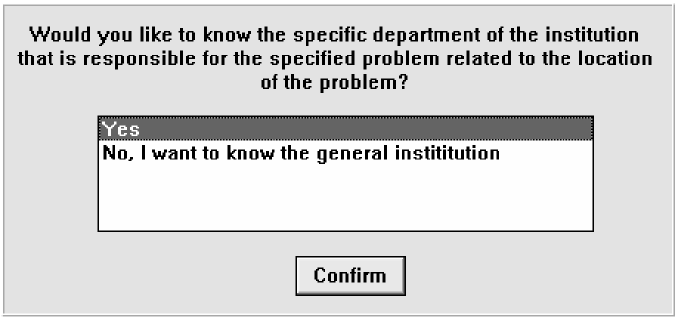

Figure 4 Selecting the general institution responsible for the problem or the specific department related to the location at which the problem occurs

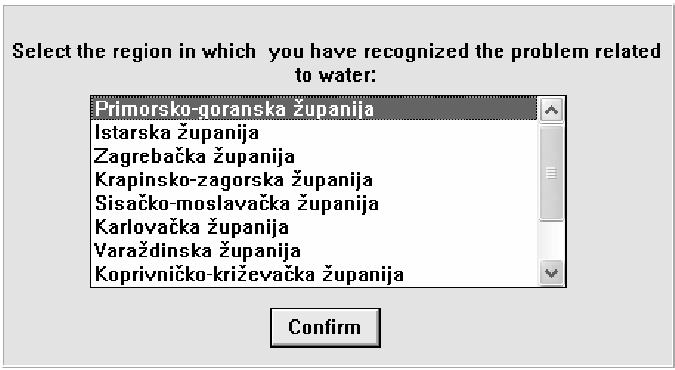

Figure 5 Selecting the region in which the problem occurs

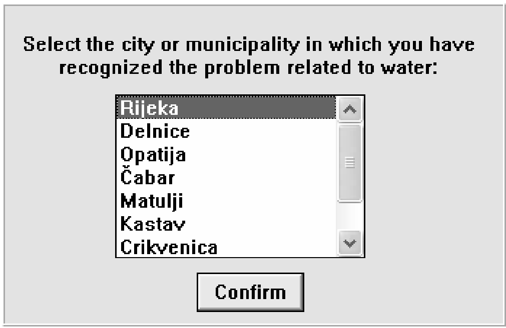

Figure 6 Selecting the city or municipality in which the problem occurs

Figure 7 The result of using the expert system: the person/department in a responsible water management institution to contact related to the recognized problem (information taken from web site (17))

Summary

An example of the public participation in the water resources management, using an expert system that guides the public to the water management institution that is responsible for a defined problem is presented. The goal of this paper is to illustrate how to achieve better communication between the public and water management institutions using today's technology (like internet) in order to reach a higher quality in planning the water management strategy and in control of the implementation of the plan.

The use of such expert system could be useful to individuals, directorate and municipal bodies, and other interested parties for directing their inquiries to the right institution.

This example is an idea of how expert systems can be used in water management with the goal of improving the water management process.

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Construction and Reconstruction of Pumping Stations for Drainage in Croatia

Josip Marušić
Diana Šustić

Abstract: Out of the total 1,673,792 hectares of amelioration areas in Croatia, gravitational drainage is not possible on 422,000 hectares. Until 1990, 82 pumping stations were constructed in Croatia, and in 2004, 71 of those were operational, with the total capacity of 316.5 cu.m/s and total power of 24,166 KW – for the drainage of 276,000 hectares of lowland amelioration areas. Along with the problem of insufficient number of constructed pumping stations, during the war (in 1991 and 1992) damaged or destroyed 29 pumping stations. Out of that number, 19 pumping stations, used for drainage of 89,119 hectares (47.9%), sustained significant damage. Their total capacity is 99.50 cu.m/s, and power 7,974 KW. In the assessment of significance of the existent 71 pumping station, one should note that they are used for drainage of 22 to 24 percent of the total sowed land in Croatia. One should also take into consideration that additional construction of existent pumping stations and construction of new pumping stations is needed for drainage of another 146,000 hectares of lowland areas with unfavourable relation of groundwater and external water levels – all with the goal of creation and maintenance of the optimal air-water regime of agricultural land.
Keywords: pumping station, drainage, Croatia

1. Introduction

Out of the total 1,673,792 hectares of amelioration areas in Croatia, gravitational drainage is not possible on 422,000 hectares. These are areas lower than the high water levels in lowland sections of river basins in Croatia. Drainage of excess water from these areas during the unfavourable relation of groundwater (on amelioration areas) and external water levels (in rivers or main recipients of excess water) would be made possible by construction and operation of pumping stations. Until 1990, 82 pumping stations were constructed in Croatia, and in 2004, 71 of those were operational, with the total capacity of 316.5 cu.m/s and total power of 24,166 KW – for the drainage of 276,000 hectares of lowland amelioration areas. The need remained for the construction of new pumping stations for drainage of 146,000 hectares of amelioration areas, but for a part of them, construction of hydrotechnical facilities for the protection from external floodwater is required. Along with the problem of insufficient number of constructed pumping stations, during the war (in 1991 and 1992) 19 pumping stations were damaged and destroyed, whose total capacity was 96.5 cu.m/s. From 1998 to 2002, 8 pumping stations were reconstructed, of individual capacity of 2.0 to 20.0 cu.m/s, but 11 smaller pumping stations were not reconstructed (of individual capacity of 0.5 to 1.0 cu.m/s).

2. Selection of locations for pumping stations

The need for construction of pumping stations for drainage of lowland amelioration areas is conditioned by the unfavourable relation of water levels in the main recipients and amelioration canals. The requirements for maintaining the water and air regime of agricultural land, as well as timely drainage of excess water from other lowland areas, determine the selection of location for pumping stations. Integral parts of that process are analysis of data, selection of the characteristic return period, and the duration of high water levels in rivers, i.e. main recipients in individual amelioration areas.

For the selection of locations for pumping stations, it is necessary to conduct the appropriate field survey and research/testing. Topographic, climatic, hydrological, and geomechanical data are of special significance for the systematization and analysis of data. By conducting detailed analyses of the said data, we can determine the mean hydraulic and construction parameters, but also the electrical-mechanical parameters of pumping stations. The total basin area (in hectares) and the characteristic hydro-module of drainage (l/s/ha) determine the total capacity (l/s, cu.m/s) of individual pumping stations. Also, detailed geomechanical research and analyses must be conducted for the purpose of selection of the optimal manner of foundation of the pumping station facilities – which represents a large share of the total cost of construction of the pumping station.

Geomechanical research should determine the composition of soil and its geomechanical characteristics and the flow conditions of groundwater, both in the process of construction and operation of pumping stations. Along the main loads, the calculation of foundations for individual facilities should also include additional loads, which are most often the seismic and dynamic influences. The influence of the level of ground water is important, for it reduces the allowed specific soil loads. For that reason, additional foundation calculations are needed for the engine room, pressure and gravitational pipeline, walls of the main supply and drainage canals (discharge facility), valve facility etc. The most complex and most important is the calculation of foundations or actual settlement of the main facility of the pumping station, which is the engine room as the heaviest and with deepest foundations. The question of insuring the stability of the engine room with the pressure and gravitational pipeline is also sensitive. Conducting of a detailed analysis of settlement of all facilities of the pumping station is necessary for the selection of the manner of foundation – on foundation plates or pier foundations. Within this, the dynamic effect of operation of the pumps, effect of suction of small particles of soil from the base of the facility, determination of differential settlement and seismic influences are also to be taken into consideration. Insuring of hydraulic stability of facilities of individual pumping stations is an integral part of this process.

3. Contents of project and other documents for pumping stations

With the goal of determining the optimal (technical and financial) project solution of the pumping station, the following activities should be performed:

- Analysis of the micro-location of the facilities of the pumping station, taking into account the natural characteristics of amelioration areas (inflow and discharge of water, foundation conditions);
- Development of different variants of the conceptual design of the pumping station – with basic technical indicators;
- Analysis of economic indicators – from the viewpoint of optimalization and typization of facilities, including necessary equipment;
- Selection of the optimal technical and economic solution of the pumping station;
- Definition of the terms of reference on the basis of previous analyses – for the purpose of developing the main project design.

The development of the main design of the facilities of the pumping station consists of the construction, engineering, and energetic sections. Within the development of the construction design of the pumping station, the following activities are of significance:

- Review and amendment of data given in the conceptual design;
- Hydrological analyses, hydrological calculation, calculation of foundation, and static calculation for individual sections of facilities of pumping stations – including the hydraulic stability of the facilities;
- Revision of project designs for the facilities of the pumping station and recommended plant and equipment of the pumping station.

The next step is the preparation of public bidding documentation in accordance with current laws and regulations, and then the selection of the contractor for the construction of the facilities of the pumping station – with adequate subcontractors (foundation works, plant and electrical equipment).

It is necessary to organize supervision of project consultants and contractors in the process of construction of facilities and installation of equipment – along with the coordination of actions of all participants.

Within the development of project and other documents for pumping stations, projects for energy supply, access roads, and management facilities should be created separately.

4. Construction of pumping stations up to 1990

Depending on the construction of hydrotechnical objects (dikes, regulation facilities) for flood protection, construction of pumping stations for drainage of lowland areas of certain river basins or amelioration areas started. Table 1 contains a summary review of pumping stations in the main catchment areas in Croatia, namely the Sava, Drava and Danube river basins, Dalmatian basin, and the Istrian Littoral basin. The oldest is the pumping station "Podunavlje" in Baranja, constructed in 1874 and reconstructed twice, in 1956 and 2001. Along with the number of pumps and year of their construction, other important data are their total capacity (cu.m/s), power (KW), number of pumps, and the area of river basins and catchment areas being drained during the unfavourable relation of groundwater and external water levels.

Table 1 Basic indicators of pumping stations for amelioration drainage according to catchment areas in Croatia

Catchment areas including the number of amelioration areas	Number of facilities	Basin surface – hectares	Total capacity cu.m/s	Total power KW	Average hydro-module l/s/ha
Sava (10)	36	99,542	165.59	12,437	1.66
Drava-Danube (3)	17	73,166	53.85	4,424	0.74
Istrian Littoral (2)	4	1,301	9.44	464	7.26
Dalmatia (4)	13	12,189	66.28	4,077	5.44
Total - Croatia (19)	70	186,198	295.16	21,402	1.59
p.s. Bosut - total	1	126,450	30.00	1,890	0.24
p.s. Bosut (Croatia - 71%)	1	89,780	21.30	1,342	0.24
Total - Croatia (including 71% of p.s. Bosut)	71	275,978	316.46	22,744	1.15

5. Reconstruction of pumping stations for drainage of amelioration areas

In the aggression of the ex-Yugoslav army and various Serbian paramilitary forces, among other things, some of pumping stations for drainage of amelioration areas were damaged or destroyed. The following amelioration areas i.e. catchment areas were damaged the most:

Baranja (9 p.s.), Vuka (6 p.s.), Subocka-Strug (3 p.s.) Brodska posavina (3 p.s.), Banovina (1 p.s.), Šumetlica-Crnac (2 p.s.), Biđ-Bosut (2 p.s.), Krka (1 p.s.), Cetina (1 p.s.), Vrana (1 p.s.) – a total of 29 pumping stations.

19 pumping stations suffered significant damage, and the pumping station "Paulin Dvor" in the Vuka river basin – of a total capacity of 20.0 cu.m/s and the power of 1,264 KW

(4 pumps of capacity of 5.0 cu.m/s and power of 316 KW - pictures 5.1, 5.2 and 5.3) – was completely destroyed. The basic indicators for the 19 pumping stations which suffered significant damage are the following:

- Catchment area 89,119 ha (47.9%), capacity 99.50 cu.m/s (33.7%) and power 7,974 KW (37.3%). The percentage relates to the total number for the Republic of Croatia (without the pumping station Bosut).

Reconstruction of pumping stations started after the Croatian military operations “BLJESAK” (May 1995) and “OLUJA” (August 1995), first in the area of Lonjsko polje i.e. the area under the authority of the Water Management Branch Office “Subocka-Strug”, a section of Hrvatske vode, the legal entity for water management, belonging to the Water Management Department “Sava”. Works on the reconstruction of both the facilities and plant and electrical equipment were conducted from 1996 to 1998 on the following pumping stations:

- *Mrsunja* (Q = 8.0 cu.m/s; N=660 KW), Migalovci (Q=12.0 cu.m/s; N=950 KW) and Dubočac (Q=4.4 cu.m/s; N=280 KW), Grlić (Q=8.0 cu.m/s; N=800 KW – in the area of Jelas-polje (Water Management Branch Office Slavonski Brod);
- *Mlaka* (Q=0.80 cu.m/s; N=100 KW), Košutarica (Q=0.75 cu.m/s; N=65 KW) and Lončarica (Q=4.0 cu.m/s; N=380 KW) – in the area “Strug-Subocka” (Sava), Water Management Branch Office Novska;
- *Hrastelnica* (Q=2.0 cu.m/s; N=180 KW) – in the area “Kupa-Banovina” (Water Management Branch Office Sisak);
- *Vedrine* (Q=8.8 cu.m/s; N=550 KW) – in the area “Cetina” (Water Management Branch Office Sinj)
- *Nadin* (Q=2x0.30=0.60 cu.m/s; M=2x50=100 KW – in the area “Vrane” (Water Management Branch Office Zadar)

Along with the damage to facilities, the pumps and other plant and electrical equipment of pumping stations also sustained significant damage. Appropriate documentation was developed for restoring the pumping stations into operational condition, according to which the construction works were conducted, as well as the reparation of damaged equipment and, if necessary, procurement and installation of new equipment. An integral part of reconstruction works were ensuring the energy supply and reconstruction of management facilities.

Pumping stations in the amelioration area of Baranja and Vuka were the most damaged and destroyed. Unfortunately, due to the slow process of peaceful reintegration of this area, reconstruction of hydrotechnical and other facilities started at the end of 1998 – after complete reintegration of occupied areas of Baranja, Western Srijem and Eastern Slavonia under the authority of the Croatian Government.

During 1998 and 1999, after conducted public bidding procedure, the reconstruction of the canal network and pumping stations in Baranja was contracted. The works were financed by a World Bank loan through a contract signed on September 8, 1998 in Washington. The company Hrvatske vode was in charge of reconstruction of protective and drainage hydrotechnical facilities. In the temporary occupation of Baranja, Eastern Slavonia and Western Srijem from 1991 to 1997, some of the protective dikes were damaged, as well as some other hydrotechnical facilities of the system of surface drainage. Due to the impossibility of performing regular maintenance, the flow profiles were reduced in relation to their designed and constructed elements, both in the main watercourses and in amelioration canals. An integral part of the reconstruction program was the activity of de-mining a part of protective hydrotechnical facilities and amelioration areas. After that, the technical clearing started – cutting of trees and bushes, extraction of tree stumps, and de-sludging of the main drainage canals and amelioration canals of categories III and IV. The project documents for the reconstruction of pumping stations were developed by the company Hidroprojekt-ing d.d. from Zagreb.

The works on reconstruction of facilities of pumping stations in Baranja started at the end of 1999, and continued throughout 2000. For some pumping stations new plant and equipment were procured due to damage sustained, and partially due to the age of the plant.

From the end of 1999 up to 2001, the works on reconstruction of the following pumping stations in Baranja were conducted:

- *Velika* (on the left bank of Drava between Darda and Osijek), capacity Q=2x1.75 cu.m/s=3.5 cu.m/s and power 2x160=320 KW;
- *Podunavlje* (in Kopački rit, next to the road Bilje-Kneževi Vinogradi), Q=1.75 cu.m/s, N=160 KW; (picture 5.8)
- *Tikveš* (Kopački rit, settlement Podunavlje) Q=2x2.5 cu.m/s=5.0 cu.m/s and N=2x200=400 KW; (pictures 5.9 and 5.10)
- *Zlatna greda* (Kopački rit, settlement Zlatna greda), Q=2x1.75+2.5=6.0 cu.m/s and N=2x160+200=520 KW; (pictures 5.1, 5.2 and 5.3)
- *Budžak* (right bank of Danube, on the border with Hungary) Q=0.4 cu.m/s and N=50 KW;
- *Draž* (next to the road Kneževo-Batina, settlement Draž), Q=1.5 cu.m/s and N=160 KW; (pictures 5.4 and 5.5)
- *Gombaš* (next to settlement Batina), Q=0.3 cu.m/s and N=50 KW;
- *Bakanka* (left bank of Drava, next to the road Baranjsko Petrovo Selo – Belišće), Q=2x2.15+2.5=4.3 cu.m/s and N=2x180=360 KW; (pictures 5.6 and 5.7)
- *Puškaš* (old course of the Danube - Topoljski Dunavac), Q=2x1.0=2.0 cu.m/s and power of N=2x50=100 KW.

The total capacity of the said 9 pumping stations in Baranja is 24.75 cu.m/s, and total power 4,424 KW.

Reconstruction of the said pumping stations was very complex due to their age and sustained damage, both on the facilities and the plant and electrical equipment, as well as the power supply. The largest capacity of Q=6.0 cu.m/s belongs to the pumping station *Zlatna greda* which, together with the pumping stations *Tikveš* (Q=5.0 cu.m/s) and *Podunavlje* (Q=1.75 cu.m/s) serves the purpose of drainage of the central part of the Kopački rit area and part of the Danube river basin – total area of 22,500 hectares. Another important piece of information is that the pumping station *Tikveš*, during high water levels of the Danube, "discharges" water into the existing amelioration system for the purpose of water supply to the fishpond *Podunavlje*, and partially for the purpose of irrigation of agricultural land. It should also be mentioned that part of the equipment of the pumping station *Budžak*, which was irreparable (for technical and financial reasons), was mounted in the former storage area of pumping station *Zlatna greda* as an exhibition or "witnesses" to the continuity of drainage of amelioration areas in Baranja by pumping stations (since 1874). Regarding the importance and need for operation of pumping stations, one should take into account that in Baranja 1,056 km of amelioration canals were constructed for drainage of 76,730 hectares of agricultural and other surfaces (out of the total 112,780 hectares of the entire Baranja area).

In autumn of 1991, the pumping station "*Paulin Dvor*" in the Vuka river basin – of total capacity of Q=4x5.0=20.0 cu.m/s and power 4x316=1,264 KW, whose construction was finished in November 1981 – was completely destroyed. Consequences of the aggression of Serbian paramilitary forces can be seen on picture 5.1. The pumping station "*Paulin Dvor*" is used for drainage of 20,000 hectares of pedologically extremely favourable soil that is under frequent influence of excess surface water and high levels of groundwater. Along with the consequences of war destruction of pumping stations during the temporary occupation, the regular maintenance works of main and amelioration canals in the lowland area of the Vuka river basin were not conducted. Only after the slow process of peaceful reintegration the conditions were created for the review of the state of hydrotechnical facilities for the protection from adverse effects of external water and drainage of excess groundwater. Aside from the completely destroyed pumping station "*Paulin Dvor*", the following smaller pumping stations were significantly damaged or destroyed in the area of the Vuka river basin:

- *Seleš* (Q=0.7 cu.m/s; N=80 KW), Vrbik (Q=0.70 cu.m/s; N=80 KW),
- *Ernestinovo* (Q=0.70 cu.m/s; N=80 KW), Oraščić (Q=1.0 cu.m/s; N=100 KW),
- *Rudine* (Q=1.0 cu.m/s; N=120 KW), Ovčara (Q=1.0 cu.m/s; N=120 KW)
- of total capacity of 5.1 cu.m/s and power of 580 KW.

The said pumping stations were used for drainage of lowland amelioration areas ("depressions") of surface of 60 to 120 hectares.

Before the works on the reconstruction of pumping stations started, the locations of the facilities, main amelioration canals, and agricultural land in the lowland area of the Vuka river basin had to be de-mined. The total area of the Vuka river basin takes up 179,300 hectares, but the hydro-amelioration facilities and systems were constructed for drainage of 141,410 hectares, out of which 24,100 hectares are naturally favourable agricultural land. An integral part of the reconstruction of pumping stations was technical cleaning of main and amelioration canals (cutting of trees and shrubs and extraction from the canals, extraction of tree stumps and de-sludging) – all with the goal of restoring them to the operational state.

Fig. 5.1 Pumping station “Paulin dvor“ – after the aggression of Serbian paramilitary forces, 1991

6. Conclusions

Up to 1990, 82 pumping stations for amelioration drainage were built in Croatia. In 2004, 71 of them were operational, with total capacity of 316.5 cu.m/s and power of 22,744 KW. There are 12 pumping stations with the capacity of less than 1.0 cu.m/s. A large percentage, namely 34 pumping stations have the capacity between 2.0 and 8.0 cu.m/s. The following pumping stations are of the largest capacity: Crnac (9.0 cu.m/s), Migalovci (12.0 cu.m/s); Mahovo (12.0 cu.m/s), and Davor (15.0 cu.m/s) in the Sava river basin, Paulin Dvor (20.0 cu.m/s) in the Vuka river basin, Trilj (10.88 cu.m/s) in the Cetina river basin, and Modrić (20.0 cu.m/s) in the Neretva river basin. In the Sava river basin, the pumping station of the largest capacity is the pumping station Bosut (30.0 cu.m/s), which is located in Vojvodina (Serbia and Montenegro), but 71% of its gravitational area is in the Republic of Croatia. A total of 70 pumping stations are used for drainage of 186,198 hectares of land in Croatia, and with the Croatian part of the pumping station Bosut, the total area is 275,978 hectares. The average hydro-module of drainage is 1.15 l/s/ha, the minimum is 0.24 (pumping station Bosut), and the maximum is at pumping stations constructed in the Dalmatian basin (5.44 l/s/ha) and Istrian Littoral basin (7.26 l/s/ha). On the amelioration areas of the Drava, Danube, and Sava river basins, the characteristic hydro-module for timely drainage of excess water is from 0.5 to 2.0 l/s/ha, and the current capacities of 53 pumping stations have an average hydro-module of 0.74 to 1.66 l/s/ha (excluding pumping station Bosut).

The aggression of the ex-Yugoslav army and Serbian paramilitary forces in 1991 and 1992 damaged or destroyed 29 pumping stations. Out of that number, 19 pumping stations, used for drainage of 89,119 hectares (47.9%), sustained significant damage. Their total capacity is 99.50 cu.m/s, and power 7,974 KW. The pumping station Paulin Dvor, of the total capacity of 20.0 cu.m/s and power of 1,264 KW, was completely destroyed in 1991. Reconstruction of that pumping station was finished in 2002. That pumping station is used for drainage of 20,000 hectares of agricultural and other land in the Vuka river basin. Nine pumping stations in Baranja, of total capacity of 24.75 cu.m/s and power 4,424 KW, also sustained significant damage. Their reconstruction was conducted from 1999 to 2002. Pumping stations constructed for drainage of lowland amelioration areas in Jelas polje, Črnec polje, Lonjsko polje and Mokro polje in the Sava catchment area sustained medium to smaller damage.

The significance of pumping stations should be evaluated in view of the need for drainage of 276,000 hectares of lowland amelioration agricultural land, but also bearing in mind their role in drainage of excess water from areas with constructed settlements and roads. In the assessment of significance of the existent 71 pumping station, one should note that they are used for drainage of 22 to 24 percent of the total sowed land in Croatia. One should also take into consideration that additional construction of existent pumping stations and construction of new pumping stations is needed for drainage of another 146,000 hectares of lowland areas with unfavourable relation of groundwater and external water levels – all with the goal of creation and maintenance of the optimal air-water regime of agricultural land.

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Integrated Water Resources Management in Malopolska Region

State of the Work

Elżbieta Nachlik

Abstract: Regional project of integrated water management development should be compatible with the National Plan of Development and the Strategy of Regional Development and with the Water Framework Directive UE 2000/60/WE (FWD) as well as it should respect conclusions of European policy in frames of flood protection.
This need a cooperation between the Cracow Regional Water Management Authority and the Małopolska Regional Government.
Te paper presents the current state in relation to some important problems in this approach, as: advances in integral approach to planning in water management, advances in activities on implementation of Framework Water Directive, assessment of hitherto existing realisation of strategy and voivodeship operational plan including mainly experiences coming out from institutional cooperation in frame of water management and advances in being prepared, for the years 2007–2013, new strategy of regional development and operational plan.6
On that basis some problems, which have to be solved, and their causes have been identified as well as the necessary conditions which could improve the situation have been pointed out. Detailed analyses and assessments have been referred to those catchments in the voivodeship, for which work on implementation of FWD is very much in progress and for which exist parallel many social and economic problems concerning utilisation of water resources and flood hazard.
A great deal of conclusions arising for the Małopolska region could be generalised for most of voivodeships. To reach this aim the diagnosis of the water management elaborated for the whole country has been used.

Keywords: water management; integrated development; regional cooperation.

Introduction

A regional plan of integrated development of water management should, on the one hand, be consistent with the National Development Plan and the Regional Development Strategy, both developed in EU planning cycles, and, on the other hand, it should take into account the long-term strategic objectives and the guidelines of water management plans in the area of water resources management and protection resulting from the EU Water Framework Directive (WFD), as well as it should meet the settlements of European politics concerning flood protection measures.

These conditions do not exclude one another; they require only an adequate cooperation level of the institutions responsible for Regional Development Planning. In the case of the Malopolska Region (Voivodeship), this concerns the Regional Water Management Authority in Kraków and the Malopolskie Voivodeship Local Government. The space boundaries are defined by the boundaries of the Malopolska Region or – wider – the boundaries of the catchments and the adjacent regions.

In social-economic and water management planning the following goals should be integrated:

- water protection, understood through the requirements of WFD and daughter directives laying down criteria for water quality and water-related environmental quality;
- water utilization for social and economic objectives, i.e., drinking water supply, water for industry and agriculture, water power utilization, navigation and water recreation;
- flood and drought protection.

The range and method of implementation of these objectives are fit to the regional and local needs. However, the most important are the principles, organization, and the effects of integration at the level of defining needs and efficiency in planning and implementation of solutions based on the institutional competence.

The Malopolskie Voivodeship is an area where a part of pilot solutions of this scope is concentrated. In general, traditional way of planning development in Poland was of character totally different from the present requirements. Political system transformations, access of Poland to the European Union and, in consequence, the implementation of European water policy in new form, all this are challenges than have not up to the present been covered by the adequate detailed legal regulations. For this reason practical experiences are being sought that, when properly consolidated, will be the basis for future modified and detailed legal regulations.

So far (to 2004), the experiences point at the low level of cooperation in attaining a synergic effect in development linked with water management development. However, a change in the approach, especially related to the implementation of regional development and European water policy, facilitates the progress, initiated in 2004 and consistent with WFD assumptions, in integrated responses process for environmental objectives achieving.

Short characteristics of the Malopolska region vs. the Upper Vistula basin district

The Malopolskie Voivodeship, located in south of Poland, has the area of 15,190 km^2 which is 4.8% of the whole country area, and takes 12 place in Poland.

The Malopolskie Voivodeship's nature's environment is the most diversified in Poland. Over half of the area is protected by law, which situates the voivodeship at second place in the country.

The Malopolskie Voivodeship's population of about 3,260,000 is 8.5% of the country population. The urbanization index of 50% is considerably less than that of the whole country (61.6%). Kraków is the main agglomeration, it belongs to the greatest five (third place in the country) and most important metropolises of Poland. It is the second Polish university and science centre.

Mean population density is 214 people/km^2 and is considerably higher than the country mean (122 people/km^2). This index varies in space: from more than 300 people/km^2 (in western part of the voivodeship) to 76 people/km^2.

Agricultural land uses cover 753,000 ha which is about 50% of the Malopolska territory area; forests and forest lands cover the area of 444,000 ha, i.e. 29% of the voivodeship area, while the other land (under buildings, water, roads, and idle land) is 327,000 ha, more than 21%. Private farms' share in the total area of agricultural farm areas in the voivodeship is 97%, their share in agricultural land is 98%. Average area of a farm is as small as 2.6 ha. Over 35,000 industry units are operated in the voivodeship area.

Regarding its natural values, diversified relief and cultural richness, the Malopolskie Voivodeship is the area of special attraction for tourists. In 2003, about 8 million people visited Malopolska and this number is systematically growing.

According to the catchment division, the following should be specify in the Malopolskie Voivodeship:

- catchments of large Carpathian tributaries: of Skawa river (about 1200 km^2), Raba river (over 1500 km^2) and Dunajec river (near 7000 km^2);
- catchments of small Carpathian tributaries and right-side tributaries: (Uszwica, Breń, Rudawa, Prądnik, Dłubnia, Szreniawa, Nidzica rivers);
- the Vistula river at the length of over 130 km links this part of the Upper Vistula basin called the Upper Vistula basin district.

On the Skawa river the Świnna Poręba storage reservoir (under construction) is situated. Its main task is to protect Kraków against flood. On the Raba river there is situated the Dobczyce storage reservoir, the main source of drinking water for the Kraków agglomeration. Basic water management problems in the basins of Skawa and Raba are connected with water resources protection, water use and also with flood and drought protection. However, the water quality problems are most important.

Basic water management problems in the Dunajec river basin the are first of all connected with flood protection, and then with the related protection of water and land ecosystems (large legal protected areas) and the economic use of water (the Rożnów-Czchów cascade of storage reservoirs on the Dunajec river).

The state of work on program and planning documents directly and indirectly connected with water management

These constitute two groups of documents: the first one is being developed by local governments, the second one – by water management authorities united within a government structure. These documents are at regional level: of the Malopolskie Voivodeship and of the Upper Vistula region.

Documents developed by the voivodeship local government (group 1):

Document 1.1: "The Malopolskie Voivodeship Spatial Development Plan" (finished 2002). It contains documentation of surface waters and groundwater, protected areas with related waters, and specifies development conditions and barriers connected with the present state of the water management including flood protection. Moreover, it specifies general directions of activities in this area, and, based on water administration data, specifies planned actions and structures necessary for implementation of these activities.

Document 1.2: "The Malopolska Development Strategy" (under development). It is a document that coordinates the development in all areas. The water management problems are included in the social-economic block (near the spatial and political blocks) in the field "Sustainable management of environmental resources" within the superior objective "Clean and safe natural and cultural environment". It is obvious that the solutions offered in the other blocks are strictly connected with the conditions resulting from the water management criteria. All the layout of the document and its theses support the integration of objectives and solutions in all social and economic spheres and at all institutional levels.

Document 1.3: "The Malopolskie Voivodeship environment Protection Programme" (developed). It specifies as superior the achievement of the following goals:

- development and implementation of the river basin management plans;
- development and implementation of the flood protection system;
- increasing basin storage capacity;
- development of an integrated regional water management information system.

Document 1.4: "Regional Operating Program (under development). It is a document determining the hierarchy of the tasks resulting from strategy implementation.

Documents developed by the Regional Water Authority (Regionalny Zarząd Gospodarki Wodnej) in Kraków (in cooperation with the Ministry of Environment) (group 2):

Document 2.1: "The Vistula river basin management plan. The Upper Vistula basin region" (under development). It is a target document (year 2009), developed in stages according to WFD harmonogram.

Document 2.2: "The Upper Vistula river basin district plan of protection against flood and drought effects" (under development, the date is not accurately specified). This document is also developed in stages. So far, the following stages have been implemented:

- institutional support which covered: institutional structure, computer-based information system and partly filled databases and 100-year flood hazard maps;
- partial implementation of the flood hazard studies in unleveed areas.

Mutual links among the program and planning documents and the achieved level of work integration

As results from the basic scope of the related documents, presented in Section 3, their mutual links are obvious. In this situation, not taking into account the time conditions of individual documents, their development should be seriously coordinated. This can be achieved in a mixed system that unite, according to the needs, two approaches:

- developing parts of the problems by either party (according to their competency) for the needs of both documents;
- joint developing or agreeing these elements in both documents that require this.

The up-to-date practice in this scope does not meet the above assumptions both because of the lack of regulations and experience and also because of overlapping of schedules and the necessity of very intensive work of relatively narrowly specified teams.

A solution to this problem introduced lately have been agreements related to:

- determining the principles of mutual contacts and document (proposal) flow and the information databases, especially from water administration to regional documents;
- participation of representatives of water management side (administration and scientific society, including e.g. the author of this paper) in regional commissions receiving and approving group 1 documents;
- informal contacts, making opinions about solutions, etc.

Next step necessary for uniting the approaches and verification of solutions includes undoubtedly the introduction of formal rules for information exchange (numeric data, descriptive data, especially referring to development scenarios).

Significant progress in work integration will be undoubtedly influenced by the stage at which, according to WFD, the programs will be developed on measures aimed at achieving environmental objectives in water management. The stage will be preceded by discussion and some settlements on defining environmental objectives under local conditions, which considerably accelerate the regional dialog.

Moreover, activities in water management were started on gaining an adequately high level of public consultation of the proposed solutions including dissemination of information. This is new in the branch. The consultations make conditions for the formal aspect of acceptation of the river basin management plans, and, first of all, accelerate the practical integration of activities.

Identification of problems to be solved vs. up-to-date experiences

First of all, the role played by European regulations and national conditions should be determined, and then the most important deficiencies should be determined in the proper realization of the integrated programming and planning in water management. Main topics from this scope are synthetically represented below in the form of questions.

What do European regulations introduce?

- criteria of "goodness" in the form status assessments and criteria of achievement of desired environmental objectives (water protection);
- general recommendations for proceeding including types of preferred strategies and the rules of selection of the measures solving a given problem (flood protection);
- general procedures for proceeding divided into groups with determination of time schedule of work implementation (river basin management plans)

What do national conditions determine?

- structural, institutional and branch competence;
- valid legal limitations;
- financial mechanisms;
- procedures for programming and planning of social and economic development, water management and flood protection;

What is lacking?

- clearly defined methods and procedures (including intersectoral connections) that are necessary for meeting the requirements of European water policy during the development process;

The most urgent needs for fast but systemic, unique and permanent reduction of the mentioned deficiencies can be summed up in two hierarchically formulated steps:

I step: strategies of proceeding, practical recommendations (best practices), reference books and guidelines specifying standards and criteria (basis for implementation), related to:

- Conditions and water resources allocation principles (large storage and water diversions);
- Managing rainwaters and floodwaters (river waters) in urban areas;
- Protection and maintenance of rivers, storage reservoirs and lakes;
- Protection of drinking water source areas;
- Achievement and/or maintenance of the desired environmental objectives in the law-protected areas.

II step: legal acts ensuring that the requirements of the modern water policy are put into force, aimed at supporting mechanisms:

- in organizational and procedural aspects;
- in economic aspects;
- in financing undertakings

within the frames of the structures responsible for development.

To show methodological part concerning standards which is necessary and requires quick elaboration, the focus was put on surface waters. In the scope of protection of water ecosystems and flood protection, it is urgent to determine the criteria of maintaining or departing from natural conditions, to meet the following two goals:

- river protection and maintenance;
- flood protection in developing areas undergoing urbanization or already urbanized.

These standards shall be defined such as to meet the criterion of integrated programming and planning solutions for achieving these goals. The up-to-date planning system was of directive character and was based on the sectoral approach. The integrated planning should not only meet the criterion of socializing this process but also it should meet specified methodical requirements. For example, below are shown a related possible solution that defines the related problem in quantitative categories of flow/discharge.

The present approach:

Sectoral approach			
Protection of waters – understood as protection of ecological status of these waters	Flood protection understood as protection from the riverside	Rainwater carried out through sewer	Possible activities increasing natural storage capacity

Required approach:

Integrated approach
Flood protection of inundation areas: maximum protection (e.g., $Q \geq Q_{1\%}$)
Flood protection of riverbanks: buffer zone of water ecosystem (e.g., $Q \geq Q_{10\%}$)
Flood protection of riverbed: morphology, biology (e.g., $Q > Q_{50\%}$)
Protection of ecological value including: water amount, biology of water, morphology and physical and chemical properties (e.g., $Q \leq (Q_{av} - Q_{50\%})$)

Within this context, below are defined main problems deciding on the task hierarchy within this scope and on the choice of "the first step":

- the problem of discrepancy of the present scale of solving water management tasks, and the scale at which development decisions are made – in regional dimension and local dimension (deciding about development financing);
- the problem of information accessibility, verification and exchange (measurement data and processed data) – adjusted to the above space scale;
- the lack of the basic standard methodics in water branch, adjusted to the above, supporting decision making at the contact point of water policy goals – economic development.

The following needs may be considered *as "the first step"* in accelerating the progress of work on overcoming these problems:

- building databases within the framework of the programming and planning supporting system. First of all, this should concern measurement data and processed data, verified and properly processed, also correctly concentrated or interpreted for interpolation and extrapolation. The last recommendation concerns especially processing point information into linear or spatial and concerns measurement geodetic data (DTM, point and vector data), meteorological and hydrological data;

- developing basic standardized methodics adjusted to the spatial scale of a problem (large scale, e.g. up to 0.5 km^2; medium scale, e.g., up to 25 km^2, and small scale, over 25 km^2) in the range of: connecting meteorological and hydrological data, methodics of river hydraulics calculations to support designing solutions in river engineering, standards of biological assessment of surface waters' state connected with morphologic assessment of riverbeds state, developing definitions and rules of formulating spatial and quantitative flood risk, standards of economic analyses of water management undertakings' efficiency, and the activities in the area of spatial development to improve protection of waters and flood protection.

This scope of work has the character of basic initial works. Experience gained and strengthened in such a way will allow to formulate correctly guidelines for detailed legal conditions of this cooperation and integration of work and solutions.

Summary

The presented state of works on integrated water management planning process in region suggested that problem in Poland is open in following main points:

- integration of environmental objectives (criteria of solution standardization on difference levels, for example for protection of waters and flood protection)
- organization process and economic and financial mechanisms.

Institutional cooperation in region must be concentrated mainly on determining the hierarchy of water management tasks which will support the regional development. Experience gained and strengthened in such a way will allow to formulate correctly guidelines for detailed legal conditions of this cooperation and integration of work and solutions

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Simulation Model for Multipurpose Water Management System with Two-Step Usage of the Water Resource

Ljupcho Petkovski

Abstract: The effectiveness of the use of multipurpose water management system on a two-step usage of the resource can be assessed by investigating the response of the system to given hydrological input during exploitation. This class of problems in management of water systems with a reservoir can be solved accurately only by using the sequential method with vector input, which means with the use of modern numerical methods. In this paper are given the fundamental principles and the algorithm of the simulation model for dual-purpose water resource system. In this system, the users are prioritised as primary (water supply) and secondary (energy generation). The key characteristic of the management model is that the secondary user must respect the operation of the reservoir in accordance with the needs of the primary user. The model is applied to determine the energy characteristics of a micro hydro power plant on the Loshana hydro system in Delchevo, Macedonia, and the results are presented as an illustration of the model.
Keywords: Simulation model, Multipurpose water management system

1. Introduction

Energetic analysis of hydro power plant, or determination of energy potential in exploitation period, in hydraulic engineering practise, is accomplished by using of two methods (USACE ED, 1985.; USACE ED, 1987.). These methods are: (a) conventional approach with flow-duration method (FDM) and (b) modern approach with sequential stream flow routing method (SSRM). Hydrological information will be condensed into cumulative probability curve of discharges, if the FDM is used. However, realistic order of discharges will be lost, which is often disallowed for management tasks with hydro power plants (HPP) with accumulation. Therefore, modern numerical methods of SSRM by utilising of computers (Petkovski L., 1998.; Petkovski L., Dodeva S., 1999.; Petkovski L., 2000.), which are based on computations on chronological set of discharges are more applicative, in comparison of conventional methods (Petkovski L., Tančev L., 1998.; Petkovski L., Tančev L., 2001.; Petkovski L., Tančev L., 2003.b).

On the following figures 1 and 2, some usual schematics for incorporation of HPP at double-used water resource system (WRS) are presented. The WRS is constructed for supply the needs of priority water user (domestic or industry, or irrigation). These configurations are very often in Republic of Macedonia, where more then one hundred accumulation exists for water supply of population or agriculture. The part of civil works for small HPP, as adaptation of valve house, could be done with neglected investments, almost for all of these hydro systems. The correct assessment of energetic production of small HPP, by respecting the needs of priority water user, is essential from analytical point of view. The selection of optimal capacity of small HPP would be according to criterion - maximisation of net benefits, calculated by Present Worth Method (James D., Lee R., 1971). The costs of electrical and mechanical equipment will have dominant influence on this criterion, for these hydro systems.

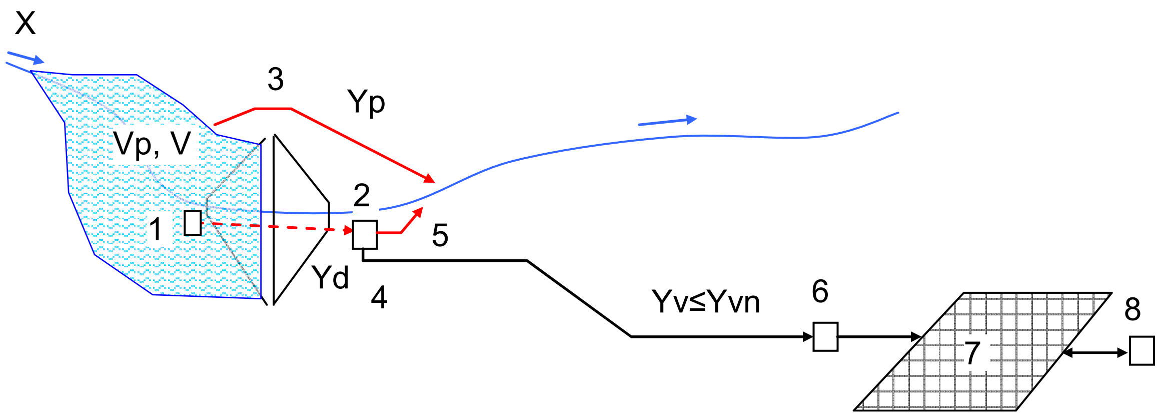

Figure 1 Schematic of WRS for water supply. 1 – intake structure in accumulation, 2 – valve station, 3 – spillway, 4 – conduit for water supply, 5 – bottom outlet, 6 – filter station, 7 – distribution network, 8 – tank. X = inflow, Vp, V = volume of accumulation at the beginning and at the end, Yp = spillway discharge of accumulation, Yd = conduit discharge, Yv = supply discharge, Yvn = normal needs of water supply

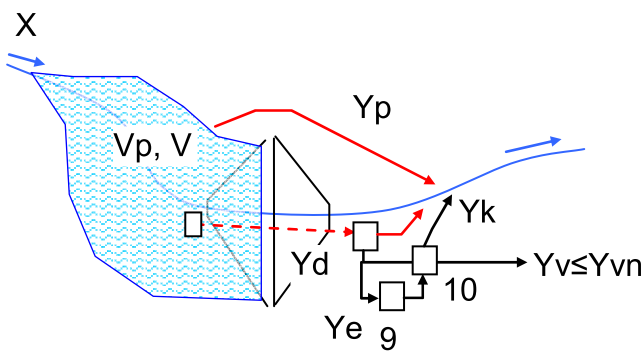

Figure 2 Schematic of WRS for water supply and energetic. 9 – power house, 10 – tailrace, Ye = discharge throw turbines, Yk = spillway discharge of tailrace

2. Mathematical formalisation of simulation model

In the following examination of response in the exploitation period, of double-user WRS (water supply and energetic) of storage HPP, by given hydrological loading, sequential method was applied. This model is adaptive to adopted configuration of the hydro system, to accomplish the research for influence of level of capacity on energetic effectiveness of HPP. According to management procedure, the mentioned sequentional method is from the class of simulation models. These models include selection of operation rules with the reservoir, based on reduction on outflows for water supply on drought periods. Therefore, these models are different from the category of optimisation models, because here is not applied normative method from mathematical programming, which directly lead to optimal trajectory of states of reservoir (Wurbs R.A., 1994.).

The basic principles of mathematical model for energetic analysis of storage HPP is presented in the following text. The invariable input data are as follows: D = diameter of conduit pipe [m], L = length of conduit [m], Rn = roughness coefficient, Sumk = sum of coefficients of local loss, Zdv = altitude of tailwater [masl], eta_g = generator coefficient of efficiency, n_god = period of analysis [years], ah = working of conduit for water supply purposes [hours/day], Vmin = minimum volume of accumulation [m^3], Vmax = maximum volume of accumulation [m^3], Vkor = Vmax-Vmin = useful storage of reservoir [m^3], Xd(i,j) = average monthly inflow in accumulation [l/s], Yvnd(i,j) = average monthly normal need for water supply [l/s]. The variable input data are listed next: QminT ≈ 0.4*(Qins/n_tur) = minimum discharge throw one turbine [m^3/s], QmaxT = Qins = maximum discharge throws n turbines or throw HPP [m^3/s], Vp = volume of accumulation at the start of the analysis [m^3], n_tur = number of turbines. The algorithm of the simulation model is stated as:

- con1=3600.*Td(j) is constant,
- Td(j) is number of days in "j" month [days/month],
- Yemax = QmaxT*24.*con1 is maximum possible monthly discharge throw HPP [m^3/ m],
- Vrel = (Vp-Vmin)/Vkor is relative starting fulfilling of useful storage,
- red=Φ1(Vrel) is reduction coefficient of water supply, figure 3,
- X=Xd(i,j)*24.*con1/1000. is monthly inflow into reservoir [m^3/ m],
- Yvn =Yvnd(i,j)*24.*con1/1000. is monthly normal need for water supply [m^3/ m],
- Yv =red*Yvn is monthly reduced need for water supply [m^3/ m],
- Y=Yv is supposed release from reservoir [m^3/ m],
- V=Vp+X-Y is computed volume of accumulation on the end of analysed time frame [m^3],
- Adopted volume of accumulation on the end is determined by following conditional relations: (1) IF (V<Vmin) THEN V=Vmin, (2) IF (V>Vmax) THEN V=Vmax,
- Y=Vp-V+X is computed summarized outflow from accumulation [m^3/ m],
- Yp=0.0 is supposed monthly spillway flow from reservoir [m^3/ m],
- Ydmax=MAX(Yvn,Yemax) is maximum possible monthly discharge at conduit to HPP [m^3/ m],

- Monthly spill-flow is computed by the following conditional expression: IF (Y>Ydmax) THEN Yp =Y-Ydmax,
- Yd=Y-Yp is monthly discharge at conduit to HPP [m^3/m],
- Yk=0.0 is supposed monthly spill-flow from tailrace of HPP [m^3/m],
- Monthly spill-flow from tailrace of HPP is computed by the following conditional expression: IF (Yd>Yvn) THEN Yk =Yd-Yvn,
- Yv =Yd-Yk is monthly discharge at conduit to treatment plant [m^3/m],

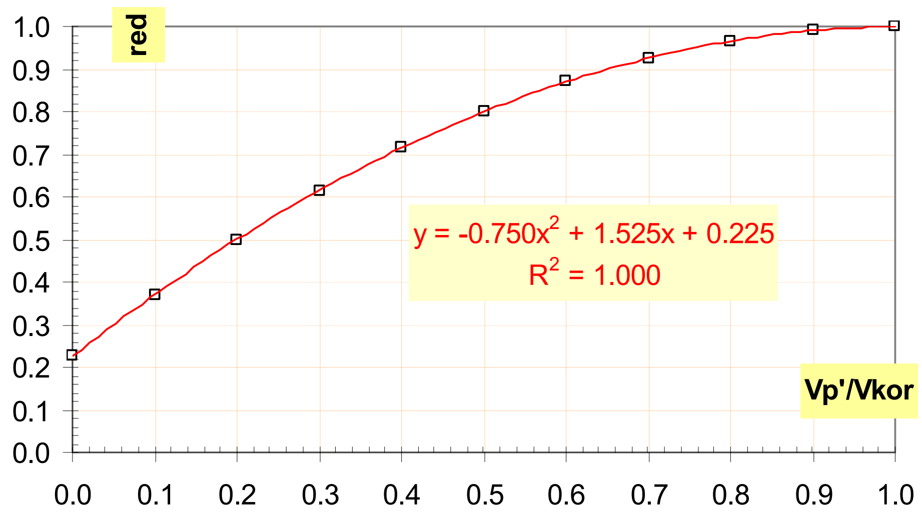

Figure 3 Reduction coefficient of water supply in correlation of relative starting fulfilling of useful storage

- For determining of average monthly - Qh [m^3/s] secondly discharge throw HPP and Th [h/d] hourly operating of HPP per day, is used the following modulus of conditional formulas: (1) IF (Yd<QminT*ah*con1) THEN Qh=0.0 and Th=0.0, (2) IF (Yd>QminT*ah*con1) AND (Yd.≤QmaxT*ah*con1) THEN Qh=Yd/(ah*con1) and Th=ah, (3) IF (Yd>QmaxT*ah*con1) THEN Qh = QmaxT and Th=Yd/(Qh*con1)
- Vsr=(Vp+V)/2 is average monthly volume of accumulation [m^3],
- Zgv=Φ2(Vsr) is average water level elevation in reservoir [m asl],
- R = D / 4 hydraulic radius [m]
- n = 0.011 is roughness coefficient of still pipes,
- $y = 2.5*n^{0.5}-0.13-0.75*R^{0.5}*(n^{0.5}-0.1)$ is coefficient in Pavlovski's formula,
- $C = R^y / n$ is Chezy's coefficient in Pavlovski's formula,
- $\lambda = 8*g / C^2$ is Darcy's coefficient,
- $vv = Q_H / (\pi*D^2 / 4)$ is velocity of flow at conduit [m/s],
- g = 9.807 [m/s^2] is earth accelerate,
- Σξj = sumk is sum of local coefficients of pressure loss (intake, grid, curves, valves),
- ξf = λ*L/D is friction coefficient of pressure loss,

- dh = (Σξj+ ξf)*vv^2/(2*g) is pressure loss throw conduit [m],
- Hn =Zgv-Zdv-dh is average monthly net head of HPP [m],
- qq =Qh/QmaxT is a relative discharge of HPP,
- Turbine coefficient of efficiency is calculated by the following condition equations and suitable diagrams: (1) IF (n_tur=1) THEN eta=Φ3_1(qq), (2) IF (n_tur=2) THEN eta=Φ3_2(qq), (3) IF (n_tur=3) THEN eta=Φ3_3(qq). In this paper, only the diagram eta=Φ3_3(qq) is presented on figure 4.

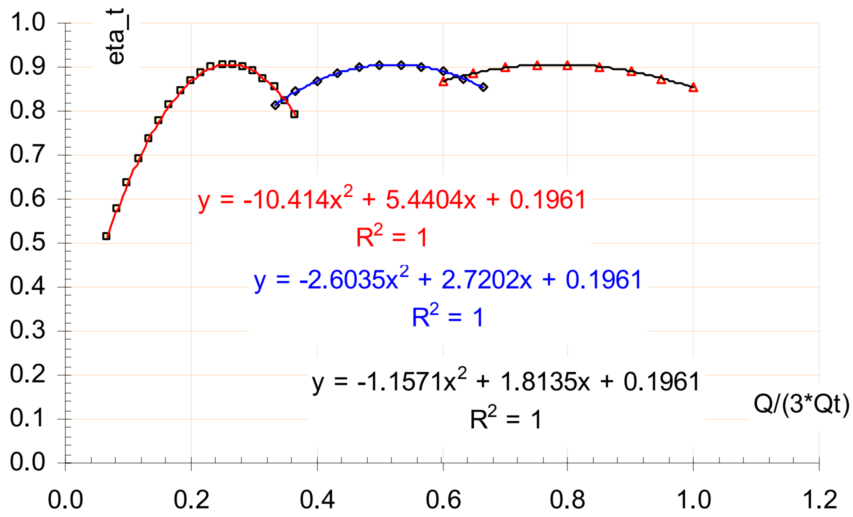

Figure 4 Relation of turbine efficiency coefficient from relative discharge throw HPP, for variant with three turbines, 0.1≤q1<0.35, 0.35≤q2<0.633, 0.633≤q3≤1.0

- Pt =9.807*eta *Hn *Qh is average monthly power [kW],
- Em =eta_g*Pt*Th*Td(j) is average monthly generation of electric energy [kWh/m],
- Average yearly production of electric energy, for period of n_g years E_Y [kWh/y] is equal on:

$$E_Y = \frac{\sum_{i=1}^{ng}\sum_{j=1}^{12} Em_{ij}}{n_g} \tag{1}$$

3. Application of simulation model on micro HPP Loshana

The economic validation of energetic effectiveness of HPP Loshana - Delchevo is adopted to be assessed by average yearly production of electric energy, which is achieved by using of representative synthetically set of hydrological data. This approximation is adopted because the analysed HPP is storage power plant on mountain stream of relatively small catchment area, where dependable power of HPP is neglectable.

The basic data of HPP Loshana are systemized into tree categories. In the first one, the unchangeable values of HPP parameters are included: D = 1.0 [m], L = 190.0 [m], Rn = 0.011, Sumk = 0.25, Zdv = 780.0 [m asl], eta_g = 0.97, n_god = 40 [years], ah = 18 [h/day], Vmin = 33,800.0 [m^3], Vmax = 1,160,000.0 [m^3], Vpoc = 800,000.0 [m3], ng = 40 [years]. Second, average monthly discharges of: inflows (for period 1951÷1990, figure 5) and water supply needs (for period 2001÷2040), and topographic characteristic of accumulation. Third, the values of HPP data which distinguish the different variants of the system: install discharge Q_{ins} [m^3/s], number of turbines N_{tur} [-], and minimal discharge throw turbine Q_{tur}^{min} [l/s].

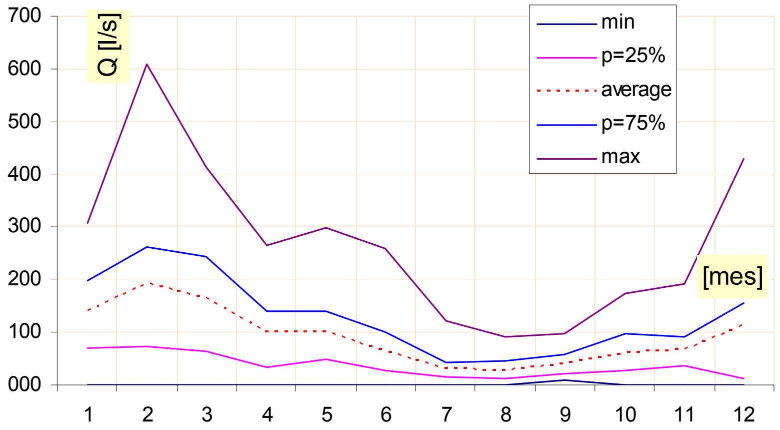

Figure 5 Probabilistic corridors of average monthly inflows in accumulation, with average multiyear discharge of Qsr = 91.6 l/s

We could conclude that water supply needs of Delchevo are admitted on satisfaction level, according to chosen operative rules of accumulation, which are presented by coefficient of reduction (figure 3). From analyse of duration curve of relative water supply (figure 6), the constatation is that fulfilling of normal requirements for water supply, with neglectable reduction of yvr = Yv / Yvn = 0.9, is provided in time domain P = 90 %.

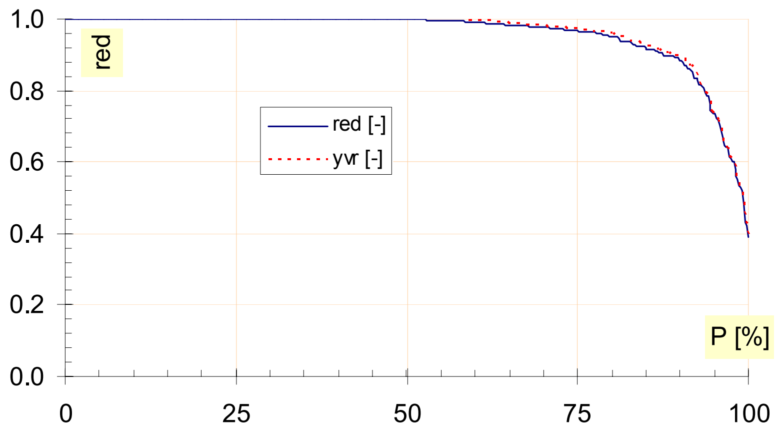

Figure 6 Duration curves of: coefficient of reduction of water supply red = Vp' / Vkor, and relative discharge on water supply yvr = Yv / Yvn

These operation rules of reservoir, which provide to achieve water supply in accordance to adopted design standards, are the same for energetic analyse independent from the level of install capacity of HPP. Hydropower is secondary water user of the hydro system, and here the effect of double usage of water resources is applied. However, the management of hydro system is according the respecting of primary user - water supply of population. Therefore, hydropower does not change the regime of using the waters, but only exploits the available hydraulic head of water flow at valve house downstream the dam site. In this case, the hydraulic head is not necessary for water supply purposes. Namely, before the dam was constructed, water supply was carried out from Tyrol's intake structure on the location of the dam profile. The final product from energetic analysis is energetic characteristic of HPP - yearly production of electric energy in relation of installed discharge, (figure 7). This is the basic relation to assess the yearly benefits from the HPP, therefore it is the key characteristic of succeeding technical-economical analysis to determine of optimal HPP capacity.

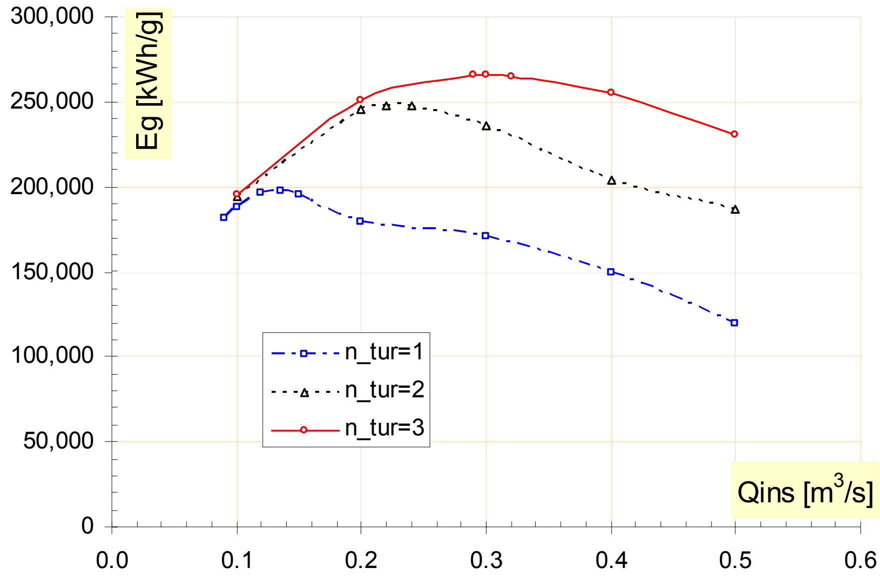

Figure 7 Yearly production of electric energy in relation of installed discharge, the influence of different number of turbines in power plant

Summary

From the energetic characteristic of HPP, we could notice that to a defined value of installed discharge, the increase of installed discharge is followed by increase of annual generation of electrical energy. This relationship is due to decrease of energetic unused flows throw the spillway. The decrease of yearly generation of electrical energy, after defined value of installed discharge, is subjected by worse management capability of HPP. Namely, for bigger values of HPP capacity, because of the conditions of the constant discharge to cover water supply needs and the limitation of minimal discharge throw the turbine, the HPP could not be operational. This effect will not be achieved on storage HPP without primary user, where to satisfy the minimal of turbine discharge, at the dry months, simply, the time of working hours per day will be decreased. Some improvement of HPP energetic effectiveness is achieved by increasing the number of turbines. This measure leads to improvement (or decrease) of down limit of HPP operation.

From the application of simulation model for management of multi-user water resource system with storage HPP of analysed system Loshana (Delchevo, Macedonia), zones, where optimal install discharge should be looking for, are clearly defined. So, the energetic analysis for variant with one turbine $n_{tur} = 1$, point out that the interval of HPP capacity $Q_{ins} = 80 \div 140$l/s should be researched for optimal capacity. For the higher HPP capacity, about the variant with one turbine $n_{tur} = 1$, because of worse management HPP capability for smaller discharges, the yearly generation of electricity decrease. That is why this region is not relevant for technical-economical analysis, because here we have only increase of the costs (according to increase of HPP capacity). Similar results are obtained for the variant of two turbines $n_{tur} = 2$, where the interval of $Q_{ins} = 150 \div 220$ l/s is actual for more precise optimisation, or for the variant $n_{tur} = 3$, where the interval is $Q_{ins} = 210 \div 300$ l/s. Therefore, with energetic analysis only, independently of succeeded technical-economical analysis, the space where the optimal solution is looking for, could be relevantly reduced.

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Integrated Water Management in Croatia

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Abstract: Integrated water management as the basic approach to global action in the field of water policy has a relatively good framework in Croatia, due to its tradition and current water management. Organized water management in Croatia based on regulated water law already started in the second half of the 19th century. Hydrographic administrative units, a certain level of public and user participation and institutional and financial independence are all traditional characteristics of the Croatian water management system.
Water as a common good is discussed in the Croatian Constitution and placed under a special protection of the Republic of Croatia. The relevant legal provisions contain some of the basic preconditions of integrated water management – hydrographic division into water districts and river basin districts, water management planning and operating at the level of hydrographic units, and water charges as economic instrument of cost recovery in water management.
The application of the concept of integrated water management and implementation of the EU Water Framework Directive require certain changes and modifications at the legislative level, in particular with regards to the enforcement of legislation, and should also include the separation of competences, the strengthening of institutional framework and the role of the general public. To this end, the development of the national water management strategy is under way as a basis for the system improvement and the planning of water management activities in the following two decades. This planning document expresses general interests, needs and priorities of different sectors and fields in terms of water regulation, use and protection, while complying with the basic principles of integrated water management, and proposes potential measures for its improvement.
Key words: water management in Croatia, integrated water management, development of water economy, implementation of EU Water Framework Directive

1. Introduction

Organized water management based on regulated water legislation appears in the area of today's Croatia in the 19th century. Over time, the contents and methods of water management practice have changed, depending on the political circumstances and dominant socio-economic issues of a period. The water sector in Croatia has always been one of the major drivers of construction and other related activities. Through preparation, construction, equipment, maintenance and operation of large water structures and systems it has significantly contributed to the economic development in general.

Initially, the dominant water management activities included those related to flood protection and water drainage from agriculturally valuable surfaces (mid-19th century). This was followed by more intensive activities related to the use of water and hydropower generation (first modern pipelines in the second half of the 19th century, first hydropower plant in 1895, and first major fish farming at the beginning of the 20th century). Later on, urbanization, industrial growth and tourism gave prominence to the provision of water services to the population (water supply and wastewater disposal), as well as to water protection for particular purposes. In the previous decade, environmental protection aspects have been gaining ever greater significance in the implementation of water management.

At present, direct activities in the field of water are performed by water management, municipal utilities and business entities for whom water is the basis for their production of commercial goods and provision of services (as raw material, capital goods, or recipient for wastewater).

2. Water management in Croatia

2.1 Legal and institutional framework

In the paramount document of the Republic of Croatia, the Constitution, water is mentioned as a resource of special interest, in the same group with the sea, air space, minerals, land, forest and other goods and resources. In legal sense, the water sector is governed by the *Water Act*, which "regulates the legal status of water and water estate, the methods and conditions of water management, the method of organizing and performing of water management tasks and functions, basic conditions for carrying out of water management activities; powers and duties of Government administration and other Government bodies, local authorities and other legal subjects, and other issues of importance to water management." The funding of the water sector is regulated by the *Water Management Financing Act*.

Pursuant to the Water Act, "*Water is a public resource which, because of its natural properties, cannot be anybody's property*." The right to water abstraction for various uses is based on concessions or, in certain cases, water rights permits, the exception being the right to general water use. The land particles for which special conditions of use and availability are proscribed due to the need for regular and undisturbed maintenance of watercourses and other water bodies, implementation of flood protection and other activities related to the securing of adequate water regime form the *water estate*. The Water Act provisions refer to all terrestrial surface water and groundwater, including mineral and thermal water. Additional to terrestrial water, in certain cases the Water Act provisions refer to coastal waters as well.

Apart from the Water Act and the Water Management Financing Act, individual water related provisions are also found in the legislation which regulates physical planning and environmental protection, as well as economic activities such as forestry, agriculture, inland navigation, utilities and power supply.

Croatia has a tradition of water management within natural hydrographic units. The country is divided into four water districts: the water district of the Sava River Basin, the water district of the Drava and Danube River Basins, the water district of the Littoral and Istrian River Basins and the water district of the Dalmatian River Basins. Decisions on determination of water district boundaries take into consideration, with minor exceptions, the water divides of the Sava, Drava, Danube and Adriatic Sea Basins. Territorially smaller water management units are the sub-basins or catchment areas (34), which include one or more catchment areas of minor watercourses. Due to having common, inter-related water issues, water system and economic conditions, one water management system is provided for the whole group.

Figure 1 Basic water management units

The entities competent for water management activities in Croatia are: the Parliament, the Government, the National Water Council, the Ministry of Agriculture, Forestry and Water Management – Water Management Directorate, Croatian Water as well as the other bodies of public administration and units of local and regional government and self-government.

The National Water Council is a body which, at the highest level, coordinates different interests and views on systemic issues related to water management. Most obligations and competences are under the Ministry of Agriculture, Forestry and Water Management – Water Management Directorate, which performs administrative and other expert tasks in the field of water management.

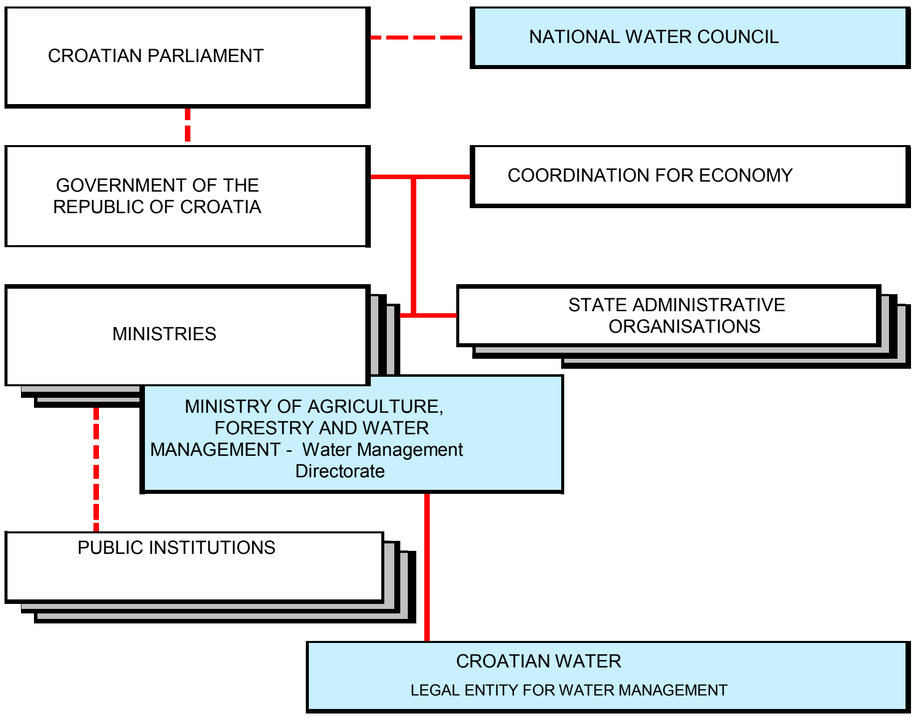

Figure 2 Position of water management in state administrative structure

Although the Ministry of Agriculture, Forestry and Water Management – Water Management Directorate is the primary body for the performance of administrative tasks in water management, some responsibilities are allocated to other public administration bodies as well, such as the Ministry of Environmental Protection, Physical Planning and Construction, which performs activities related to the general policy of environmental protection and the fulfilment of conditions for sustainable development; the Ministry of the Sea, Tourism, Transport and Development, whose activities also include the implementation of general measures of protection and rational use of the sea and space along the coast as well as the organization of the major infrastructure works of interest to sustainable development of the Republic of Croatia; the Ministry of Health and Social Welfare, which is competent for administrative and other expert activities related to the sanitary inspection and sanitary quality of foodstuffs and objects of general use, including water intended for human consumption.

Croatian Water is the legal entity for water management, established pursuant to the Water Act, for the purpose of "*constant and undisturbed performing of public services and other tasks accomplishing managing of water to the extent determined by the accepted plans and in accordance with the funds provided for such purposes on the basis of laws and regulations based on the laws*." It performs its activities on the entire territory of the Republic of Croatia and covers all water districts and river basins. Apart form its head offices in Zagreb, Croatian Water has water management departments for water districts, which include water management sub-departments for catchment areas.

Units of regional and local government and self-government (counties, towns and municipalities) have competences and responsibilities generally relating to water issues of their areas, but also to water issues of the river basin to which they belong.

2.2 Base water management documents

The obligation to develop the National Water Management Plan of the Republic of Croatia (NWMP) is prescribed by the Water Act, whereas the definition of conditions with regards to the contents and procedures of adoption is prescribed by the *Regulations on the Development of the National Water Management Plan of the Republic of Croatia*. The NWMP is enacted by the Parliament of the Republic of Croatia. A water management plan or scheme must be developed for each individual river basin, which then serves as a basis of water management in a water district. These documents are baseline information for awarding concessions and issue of water right acts. Pursuant to the Water Act, other planning documents which serve as a basis for direct operative work are adopted as well, such as the *National Flood Protection Plan* (determines conditions and methods of organization and implementation of immediate flood protection), *National Water Protection Plan* (identifies activities of implementation of water protection from pollution and contamination), A*nnual Water Management Plans*, and relevant local-level plans.

Apart from planning documents in water management, planning and development documents which, to a higher or lesser degree, deal with water issues, are passed in areas outside water management as well, such as the strategy and programme of physical planning, strategy of environmental protection, development plans for the internal navigation system, and similar documents in other sectors, which either significantly depend on water, or have a significant impact on water. Their harmonization with the NWMP is proscribed by law.

Base information for water management also includes other planning, technical, administrative, legal, financial and other documents, and data, information and maps on other resources. A number of institutions outside water management collect data of importance to water management, which are exchanged via reports, studies, decisions, preliminary, project and detailed documentation, and cartographic overviews.

Data and information on water resources and systems and their users form major baseline information for the development of water management plans. The Water Act proscribes the establishment of an integrated Water Information System. The obligation of data and information collection and maintenance is also proscribed by other legal acts, regulations, standards, international agreements and conventions. The Environment Act proscribes the establishment of the environment information system, with the water information system as one of its topical, dislocated centres.

2.3 Implementation of water management plans

Water management is established at the national level and encompasses activities of public importance, for which funds are not secured by market methods. It does not directly produce any commercial output; it, however, provides vital public services, thus improving the general living conditions both for the population and for the development of a number of socio-economic activities and ecological functions. In this way, it indirectly creates results within the results of other sectors and areas which depend on water and the regulated water regime, through the following major activities:

- regulation of water and protection from flash floods and torrents – construction, technical and economic maintenance of regulative and protective water structures, amelioration drainage;
- water use – water research works, co-financing of construction of municipal water structures, amelioration and irrigation systems;
- water protection from pollution - monitoring and control of pollution and contamination, co-financing of construction of municipal water structures.

Unlike water management, municipal water utilities are organized at the local, exceptionally also regional, level, for activities where the market mechanism is limited. This includes water use (construction, maintenance and operation of drinking water supply systems) and water protection (construction, maintenance and operation of sewerage and wastewater treatment systems).

The connection between water management, municipal water utilities and economic water use is established by the Water Management Financing Act, which regulates the system of water charges collected by Croatian Water as its revenue.

The economic segment of water use (hydropower generation, irrigation, aquaculture, aquatic tourism, etc.) is based on market principles, i.e. water use (as raw material, capital goods, or recipient for wastewater) produces goods or services for which adequate market price is charged. Economic subjects are granted certain rights on water, based on which they independently plan, construct and maintain water structures and use water in appropriate manner.

In water management, for which Croatian Water is the primary operator, profit is not created on market principles - no products are produced, no direct services are provided for which it is possible to charge the equivalent price from persons considered water system users. These users, however, are the recipients of the effects of watercourse regulation, flood protection and protection of water from pollution, which are activities of public interest and as such regularly financed from fiscal sources at the national and local levels, or from autonomous contributions or charges of parafiscal nature, such as the state budget, budgets of local self-government units, water use charge, water protection charge, gravel and sand extraction charge (from renewable deposits in areas important for the water regime).

3. Basic water management guidelines

3.1 Assessment of water management situation

Croatia belongs to a group of relatively water-abundant countries, with a large proportion of external and transit water. Its water resources are used in a moderate manner, and at present their sustainability is not threatened. Water quality is mostly preserved, with no major changes in the previous decade (on the contrary, some areas have experienced improvements).

About one billion m^3 water is abstracted annually for different purposes (water supply, fish farming, irrigation, etc.), which is less than 1% of the total water resources, i.e. under 4% of endogenous water. Based on the specific consumption of both the population and industries, Croatia shares the European average. There is, however, unevenness of spatial distribution and characteristics of water resources in comparison between the Black Sea and the Adriatic River Basins. The most part of the Black Sea River Basin, particularly eastern Croatia, although lacking in endogenous water, is extremely rich in transit water. The Adriatic River Basins are more independent of exogenous water, and thus easier to manage; however, due to precipitation regime and karstic characteristics, it has significantly less favourable conditions for water protection and use. The majority of precipitation falls in the periods with the lowest needs, so the generated runoff in natural conditions either accumulates in underground retentions or rapidly disappears into the sea. Most constructed reservoirs are located in the Adriatic basin, which greatly improves temporal and spatial usability of water resources.

The safety level from floods for towns and settlements is somewhat lower than the European standard. Public water supply systems are relatively developed, and include approximately 76% population. Development level and efficiency of the system and measures of water protection change depending on the area; however, protection levels are generally unsatisfactory. The construction level of the wastewater and rainwater collection system is insufficient (connection rate to sewerage systems equals 42%), with particularly insufficient construction level of wastewater treatment plants (about 25%).

Other water uses (industries, hydropower generation, irrigation, fish farming) have in the previous 10 to 15 years mostly stagnated, or even decreased.

The activities aimed at developing the National Water Management Plan (NWMP), initiated several years ago at Croatian Water - Institute of Water Management, have been an incentive to analyze the present status of water management. For this purpose, comprehensive study documentation has been developed, where data and information on the status and development potentials of Croatian water management have been systematized, as well as a number of expert studies elaborating in detail individual thematic areas relevant for the synthesis of future water management plans.

Water management planning documents in Croatia traditionally cover smaller catchment areas by selecting the most favourable water management solutions. Due to a change in the approach during the development of the NWMP, a lack of an articulated national policy and a strategy with insufficiently clear development objectives and goals as well as undefined environment in which the future water sector would function, the development of the document suffered delays. It became obvious that it was necessary to first determine general social objectives and priorities in the field of water, and only then planning could start at the lower, technical level, which is what is traditionally meant by a water management master plan. Based on such experiences and insights, the competent state body passed in 2003 the Regulations on the Development of the National Water Management Plan of the Republic of Croatia, which to a certain extent redefined, i.e. specified the contents, method and procedure of enacting such planning document. According to the Regulations, the NWMP consists of two parts (i) Strategic Water Management Master Plan, and (ii) Water District Management Plans. The process is under way of adoption of the I. part of the NWMP – Strategic Water Management Master Plan, which is a framework planning document aimed at the determination of starting points and guidelines for integrated Water District Management Plans, and the creation of legal and institutional preconditions for their development and implementation. Among other issues, the document analyzes the present water management framework, primarily in relation to the requirements originating from the EU water legislation, in particular the EU Water Framework Directive.

3.2 Proposals for improvements of water management framework

Sustainable development is the strategic orientation of the Republic of Croatia. In water management, this means the adaptation of the traditionally sectoral water management and the adoption of the integrated concept of water management, which starts out from the totality and limitation of water resources, and enables optimal and sustainable distribution of these resources to the existing and potential users (population, economy, ecosystems). Three complementary interest groups are taken into account, belonging to both present and future generations: social justice and equality (all individuals and social groups have the right to water and basic water services), social well-being (water is a development resource and should contribute to the general future prosperity and economic development) and ecological sustainability (vital water and water dependent ecosystems need to be protected). In line with such orientations, the policy and development strategy of water management are planned, which shall, on the entire national territory, in the long-term, rational manner:

- secure sufficient quality drinking water for population water supply;
- secure necessary water quantities of adequate quality for various economic uses;
- protect people and material goods from adverse effects of water;
- protect and improve the status of water as well as water and water dependent ecosystems.

The proposed changes are primarily targeted at creating the preconditions for the implementation of the principles of integrated planning and management of water districts, which is anticipated by the EU Water Framework Directive as well. The Strategic Water Management Plan anticipates the improvement of the present water management, and thus also fulfilment of the principal goal emphasized in the EU WFD - the achievement of good status of all waters. Particularly important are the following issues:

- secure sufficient quality drinking water for population water supply;
- integrated river basin management and care for all water resources in order to permanently satisfy user needs, achievement of safety of citizens and their property as well as environmental protection (river basin management planning must also include transitional and coastal waters);
- consideration and coordination of all sectoral requirements in water management and mitigation of conflicts of interest among shareholders in the water system (improvement or establishment of institutional forms for inclusion of direct users into the planning process);
- awareness-raising and compliance with the principles of ecological sustainability when initiating all future projects related to water management;
- development of the legal framework for cost recovery from service users in the water system, while taking into account socio-economic impacts;
- clear distinguishing between public and private interests, and directing state funds to the implementation of projects of general interest (due to the relatively law construction level of water infrastructure, state investments and other subsidies into the water sector are still necessary);
- democratization of water management, i.e. inclusion of all interested shareholders into the planning and decision-making process, and monitoring of the implementation of adopted plans (improvement of institutional forms of public participation into the process of planning and monitoring of implementation of plans);
- improvement of regional and bilateral cooperation in the planning and implementation of plans;
- improvement of the management framework (legal, institutional, financial, information, planning, operative, control and supervising), in line with the existing tradition and changes the country undergoing;
- securing stable, sufficient funding of water management activities, particularly the works on water infrastructure of public interest.

4. Conclusion

The traditional framework of water management in Croatia has already legalized some basic principles of integrated water management, which are implemented in practice, to a higher or lesser degree. The strategic national document for the water sector, which is in the adoption procedure, proposes certain changes in the present framework. They primarily relate to the harmonization with the international legislation and adopted international obligations, in particular those of the European Union.

It is expected that the adoption of the European principles, tools and instruments, and their introduction into the Croatian water management practice will be a step forward towards the strengthening and updating of the integrated approach to water management. The general objective of all proposed activities and measures is a rational, sustainable use of water and other resources, and financial resources invested in the water regime regulation and the provision of adequate quantities and qualities of water for various developmental and environmental uses.

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Opportunity Study of the Danube Region – the Adriatic Traffic Corridor

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Duška Kunštek, Marko Pršić

Abstract: Croatia has a very advantageous traffic location in the centre of Europe. It is located on the main European traffic corridors, among others, on the VII Danube and the X Sava corridor. They can be connected with the Adriatic through the corridors Vb and Vc. Existing traffic system in Croatia is inappropriate to meet most recent technological demands, as well as economy needs for synchronization of the European traffic network and philosophy. The future of Croatian inland navigation is to include Danube into a "combined river-railway traffic corridor Danube region-The Adriatic". It starts in the Danube port of Vukovar and ends in the Adriatic port of Rijeka. The Combined river-railway traffic corridor Danube Region-The Adriatic will consist of the future multipurpose Danube-Sava canal between Vukovar and Šamac, improved Sava waterway from Šamac to Sisak, followed by Sisak-Zagreb-Rijeka railway which exists already. Three possible development scenarios for the riverine part of the corridor have been observed. The first scenario includes river training of the Sava River toward the IV navigation class, the second scenario includes training of the Sava River as a IV class waterway with the multipurpose Danube-Sava canal (Vb class) and the third scenario includes the canalized Sava River as a Vb class waterway with the multipurpose Danube-Sava canal. The considered economic aspect includes navigation, use of water power, agricultural aspect, forestry, water management and environmental aspect for each scenario individually. On the basis of the scenarios, corresponding cost and benefit scheme has been determined as well as investment analysis which includes profit and other financial criteria. Performed multi-criterion analyses state precise propositions for realization of each scenario respectively.
Keywords: traffic, corridors, river-railway, waterway, multipurpose canal

1. Introduction

Croatia has an extremely advantageous traffic and geographical location by being situated on the main European traffic corridors which, accordingly, includes the VII European (Danube) corridor which is linked with via the Adriatic with the corridor Vb, Vc and X (Sava). Existent traffic system in Croatian does not comply with modern technical demands, nor does it meet the needs of the economy sector. It has not taken advantage of meeting positive preconditions in order to be incorporated into and technically coordinated with the European network.

The future of Croatian navigation lies on the Danube River and in the sphere of international traffic via the combined river-railway traffic corridor Danube Region-The Adriatic from Vukovar to Rijeka.

The combined river-railway traffic corridor Danube Region-The Adriatic would include the following:

- 61.4 km long future multipurpose Danube-Sava canal between Vukovar and Šamac
- 306 km of the Sava River from Šamac to Sisak
- 280 km of the railway track Sisak-Zagreb-Rijeka.

Economic surrounding of the traffic corridor – navigation, water power system, and aspects of the corridor in the field ecology, agriculture, forestry and water management – were observed. Analysis of investments, costs and benefits evaluation, and economic and financial evaluation was performed on the basis of the costs and benefits per scenario. Scenarios were proposed following a multi-criteria analysis.

2. Development scenarios

Three possible scenarios for the development of the riverine part of the traffic corridor were analysed and they are the following:

- Scenario 1 regulated Sava with the IV navigation class from Jamena to Sisak
- Scenario 2 regulated Sava with the IV navigation class from Šamac to Sisak via the Dunav-Sava canal with Vb navigation class
- Scenario 3 canalised Sava with Vb class from Šamac to Sisak via the Dunav-Sava canal with Vb navigation class.

Scenarios are mutually independent which means that subsequent scenarios do not incorporate preceding one(s) as a specific earlier stage. Though some segments of technical solutions are overlapping in different scenarios, the tree possible scenarios presented herein contain different technical and traffic solutions each of which has a different level of costs and benefits.

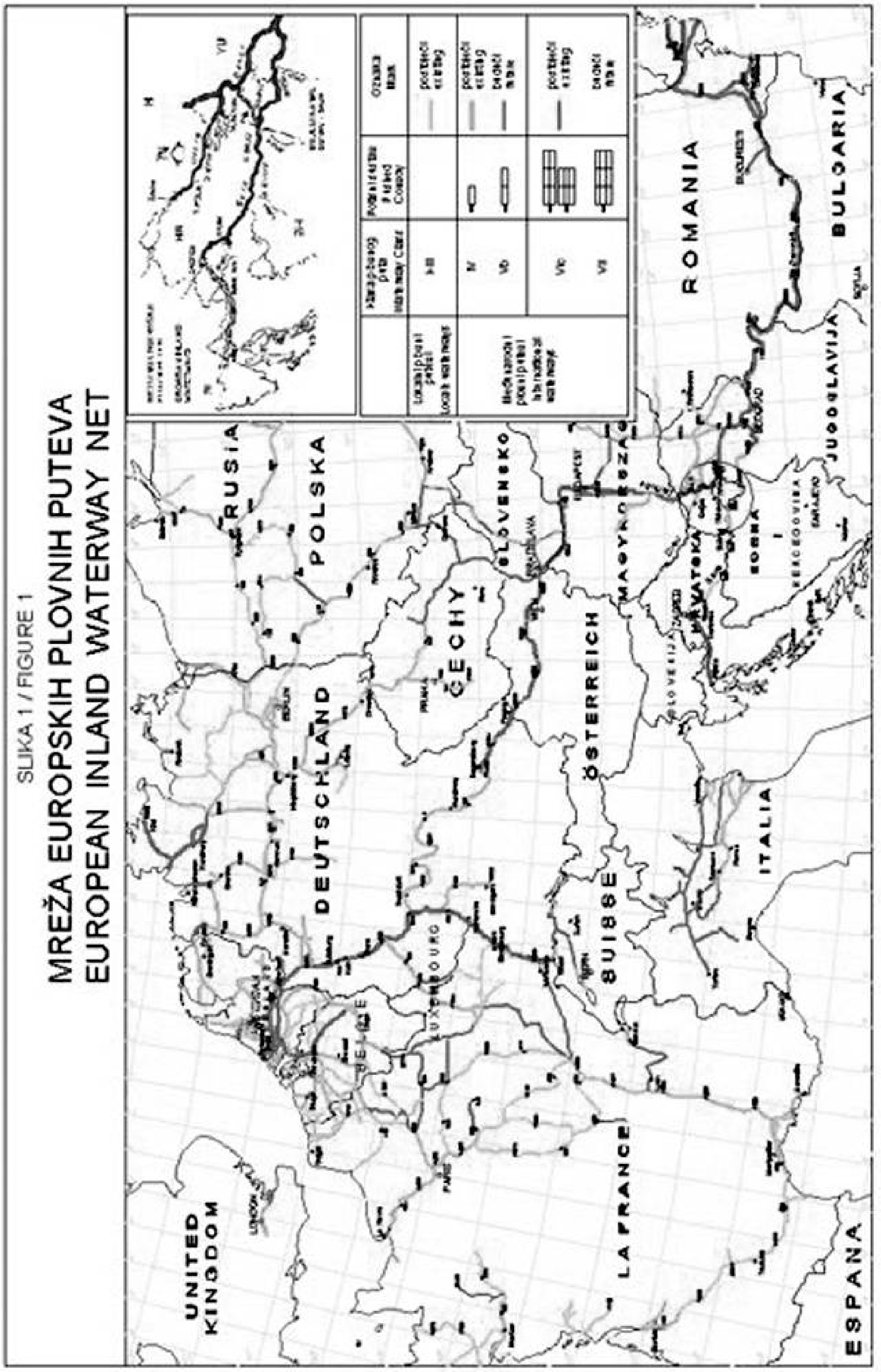


Figure 1 European inland waterways and the indicated corridor

2.1 1st Development scenario

TECHNICAL SOLUTION OUTLINE

This scenario simulates the state which excludes construction of the multipurpose Dunav-Sava canal. Connection to the European inland waterway would be established via Serbia and Montenegro, as done so far.

Elements for tracing the Sava waterway are: the IV class waterway, two-way navigation secured where possible, a minimum waterway radius of R_{min}=360m (on locations where river morphology prevents it, a smaller waterway radius and one-way traffic will remain), a minimum waterway gabarit of 2.2x70 m is secured on the water table 95% of days annually.

ECONOMIC AND WATER MANAGEMENT SOLUTION OUTLINE

Following arrangements pursuant to this scenario, the whole Sava River from Jamena to Sisak would become a IV class waterway without limitations in terms of radius of curvature (one-way traffic). Ecologically speaking, works do not impose any changes to existent habitat nor do they jeopardize it. Furthermore, there will be no changes of the existent drainage system. There is also no possibility for securing sufficient amount of irrigation water from Sava due to minimum summer flows of natural water regime. Technical solution pursuant to this scenario does not effect forestry production. This scenario presumes that protection against floods of the Sava River will be solved by combined regulation of the water bed and water regime which is currently being used. Existent and planned development of this type of flood management system of the Sava River will not change following regulation of the Sava waterway pursuant to Scenario 1. Basic protection measures are relief canals, gates, retentions and dikes. This stage does not enable securing additional amount of water to compensate water deficit during drought periods. Regulation of the waterway on this level does not guarantee a secured water intake from Save for technological waste water, so conditions for existent intakes remain the same.

2.2 2nd Development scenario

TECHNICAL SOLUTION OUTLINE

This scenario simulates connecting the Sava waterway with the international inland waterway net through the Republic of Croatia via the future multipurpose Dunav-Sava canal along Šamac-Sisak track.

Elements for tracing the Sava waterway are: the IV class waterway, two-way navigation secured where possible, a minimum waterway radius of R_{min}=360m (on locations where the river morphology prevents it, a smaller waterway radius and one-way traffic will remain), a minimum waterway gabarit of 2.2x70m is secured on the water table 95% of days annually.

Elements for tracing the Dunav-Sava canal are: the Vb class waterway, two-way navigation, a minimum waterway radius of R_{min}=750 m, a minimum waterway gabarit in the direction of 4x34 m dimensions will be secured by the net present value (NPV).

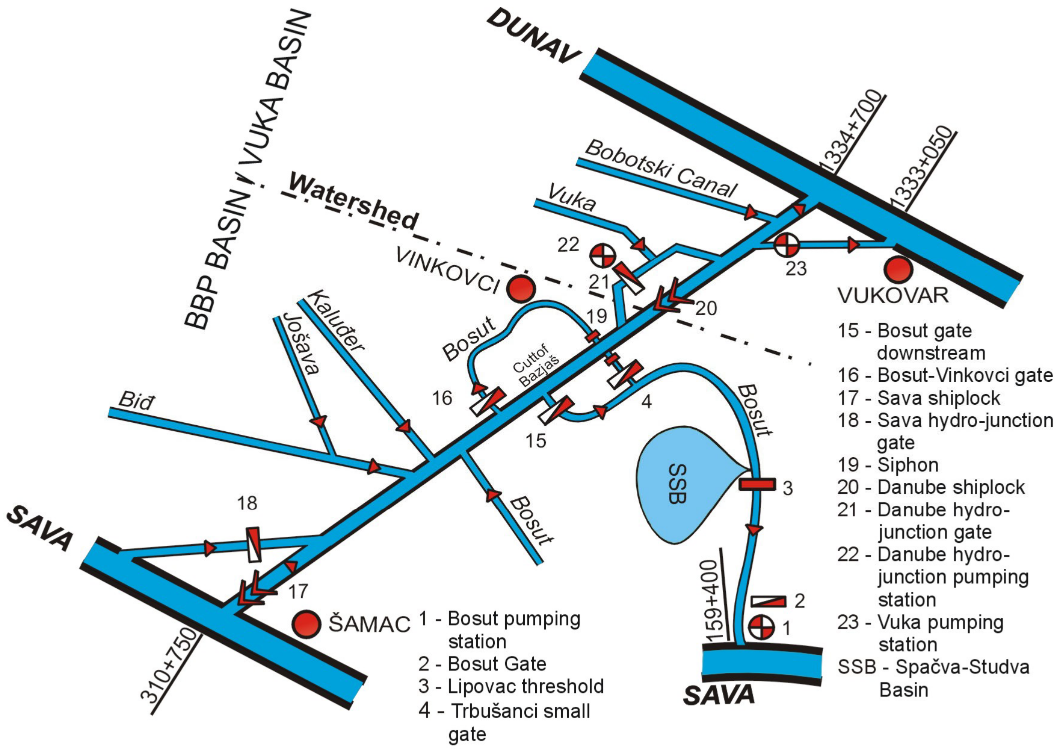
ECONOMIC AND WATER MANAGEMENT SOLUTION OUTLINE

The part of the waterway on the Sava River (from Šamac to Sisak) is described in Scenario 1. The multipurpose Dunav-Sava canal in the length of 61.4 km will be constructed for a standard international Vb class waterway. Navigation from Sava towards Western Europe will be performed via the Dunav-Sava canal the length of which would be 417 km and towards Eastern Europe the length of which would be 85 km.

An essential ecological contribution of the canal to the existent system is the following: during low water level periods in the canal basin, the water intake from Sava will include releasing of 30 m^3/s water into the canal in addition to the quantity necessary for irrigating agricultural grounds. Hydro-technological facilities, i.e. gates will be used to release 5 m^3/s water into Bosut, and 4.5 m^3/s water into Vuka. Furthermore, it is possible to perform controlled recharging of forest basins in view of simulating natural habitat of the dominant and most valuable common oak. The forestry study has shown that regulating water regime in common oak forests along the canal can increase timber mass three times in comparison with the current status which represents a measurable ecological contribution. On the Biđ-Bosut segment of the basin of the Dunav-Sava canal, construction of the Dunav-Sava canal enables connecting that basin with inland waterways as well as supplying it with water from Sava by means of gravitation or pressure in case of small water bodies, or from Danube by pressure alone. On the segment of the basin of the Dunav-Sava canal where the Vuka River flows, there will be no changes: the canal does not enable irrigation, and drainage will go directly into Danube as was the case before construction of the Dunav-Sava canal. The Dunav-Sava canal can contribute significantly to forestry production because it can enable forest irrigation by controlled increasing of the level of groundwater in the forests of the Biđ-Bosut plateau. The forestry study has indicated that regulating water regime in common oak, ash and hornbeam forests can increase timber mass for 10% and general forest function for 20% (during production cycle of 120 years which implies a cycle from cutting to a full grown oak forest). Construction of the canal will enable occasional controlled releasing of water from the Sava River into the Dunav-Sava canal via a gate on the part of the canal on Sava, and, subsequently, into the water release system in the riparian part of the canal. These waters will make up for major summer water deficits on the lowlands of the Biđ-Bosut basin which threaten the existent ecosystems located therein.

PROTECTION AGAINST FLOOD

In future, digging the multipurpose Dunav-Sava canal via a watershed (from 10th to 18th canal km, Figure 2) will connect the basins of Biđ-Bosut and Vuka thus creating a canal basin the surface of which will be 4000 km^2. Connection of the basins of Biđ-Bosut and Vuka will be controlled by a shiplock [20] and a gate [21] of the Danube hydro-technical junction, and connection of the canal and Sava by a shiplock [17] and a gate [18] of the Sava hydro-technical junction (60th canal km). Accordingly, of the total canal length of 61.5 km, 10 km will be in the Danube regime, 1.5 km in the Sava water level regime, and the major part in the length of 50 km in the regime of the canal with a continuous water level of +80 m over the sea level for the most part of the year. Vuka and the Bobot canal will be drained directly into Danube (as prior to construction of the canal), and following its construction, one part of Biđ-Bosut floods as well.



Slika 2 Basin drainage scheme – future state

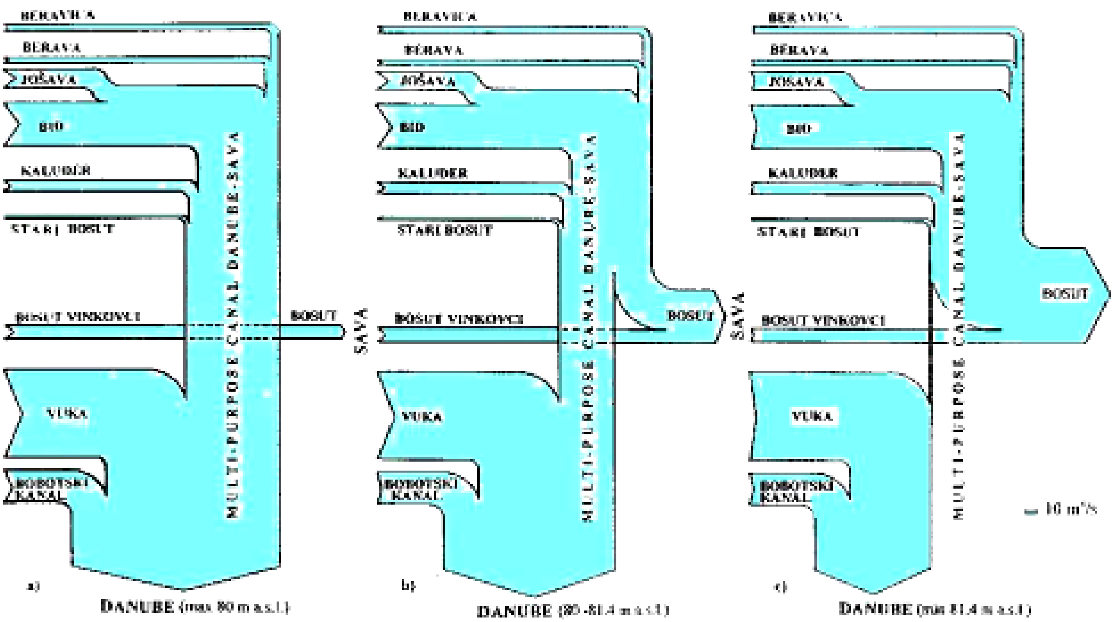
Floods of Biđ, Jošava, Kaluđer and Bosut flowing in the direction of Vinkovci will be distributed via the Bosut gate located downstream [15] into the downstream Bosut river channel towards Sava and via the canal towards Danube. In this manner Vinkovci will be protected from floods. Major part of floods will be directed towards Bosut and released by means of gravitation into Sava via the gate [2] located at the river mouth of Bosut into Sava. If, due to a high water level of Sava, it is not possible to enable gravitational runoff of Bosut via that gate, Bosut will be pumped into Sava with a pumping station. If the Bosut inflow surpasses the capacity of the pumping station, a specific amount of Bosut will be released into the retention of the Spačva-Studva basin. It is anticipated that gravitational runoff of the remaining amount of floods via the canal into Danube is performed on the Danube hydro-technical juncture via the gate [21] (Figure 3b). From there, water is released in a controlled manner downstream into the river channel of the canal belonging to the Danube regime. This scheme is applied for Danube water level from +80 m over the sea level to +82 m over the sea level. The drainage scheme “everything into Danube” is possible when the water level of Danube is equal to or below the water level in the canal (+80 m over the sea level)(Figure 3a).

In case of a high water level of Danube, over +82 m over the sea level, complete drainage from the Spačva-Studva basin region is directed into Sava via the gate and the pumping station located on the rivermouth of Bosut into Sava, as the case before construction of the Dunav-Sava canal (Figure 3c). Pursuant to hydrological analyses, this case reoccurs once every five years or once every other year when VES N. Sad (hydroelectric power station) is constructed. Since according to the proposed future hydro-technological solution for basin drainage of the Dunav-Sava canal, water from Vuka will be separated from water of Biđ-Bosut by means of a gate [21] and a shiplock [20] in the Danube hydro-technical junction, the possibility of flooding of the Biđ-Bosut basin with waters from Vuka is removed owing to deceleration of the high waters of Danube.

2.3 3rd Development scenario

TECHNICAL SOLUTION OUTLINE

This scenario of the traffic corridor Danube - Region-The Adriatic, as well as Scenario 2, simulates connecting the Sava waterway with the international inland waterway net, i.e. Danube, through the Republic of Croatia.



Slika 3 Drainage scheme depending on hydrological conditions for Q_{25y}

Hence, this scenario focuses on the Sava River in the length of 290 km from Sisak to Šamac (the rivermouth of the multipurpose Danube-Sava canal into Sava) and the future multipurpose Danube-Sava canal between Šamac and Vukovar in the length of 61.4 km. Sava waterway will be shortened for ca 40 km from its used length of 290 km because certain number of tunnels will be dug on meanders having too small radiuses. In this manner this scenario incorporates the total of ca 311.4 km of waterway in Croatia. It includes regulating

Sava to Vb navigation class and a complete construction of the Dunav-Sava navigation canal of Vb class. Observed scenario is in accordance with navigation regulations stated in the UN Study from 1972. However, technical parameters concerning waterway have been updated. Sava will be canalized with two water steps: Šamac and Košutarica (Jasenovac).

Elements for tracing the Sava waterway are Vb class waterway, two-way navigation secured along the whole length of the route, a minimum waterway radius of R_{min}=450 m, a minimum waterway gabarit of 3,7x89 m will be secured on the water table 85% of days annually, with a radius of curvature $R \geq 700$m (i.e. in less curved segments of Sava, the width of the waterway can be narrower).

ECONOMIC AND WATER MANAGEMENT SOLUTION OUTLINE

Technical solution for Sava according to this scenario has several purposes. By canalising the river with two water steps. Šamac and Jasenovac, the following will be improved: quality of waterway, protection against flood and irrigation when compared to the same functions pursuant to Scenario 1 and 2 which incorporate only regulation of the river channel. Furthermore, this scenario incorporates the function of water power management of Sava. Construction of water steps changes water regime of Sava, in case of low and medium water level, whereas during high water level there will be no major changes to the water regime. This is caused owing to the feature of water steps which in case of low and medium water level, keep deceleration only in the current river channel of Sava, i.e. below the level of existent surrounding ground. Water appears in inundations only in case of managing floods as is the current state of affairs. In this manner the existent flood protection system for the central part of Posavlje does not change. Solution to the multipurpose Danube-Sava canal corresponds completely to the one described in Scenario 2.

3. Project costs and benefits

Pursuant to the needs of the above stated technical features, the following estimation of costs has been made:

Construction scenario	Project description	LAND REDEMPTION [€]	[€]	EQUIPMENT REPLACEMENT COSTS within 100 ys. [€]	ANNUAL COSTS [€]
Scenario 1	Regulation of the Sava waterway to 4^{th} class	0	14.093.781	0	422.813
Scenario 2	Regulation of the Sava waterway to 4^{h} class with the multipurpose Danube-Sava canal	13.393.333	744.259.247	0	8.292.457
Scenario 3	Regulation of the Sava waterway to 5b class with the multipurpose Danube-Sava canal	13.875.613	1.242.257.610	358.012.775	23.010.524

Basic benefits achieved by realisation of this project are the following:

- establishing a new traffic route and new means of transportation which represents a major step forward for economy, connects Croatia with the global market and creates conditions for the development of local economies along the entire length of the corridor;
- regulating the water regime of Sava and its tributaries along the major part of the riverbank, complete control of floods with a 100-year payback period, establishing safe development conditions, increasing land and assets value;
- developing power supply system;
- securing safe water sources for irrigation, technological needs and improving small watercourses;
- improving water supply owing to a continuous bioremediation of groundwater;
- increasing agricultural profits, increasing production assortment, guaranteed yield;
- increasing water regime of the forest ecosystems, increasing timber mass yield;
- maintaining and improving natural conditions of wetlands by securing water during summer periods, developing special branches of tourism, hunting, fishing, sports and recreation activities;
- increasing employment rate in numerous spheres analysed herein, as well as additional activities during construction works and when put into operation.

4. Economic and financial evaluation of the project

Since evaluation of the project is primarily performed by banks interested in monitoring financing of the project, the evaluation of economic and financial analyses includes the following:

- Economic flow and payback period;
- Net present value (NPV)
- Internal rate of return (IRR)
- Possible credit arrangements
- Financial flow.

Financing model is concluded by three important interested parties, i.e. the Republic of Croatia, strategic partners and financial institutions prepared to participate in the realisation of the project.

In view of controlling and evaluation the validity of the project, additional criteria have been included. Since this is an infrastructural project of a strategic importance for the Republic of Croatia interconnecting 7 counties, beside the stated evaluation criteria, investment criteria was also included in the economic and financial evaluation of the project because it directly triggers increase in gross national product (GNP) of the region. More precisely, according to this criterion, the threshold for minimal acceptable value of investments for Croatia has been estimated to 4 billions HRK (e.g. Scenario 1 does not comply with this condition).

Interests of strategic partners are presented through the internal rate of return (IRR), and minimal acceptable amount of the rate is estimated to 5%. According to this criterion, all three Scenarios meet the condition. However for this type of project it is desirable to find an acceptable interest (< 4%) on the money market. Interests of financial institutions are present through the value of the debt-service coverage ratio (DSCR) of 1.15. In this case, all Scenarios meet this criterion.

Investment period for Scenario 1 is 15 years, with annual investments from 4 to 14 billions HRK. With this minimal investment, the expected traffic should also give minimal profit which means that the payback period is rather long, 20.24 years. This also testifies to the claim that the corridor should interconnect Sava and Danube in Croatia, shorten freight routes towards Europe which would make this inland waterway more attractive.

Investment period for Scenario 2 is 12 years, with annual investments from 250 to 740 billions HRK. Since this Scenario incorporates construction of the multipurpose Danube-Sava canal, major investment connotes numerous benefits – project profits. Though the payback period is 23.81 years, it should be borne in mind that this is an infrastructural project which will bring about other invaluable benefits beside valorised and identified ones.

Investment period for Scenario 3 is 15 years, with annual investments from 500 to 900 billions HRK. This maximal scenario brings about versatile direct and indirect benefits. Accordingly, the payback period of 24.5 years can be regarded acceptable since this is an infrastructural project of national importance.

5. Conclusion

Pursuant to the above stated the Republic of Croatia should choose between Scenario 2 and 3. Owing to major investments, both Scenarios have a potential to become the basis for economic development of the region along the length of the corridor. To that effect, it is important to protect interests of the local industry and secure its partial or significant recovery by its direct participation in realisation of the project.

The first step in decision making process is to reach strategic decision on the Scenario in order to launch negotiations for realisation of the project. By accepting the selected Scenario it is no longer possible to change allocation of benefits established by that specific Scenario; however pending negotiation period it is possible to gain additional benefits. The next step is a tactical decision on the variant of the selected Scenario owing to changeable parameters of each Scenario: ownership share, credit grace period, interest rate and manner of payment, duration of credit payback period, issuing guarantee and total duration of concession. .

Incentives with which the Republic of Croatia is trying to revive economic activity today are necessary but not enough for this kind of development. The State should have a strong ally in commercial banks. If this is not the case, the State should achieve its macro-economic goals in the sphere of production, employment, stability of prices and net export through the mechanisms of monetary, fiscal, income and foreign trade policies.

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Theme II

Hydraulic Engineering and Environmental Impacts

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Water Management Structures Functionality Assessment in the Light of Environmental Requirements

Antoni Bojarski

Abstract: Rivers and streams of the Upper Vistula river showed in 20. century an excessive erosion that reached 2–2.5 m and often more. This phenomenon is adverse to the economy and environment, e.g., uncovers and scour the river regulating structures and engineering structures situated in the river bed, makes water intake functioning impossible and causes the necessity of water damming, lowers the groundwater table in river valley bottoms. Stopping this process requires that changes in water management have to be made by changing water management structures' functioning. The changes have to include the required environmental objectives concerning among others river morphology and conservation of proper flows. Diversified range of changes in water management structures' functioning is conditioned upon the existing spatial development and economic demands. Identification of the problems in the catchment of the Upper Vistula river is presented in the paper. Directions of change and types of action to improve the situation taking environmental objectives are proposed.
Keywords: Vistula river, river regulation.

Introduction

According to Water Framework Directive (WFD), ecological status of surface waters is basically the measure of the results that were caused by man's activity in these waters. All activity aiming to improve the ecological have to go through defining the environmental objectives that are feasible to achieve. The assessment of the existing state in watercourses is one of the elements necessary for defining the objectives that are feasible to achieve. All large engineering structures situated in river beds and nearby and, especially, water management structures with their spatial layout and function, have significant adverse impact on the ES of surface waters. To ensure the best possible ES of surface waters, all activities mitigating the adverse impact of the existing and planned water management structures situated in river beds should be implemented. The problem is of special importance for the rivers of the Upper Vistula river which showed in 20. century an excessive erosion that reached 2–2.5 m and often more. This phenomenon is adverse to the economy and environment and increases the flood hazard [Wyżga 2001]. For example, this problem results in that:

- the river regulating structures and engineering structures situated in river beds are uncovered and scoured;
- water intake functioning is made impossible and involves the necessity of water damming;
- groundwater table in river valley bottoms is lowered.

Stopping this adverse process requires that changes in the approach to water management functioning and planning have to be made. The problems concerned with water management functioning (except for storage reservoirs) will be discussed in the paper.

Characteristic of the problems of rivers in the Upper Vistula river catchment

The average development level of the Upper Vistula rivers and streams is estimated at 28% and fluctuates in the range of 8÷60 % for tributaries and is about 19% for Vistula. After the 1997 flood the development has increased, however with unchanged approach which insufficiently takes into account the environmental requirements. The life of regulation structures is rather short; after the 1997 flood half of them was classified to be repaired or rebuilt and the costs reach 4% per annum. For these reasons and as the tourism interest increases, it is necessary to change the approach to functioning of regulation structures which together with the shortening and narrowing of riverbeds are important causes of losing again by the structures their stability and of other adverse consequences. Other factors facilitating riverbed deepening are considerably lower sediment transport from the catchment and river regulation technologies which makes riverbeds flat and destroy the bottom sediment structure and the bottom stone structure.

Threat to biodiversity

Deep incisions of Carpathians rivers and streams in 20th century and many engineering activities including river regulation, construction of bridges on these watercourses of roads in their nearby surroundings, also pose significant threat to the organic world of these watercourses and the land in their proximity. Lack of places suitable for any phase of animal growth caused by depriving the organisms of chance to get to such places, eliminates the possibility of their effective development. Periodic using the space in between gravel grains is another important factor for the sets of animals characteristic for gravel rivers. Eliminating these spaces by deposits, concreting or the incision of the bed into the rock substratum excludes the survival possibility of these organisms. Deepened riverbeds cause water rarely overflows the riverbanks, which limits the exchange of biogenic substances in the valley and, in consequence, leads to the considerable limitation of the composition and diversity of fauna and flora species. An important element of biodiversity is ensuring animals access to river. Steep slopes, their stoning, make such an access or river passing by animals impossible.

Desired change directions in maintaining Carpathian rivers and streams

Stopping the adverse phenomena in Carpathian rivers and streams is important for two reasons: from nature's point of view and for overcoming the adverse trends in hydrological processes. To implement the process of slowing overland flow, it is necessary to restore the possibility of flood water storing in the flood inundation areas where rivers flow through the areas of riparian forests and agriculture lands. In urban areas, the riverbeds' formation will be more limited [Bojarski 2005]. Desired change directions should cover the following activities:

- lower the river transportation capacity by forming sinuous and wider riverbeds;
- reduce the destruction of riverbed deposit structure resulting in increasing the possibility of deposit moving during protective works;
- ensure water organisms free way through newly erected structures and make such free ways in the already existing structures;
- restore the river corridors of high diversities of fauna and flora habitats and of highly diversified landscapes.

The listed above activities are limited by many factors. Of more importance are the following:

- the proximity of development that needs to be protected;
- the boundary of property and inundation areas;
- the water management structures existing in the riverbed.

Proposing a solution for maintenance or improvement of ecological quality for individual rivers and streams, it is necessary to carry out the functional assessment of all water management structures that exist in the riverbed.

Functional assessment of the existing and planned water management structures

It is proposed to carry out functional assessment basing on partial assessments that are based on the criteria concerning:

- the levels of meeting the goal of the structure, taking into account also technical conditions of the existing structures;
- locations;

- shapes;
- impact on the environment.

The listing of the adequate criteria is presented in Table 1. The criteria are related to the gravel and large-gravel rivers of slope values ranging from 5% to 0.5% as the beds of rivers of slope values exceeding 5% are in most parts rocky.

Table 1 Criteria of structures' functionality

Structure function	Parameters of and recommendations for the existing structures	Parameters of and recommendations for the planned structures
Stabilizing water level of water intake	If the intake is active, keep unchanged, otherwise, if useless at present or in near future, partial or total demolition recommended, possibly in stages.	Consider if it is possible to take water without building a damming structure. If the structure is necessary, it should be designed to have a by-pass for fish or a proper fish-pass and ensure free way for fish along the whole length of biological flows.
Stopping bed load	If the difference between the average grain size upstream and downstream of the debris dam is considerable, or large uncovered rocky surfaces occur downstream of the dam – lower, dissect or remove, protecting the difference in slope with rock filling or ramp, or rebuild.	Apply only in the cases of proven permanent delivery of excessive amounts of bed load. Design dams with an outlet having operating openwork gates along the whole height of the structure, allowing to carry bed load of diameter up to 80-150 mm.
Reduction of river slope	For slopes exceeding 2%, do not change; for slopes ranging from 0,5% to 1%, leave to its technical death, replacing by a ramp of boulders; for slopes not exceeding 0.5%, systematically remove.	Apply for river slopes exceeding 5% as rocky structures shaped according to conic sections at the vertical and horizontal cross-sections; avoid straight lines and trapezoidal cross-sections. The structures should be consistent with the shape of fish-passes and ramps for fish migration. The structures consistent with the old troughs should have variable widths, two-channel and multi-channel cross-sections, and always adverse slope.
Free way for fish along the watercourse	Water rise at the impounding structure of up to 1.0 m, with maintaining downstream of the overflow the depth of 1.25 times the rise. Clear run through the structure; if several obstacles in series, the overflows designed as to the inlet and outlet of the structure were consistent with the main stream (thalweg) of the watercourse, also by depressions in stilling basin outlet walls and in rock fill protections. No narrow cuts into stilling basin outlet walls, no wide flat	First of all, ramps from boulders. Pool-and-weir and slot fish-passes for slopes exceeding 0.5% only, hidden in the spillway of the weir or dam and adjusted to salmon fish passing in rivers with status assessed as average, and for all fish in rivers with status assessed as good or excellent. For slopes not exceeding 0.5%, only ramps (exceptionally slot fish passes) adjusted for passing all fish. All spillways shaped according to conic sections at the

	stilling basins facilitating destruction of fish populations. No large flat concrete surfaces of uniform slope (escarpments) and long straight-line walls protecting inlet and outlet. Diversify structures' architecture adjusting to the character of the neighbouring terrain.	vertical and horizontal cross-sections; avoid straight lines and trapezoidal cross-sections. Avoid narrow cuts into stilling basin outlet walls and wide flat stilling basin facilitating destruction of fish populations. Diversify structures' architecture adjusting to the character of the neighbouring terrain and using natural (rock) material.
People's safety	Filling with rock vast places where back currents occur and where hydraulic traps occur during floods.	Slope of reinforced scarps 1:2 and smaller, basin depths not exceeding 1.5 m, circular or parabolic horizontal shape of spillway eliminating back currents and hydraulic traps.

Summary

The proposal presented in the paper of assessing the functionality of the existing and planned water management structures that take into account the environmental requirements is an important element allowing to look for variants of repair and investment solutions. Based on the assessment made according to the presented criteria, conclusion may be drawn that lead to controlled total or partial withdrawal of the structure from operation. Also, a proposal was given of directions of change in maintenance of Carpathian rivers and streams that lead to stop the adverse phenomenon of excessive erosion of their beds.

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Lech – Bed Load Entrapment Hornberg-Ehenbichl A Physical Model

Peter Hanisch, Michael Kremser, Karl Wibmer

Abstract: Because of a widely varying sediment yield and a decreasing bed slope in the longitudinal profile of the Tyrolean river Lech, the central settlement area around Reutte is often in threat by high water levels caused by sediment deposition. In the past this unfavourable bed load deposits could only be removed by multiple, ecologically disputed bed load dredgings from the river channel. The general solution for this problem is based on a centralized bed load retention and dredging zone upstream of the endangered area. Therefore the river will be divided into two channels. One channel works as a bed load entrapment only for high discharges. The second channel, forwarding the bed load according to the bed load transport capacity of the endangered downstream river reach, works like a bypass to the deposition zone and remains untouched by the dredging. Due to the complex discharge and bed load distribution between the two channels the numeric simulations had to be complemented by a physical model. This model mainly focused on the discharge and bed load distribution using a special diversion structure. Furthermore a system of dikes and groynes in the upstream reach and in the bypass had to be optimised to channel the bed load transport.
Keywords: bed load retention zone, bed load dredging zone, physical model

1. Introduction

In June 2000 ca. 41 km² of the valley floor of the river Lech was nominated as "Natura 2000"-reserve. Despite various river training measures throughout the 20^{th} century the river was able to maintain most of its dynamic and natural river ecosystems. But because of a widely varying sediment yield from the upper catchment area and a decreasing bed slope in the longitudinal profile the central valley around Reutte is often in threat by high water levels caused by sediment deposition. Together with some other measures concerning the sediment management the bed load entrapment Hornberg-Ehenbichl is intended to solve this problem.

2. Presentation of the Problem

The basics for this bed load entrapment were set by a detailed hydrologic, hydraulic and morphologic analysis of the current status followed by an extensive study concerning the sediment budget of the entire Tyrolean Lech from 1992 to 2001. The substantial results are conclusions about the geology of the catchment area, the developing of the characteristic grain size, the bed deposition and erosion, the mean bed load transport capacity and the mean, annual bed load sediment balance with consideration of the flood events in the investigation period.

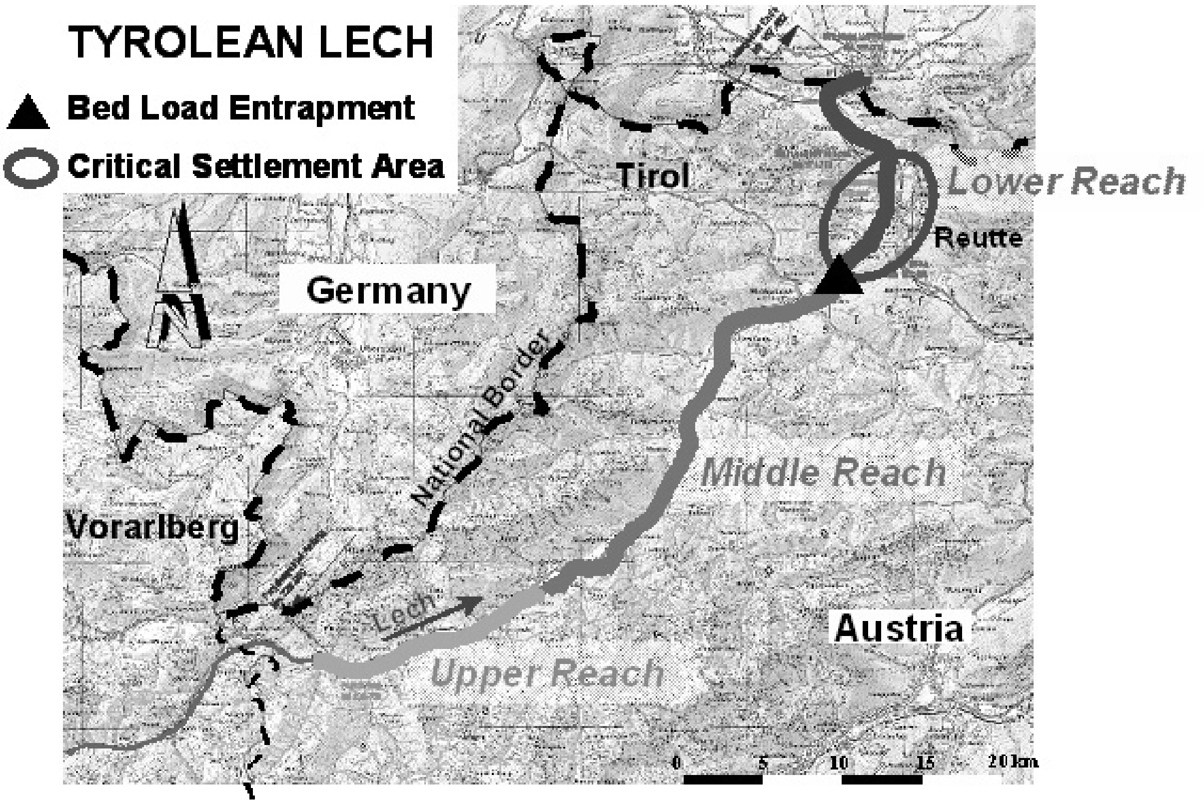

Figure Nr. 1 General map of the Tyrolean Lech

According to a rough classification and characteristic the bed load sediment balance of the river Lech can be divided into three sections.

In the bigger part of the *upper reach* the river bottom is stable. The river follows a more or less straight course.

The *middle reach* is in threat by erosional processes. To stop this negative trend specific measures have already been taken. Because of that and some extreme hydrologic situations in the past few years the bed load sediment balance in the middle reach became much more stable and a reversal of the erosional process can be recognized.

In the *lower reach* flood events caused unfavourable bed load deposits in the critical settlement area around Reutte. The morphological dynamic disequilibrium between the middle and lower river reach expresses through the reduction of the total head line and the sediment transport capacity. Furthermore in large parts of the lower reach the average cross section doesn't provide enough buffers for changes on the river bottom. The sediment deposits reduce the effective flow area and the freeboard becomes smaller. To ensure the flood safety in the central settlement area in the past years this unfavourable bed load deposits could only be removed by multiple, large-volume, ecologically disputed dredgings form the river channel.

3. Goals of the project

The Natura 2000 directives demand that technical interventions within river habitats have to be completely banned or limited to a minimum. Thus, in future only one construction upstream the critical settlement area should solve the actual problems concerning the bed load sediment balance and flood safety. After discussing and evaluating several options it was decided to build a bed load entrapment, which should meet the following requirements.

- Only a part of the arriving total bed load should be excavated. The other part should be forwarded to ensure the morphologic dynamic equilibrium in the downstream river reach
- The actual multiple, widely distributed excavations will be concentrated to the site of the bed load entrapment
- The dredging should not affect the free flowing water body
- The flow continuum is not to be interrupted
- Demands concerning ecological, urban and regional planning have to be taken into account

4. Solution

The general solution for this problem is based on a controlled bed load retention and dredging zone (= bed load entrapment) upstream the endangered settlement area. Therefore the river will be divided into two channels simulating to the natural, braided river morphology. One channel works as a bed load entrapment only for high discharges. The second channel, forwarding the bed load according to the bed load transport capacity of the downstream river reach, works like a bypass to the deposition zone and will remain untouched by the dredging. Thus, the concept is intended to fulfil both, the morphological dynamic equilibrium of the downstream river reach and reduced human interferences with the river ecosystem.

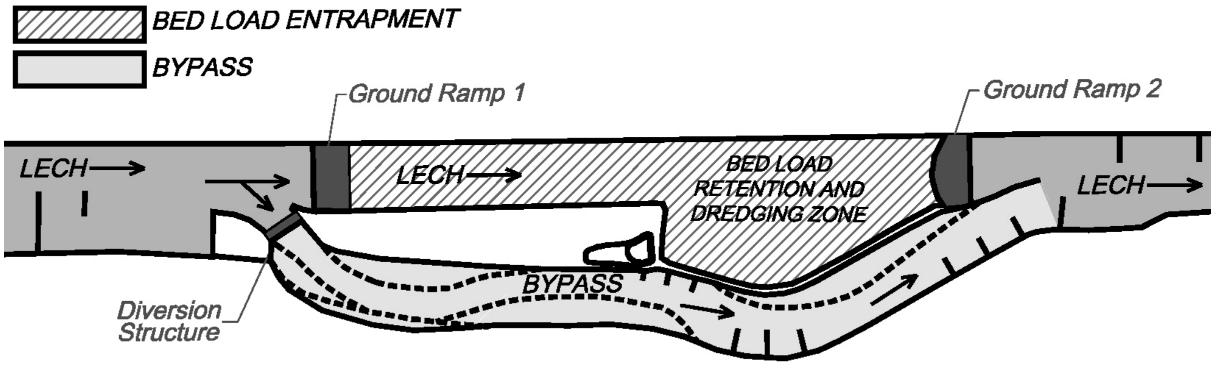

Figure Nr. 2 General map of the "Bed Load Entrapment Hornberg – Ehenbichl"

The secondary channel (=bypass) will be almost 1.000 m long. The inflow into the channel is limited through a special diversion structure at the beginning of the secondary channel. This diversion structure controls the distribution of discharge and bed load between the two channels. Discharges lower than mean flow are forwarded only through the secondary channel. Thus the dredging in the dredging zone can be done in the low water flow period, from november to march, outside the free flowing water body. Discharges higher than mean flow and the corresponding bed load get divided between both channels

The deposition zone is bounded by two ground ramps. The crest of Ramp 1 is slightly higher than the bed level of the diversion structure. Ramp 1 supports the distribution of discharge and bed load and ensures the stability of the river bottom upstream. Ground ramp 2, which is positioned at the downstream end of the widened retention zone, reduces flow velocity an bed shear stress in the retention zone to improve sediment deposition.

The sediment management within the dredging zone will be adjusted to the bed load capacity of the downstream reach, which has been assessed to an average of 50.000 m³/a. Actually the mean bed load volume at the deposition zone is about 100.000 m³/a. In response to this difference the annual bed load dredging was fixed to about 50% of the annual total bed load.

5. Physical Model

The design is based on complex flow behaviour with various hydraulic components influencing each other. During the numerical simulation the system reacted very sensitive to small changes of single parameters. The success of this project substantially depends on correct rating of the distribution of discharge and bed load at the diversion structure, the qualitative and quantitative bed load deposition in the deposition zone and the dynamic of the river channel. To investigate the synergies and interactions of all components the Department of Flood Protection at the Federal Ministry of Agriculture and Forestry, Environment and Water Management ("Lebensministerium") and the Tyrolean Regional Administration decided to set up a physical model running numerous tests at various configurations.

The physical model pursues the following main goals.

- Examination and optimisation of the theoretical solution with regard to hydraulic and morphological aspects
- Tests and analysis of possible control elements to influence the bed load distribution between the two channels with simple methods if the geomorphologic boundary conditions change

Figure Nr. 3 View of the hydraulic model

Using all of the available space in the water laboratory it was possible to build a physical model on a scale 1:80. At this scale the morphological results had to be assessed mainly qualitatively. The modelled river section had a length of approximately 1.600 m in nature and was built with a movable riverbed (fig. 3). The testing and optimisation of structures were done with steady discharges and flood waves up to the design discharge of a 100-year flood. All tests were done with continuous addition of bed load. The bed load transport was calculated from the formulas according to Meyer-Peter (1949) and Einstein (1950).

First of all the current status was investigated. This test was used to calibrate the model and to get a base of comparison for all other variants. After that the primarily planned solution was built into the model and tested in detail. Due to some shortcomings in the results of this variant it was necessary to change the structure and then the optimised variant was investigated step by step.

It took more than 20 basic variations to get the optimal solution for the described problem. The high number of variants was a result of the variety of factors influencing the hole regime of discharge and bed load. The height of the ramps, width and height of the diversion structure, flow direction to the diversion structure, bed width and course of the bypass channel, the design of the confluence of the two channels and various versions of groynes and dikes in the upstream river reach were the most important factors to identify the optimal system.

6. Proposed System

The optimised variant yields the best results in dividing the discharge and bed load between the two channels, corresponding to the aims of sediment deposition in the bed load retention zone and achieving a state of morphological dynamic equilibrium in the bypass channel and the downstream river reach. The main characteristics of the optimised solution are:

- Two groynes on the right bank upstream of the diversion structure and a distance of 1,7 m from the crest of the ramp to the bottom of the throttle (= diversion structure) guarantee the best diversion of discharge and bed load into the bypass channel and the deposition zone.
- The limitation of the width of the channel bottom by means of dikes and alternating groups of short groynes in the bypass channel provide a dynamic equilibrium in the bypass and the downstream reach.

The optimised variant was investigated by means of a detailed test programme. For that purpose representative flood events from the discharge hydrographs 1996 - 2001 were simulated (fig. 4).

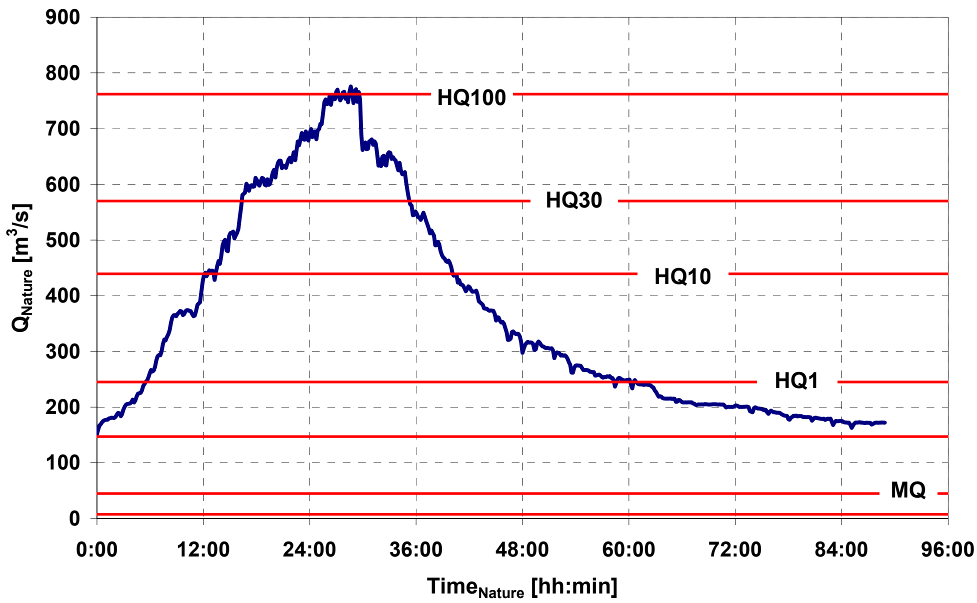

Figure 4 Simulated discharge hydrograph HQ100

Comparing bed level measurements before and after the tests it was possible to calculate balances of volumes. Using the differences of cross sections area and profile distances, the differences of volume of each test were determined. The differences of area and the added volumes were calculated for both channels and presented graphically. The diagrams show clearly the areas where the riverbed is in erosion and where sediment deposition occurs. Figure 5 shows the distribution of the areas and added volumes of the simulated HQ100 wave.

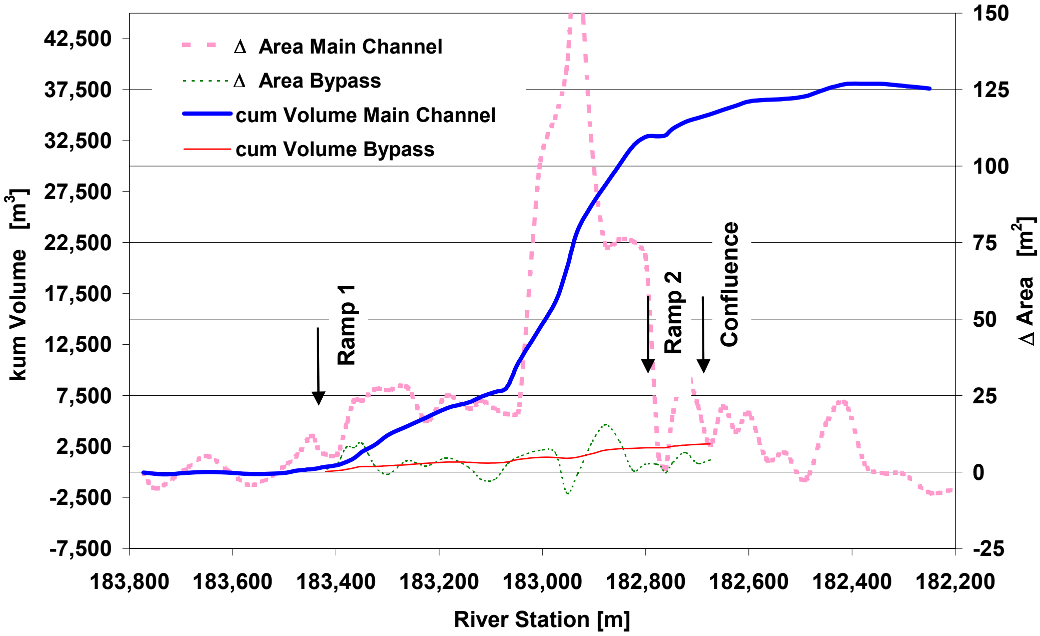

Figure Nr. 5 Added areas and volumes of the simulated HQ100 wave

The graph of the change of cross section area in the main channel shows the large sediment deposition in the widened part of the bed load entrapment whereas the upstream and downstream sections were in stable state between erosion and deposition. In the bypass channel only a minor deposition near the mouth is detectable, that will be transported to the downstream river reach at lower discharges.

All in all the investigations proved the long-time stability of the modelled river section and the bed load deposition in the planned sediment entrapment. Figure 6 presents the bed load deposition in the deposition zone by all investigated discharges.

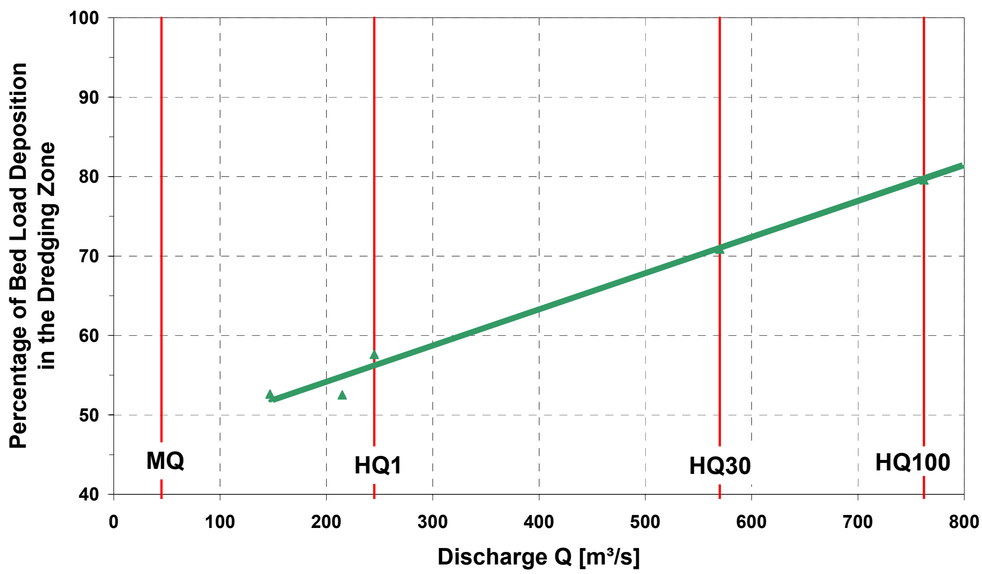

Figure 6 Relationship of bed load retention vs. discharge

The regression line shows that the sediment retention in the bed load entrapment increases with higher discharges. By simulating real hydrograph curves it was established that during an average year nearly 55 % of the bed load could be excavated and the retention increases to 80 % at extreme discharges (HQ100). If it is necessary, the downstream river reach can be supplied with bed load over the downstream ramp as well. This process was simulated in the model with fully filled bed load entrapment.

Furthermore the numerous model investigations provided valuable findings concerning the cycle of deposition and erosion of bed material in the bypass channel varying with the seasonal changes of discharge and the complex connections of the entire flow behaviour.

The model investigations did not focus only at the hydraulic requirements but also at the ecologic design of the bypass channel. During an interactive cooperation with limnologists it was possible to identify and design ecologically valuable river areas. The results of this cooperation have been integrated into the project.

The physical model “Lech - Bed Load Entrapment Hornberg-Ehenbichl” was essential for assessing the interaction of the different components and for testing the reliability of the complex site layout in the most efficient way. Although the original design, based on theory and numerical modelling, was confirmed by the investigations, the model furthermore proved its value in fine-tuning the design and evaluation of operation and maintenance guidelines. However, stochastic factors like discharge regime or saturation level of the sediment transport capacity affect bed load transport and deposition processes. Hence the permit planning is complemented by a complex river bed stability and bed load monitoring program, the results of which will be presented in the near future.

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Multidisciplinary Approach of Flood Management in a Nature 2000 Area

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Abstract: A small village – Pertisau, in the Tyrolean Alps is threatened by frequent flooding. The possibility to decrease the discharge peak by reactivation of natural floodplains by diversion of water into them and through infiltration, is evaluated by the project of the Forest Technical Service of Torrent and Avalanche Control. In order to minimize the conflicts between nature protection aspects and the technical measures, a sensitive flood protection management strategy is proposed. The reactivated flood plains in the area are used for grazing and forestry and are also a part of a Natura 2000 region. The different steps of investigations are: a) calculation of the infiltration capacity of the flood plain, b) hydraulic simulation of the retention using 2-D flow model and c) hydraulic scale model simulation to ensure the appropriate design of the water diversion structure.
Keywords: flood management, Natura 2000, flood plain, infiltration, diversion structure.

1. Introduction

The main objective of a project done by the forest technical service of torrent and avalanche control in Pertisau, Tirol, is to decrease the discharge peak by diversion of water to existing floodplains (Hübl et al, 2002). The need for flood management is evident through the frequent flooding of the village Pertisau. The investigated area is geomorphologic an alpine catchment (Figure 1). The three main rivers are characterised by a periodic discharge.

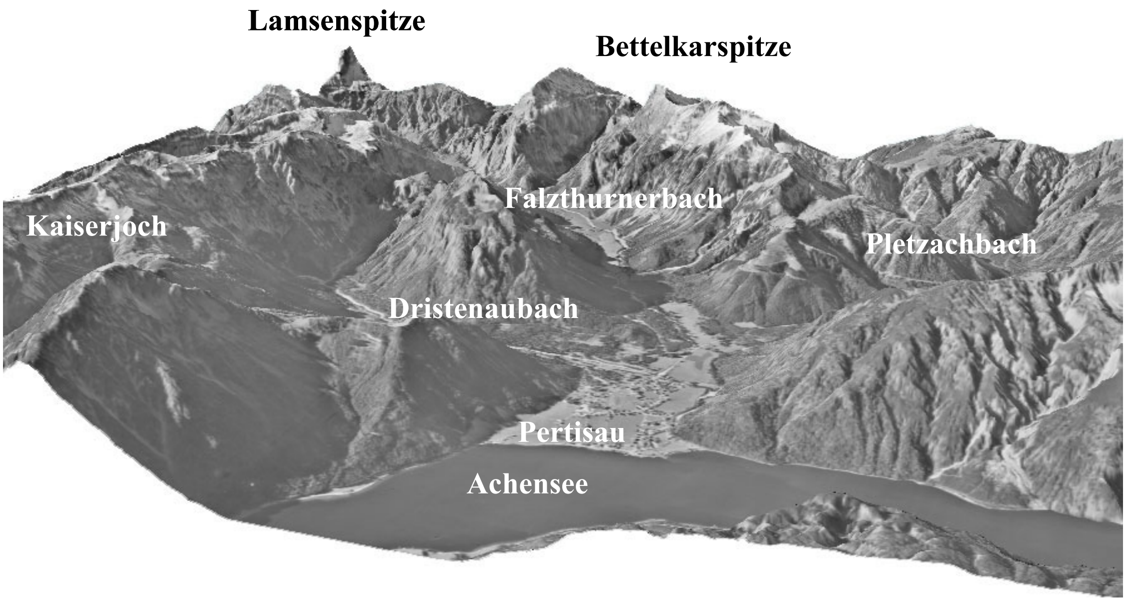


Figure 1 View on the project area, Tyrolean Alps, Achensee

The hydrological data are supplied by a rainfall-runoff model (IHRINGER et al., 1988). The results are the requirements for the flood management concept. The validation of the hydrological parameter is performed by the precipitation events from 1992 (PLONER & SÖNSER 1997; zit. in: FTD f. WLV, GBL. WESTL. UNTERINNTAL 2001). A comparison of hydrographs before and after proposed measures are summarized in the Table 1. The discharge load is limited by the capacity of the open channel at observation point 47 (FTD f. WLV, GBL. WESTL. UNTERINNTAL 2001: 19). The discharge load, which needs to be diverted is the difference between design discharge and evaluated design event. The last column in the Table 1 represents the volume, which needs to be diverted and retained.

Table 1 Comparison regarding loads and peaks of runoff by the event from 1992 and of present design discharge before and after flood management measures at important observation points. Q_{max}: peak discharge, QF: discharge load; [m^3]

No.	Grid point	event 1992*)		design discharge		evaluated design event	
		Q_{max}	QF	Q_{max}	QF	Q_{max}	QF
		[m^3/s]	[10^3m^3]	[m^3/s]	[10^3m^3]	[m^3/s]	[10^3m^3]
43	Gerinne Tristenau	20,0	199,1	22,8	303,0	5,0	82,0
44	Mdg Pletzach	14,6	201,0	14,6	269,3	5,0	84,6
46	Pletzach 4	32,7	674,2	34,2	808,7	18,3	334,4
47	Gerinne Pertisau	32,8	534,3	34,4	826,3	19,6	351,4
48	Mdg Achensee	18,7*	537,7	18,7*	632,1	17,1	355,9

The specific objective of the study conducted by the University of Natural Resources and Applied Life Sciences, Vienna is the evaluation of this protection concept. By laboratory experiments, the design of the diversion structure to conduct water in the flood hutch channel has been optimized. A major factor for the success of the project concept is the infiltration capacity of the flood plains. In order to derive the data for describing the infiltration capacity, field studies and laboratory experiments were carried out and a monitoring system was installed to characterize the soil water balance. A numerical model is utilized to simulate the infiltration processes. Finally, flooding is simulated by a hydraulic model, while taking into account different scenarios. The combination of the results achieved by the above described methods, should contribute to a proper project evaluation.

2. Methodology

To ensure a holistic view on the flood management concept, different disciplines are involved, that are specifically related to certain tasks within the project (Figure 2). The coordination is in the hands of the Institute of Mountain Risk Engineering.

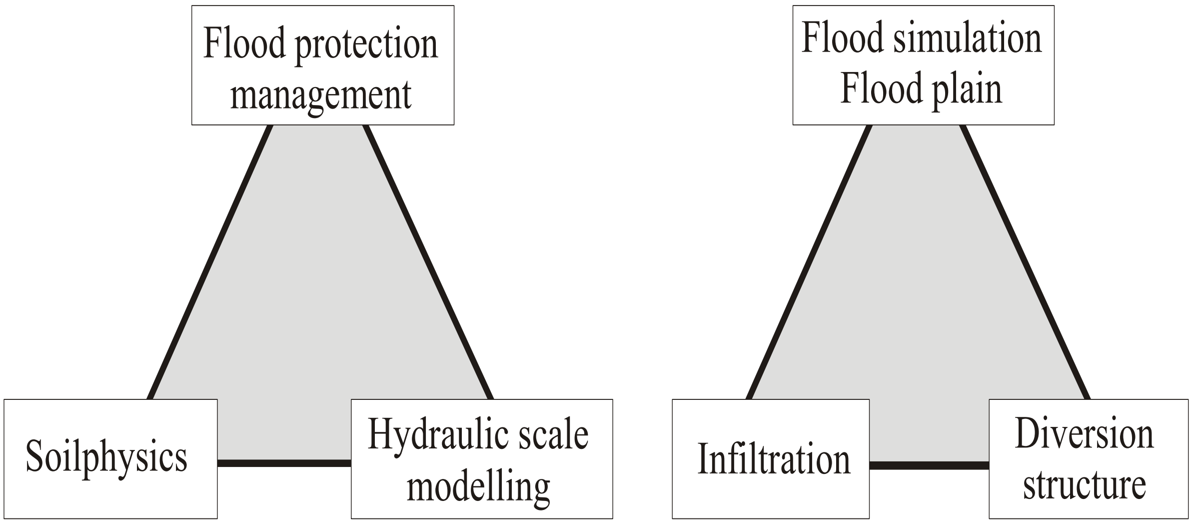


Figure 2 Disciplines and expertises involved

2.1 Hydraulic scale model

As described before, due to the resulting restrictions of discharge loads, especially by the high water events, one part of the total discharge has to be diverted into the natural flood plain. The diversion system consists of two main units, the diversion structure itself and the diversion flood hutch (diversion channel) (Figure 3). According to the specific hydraulic conditions (flow velocity pattern and water levels) by various discharges through the retention basin, the hydraulic scale model experiments aim to achieve an optimal design of the diversion structure, diversion channel and their interaction. In order to obtain the desired ratio between the total discharge, diverted discharge and the remaining discharge in the mountain torrent, four series of experiments are undertaken. Additionally, the chances of arising the hydraulic conditions for an extreme flood event and possible scenarios, e.g. failure of the diversion channel dam, clogging of the diversion structure, are analysed.

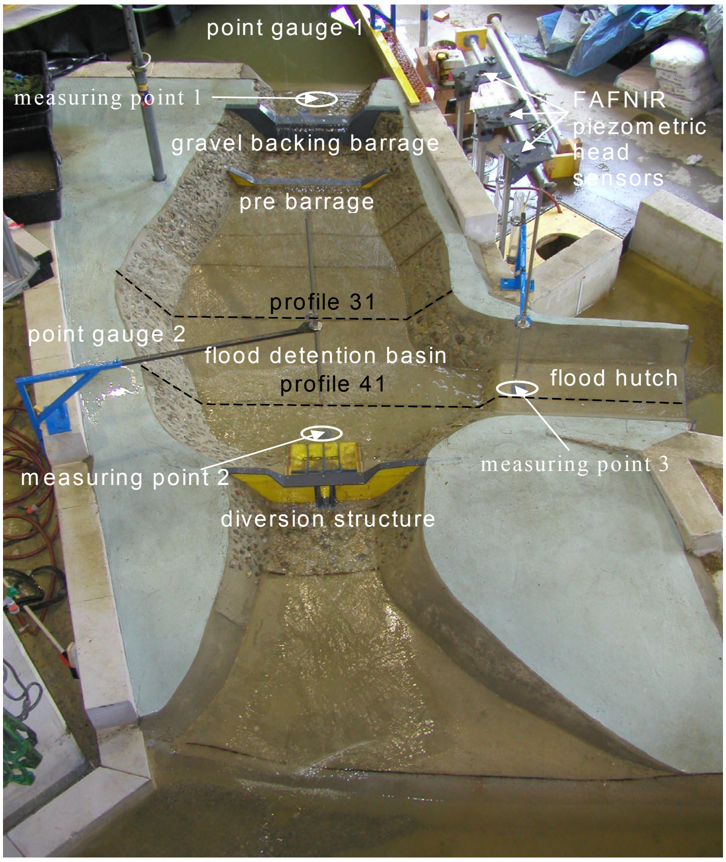
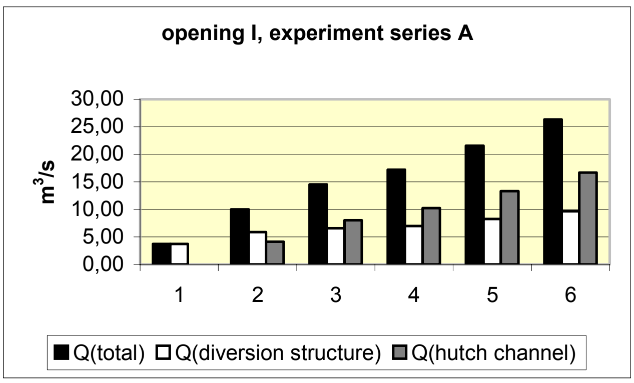


Figure 3 Hydraulic scale model, diversion structure and diversion channel

The diversion structure is constructed respectively the design conditions for an extreme flood event. Diversion initiates by a total discharge of 3,74 m^3/s and by 6,03 m^3/s, the flood plain is starts to be inundated (Table 2). This complies with expected flood peak reduction for the given threshold of 10 m^3/s, as the remaining discharge in the river. To guarantee a proper operation of the diversion structure, logging and sedimentation should be possibly prevented.

Table 2 Diversion ratio river diversion channel and total discharge

opening I, experiment series A		
Q_{total}	$Q_{diversion\ structure}$	$Q_{hutch\ channel}$
[m^3/s]	[m^3/s]	[m^3/s]
3,74	3,74	0,00
9,98	5,86	4,13
14,56	6,55	8,01
17,20	6,95	10,25
21,56	8,24	13,32
26,31	9,65	16,66



2.2 Estimation of the infiltration rate of the flood plain

A key factor for the water diversion is the infiltration capacity of the floodplains. Different scenarios to estimate infiltration processes for typical soil conditions are simulated with the software package Hydrus 2D. The necessary data for the simulation are provided by field studies and laboratory experiments (Loiskandl et al., 2003). Also, a soil water status monitoring system was installed to characterize the soil water dynamics and soil water balance. The combination of these approaches provides a proper evaluation of infiltration capacity and supplies the required input data for the hydrologycal simulation. Soil profiles are selected to account for the heterogeneity of the investigated area, hence from the soil physical investigation four profiles were selected (table 3). Top layers show a smaller hydraulic conductivity and are therefore dominating the infiltration process. Below the simulated profile soil contains a high percentage of gravel and functions as a drainage layer.

Table 3 Soil profiles for simulation

Profile	Layers [cm]	Soil type	k_s-value [cm/s]
A1-1M	0-10 ID	sU	$5{,}0 \times 10^{-5}$
	10-30 TI4	U	$1{,}5 \times 10^{-5}$
	30-40 Rosetta [1)]	sT	$4{,}4 \times 10^{-4}$
A1-1	0-20 ID	sU	$2{,}0 \times 10^{-4}$
	20-40 Rosetta	tS	$4{,}1 \times 10^{-1}$
	40-60 Rosetta	S	$2{,}9 \times 10^{-2}$
A2-1M	0-20 IA	sU	$3{,}6 \times 10^{-3}$
	20-50 Rosetta	tS	$4{,}1 \times 10^{-3}$
A2-1Mb	0-20 IA	sU	$3{,}6 \times 10^{-3}$
	20-50 Rosetta	tS	$1{,}0 \times 10^{-1}$

1) hydraulic conductivity values are derived from soil texture, Rosetta 1999

For each soil profile, two simulations with changed initial conditions are performed. Initial water tensions are 0,02 and 0,08 bar respectively. Profiles A1-1M and A2-1M are in accordance with the soil physical situation at the flood plains at the respective measuring sites. The simulation time was 1 hour, equal to the flooded period.

The boundary conditions for the simulation are:

- Top: constant head, represents 10 cm water level during flooded period
- Bottom: free drainage, deep layers contain high amounts of gravel
- Sides: impermeable to ensure vertical flow only

Water retention curves and hydraulic conductivity functions are derived from laboratory and field test using the relations established by van Genuchten.

For stability reasons, the profiles A2 were simulated for the dominant layers only. The summary of obtained infiltration rates is given in Table 4. Cumulated infiltration varies between 0,2 cm/h and 31 cm/h. This is in a significant agreement with the hydraulic conductivities of the top layers, which are limiting the infiltration rate. Different simulated scenarios represent the soil heterogeneity of the flood plain. Low infiltration rate is related to visible water logging areas. The other extremes are the areas with high infiltration capacity.

Table 4 Simulation of infiltration

Profile	Cumulated Infiltration for 1 h (cm)	Cumulated Infiltration for 1 h (cm)
A1-1M	0,2	0,7
A1-1	1,67	2,04
A2-1M	16,2	17,1
A2-1Mb	30,5	31,05

Dew to the limiting condition and the relatively high initial water content in the top layers, the infiltration rate reaches very soon a constant value (Figure 5).

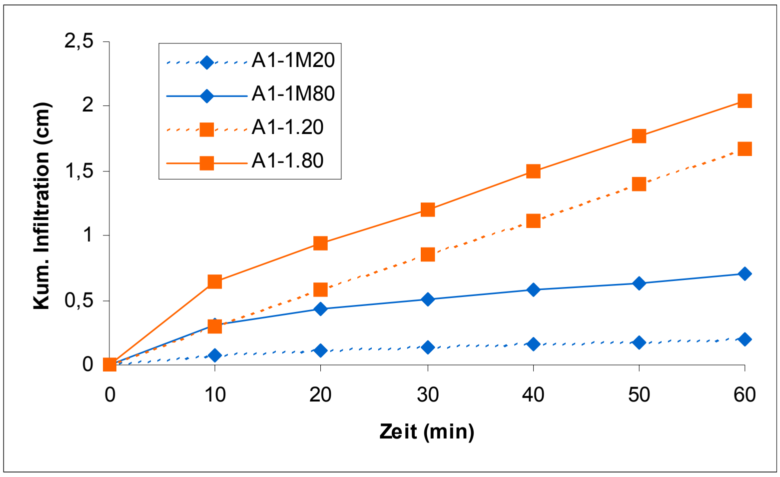


Figure 5 Infiltration at the Profiles A 1-1M and A 1-1; Simulation time: 1 hour

If it is assumed, that all the profiles are equally distributed over the whole domain, then for D-A1 and D-A2 averaged infiltration rates are 1 cm/h and 25 cm/h respectively. To make this more relevant, a weighing according to the land pattern needs to be performed. Water logging areas are not comprising 25 % of the total area. On the other hand, for the hydraulic calculation a single representative parameter is required. A first attempt is to relate the infiltration rates to the size of the two flood plains. Flood plain D-A1 has an area of 0,076 km² and D-A2 of 0,287 km. According to this information, an infiltration rate of 20 cm/h for the total inundated area is reasonable.

2.3 Hydraulic/Hydrological simulation of flood

The hydraulic/hydrological simulation aims to quantify the effects of flood management measures. The catchment Dristenau was selected as a test region. So, conclusions should be transferable to all the other sub catchments. Two processes are influencing the reduction of the flood wave, retention and infiltration. The surface of the flood plain was reconstructed digitally, by a geo-statistical method. Flow-2D was utilized to simulate the flow pattern of the flood plain. Figure 6 is an example of flow simulation for scenario 1 and an infiltration rate of 150 mm/h.

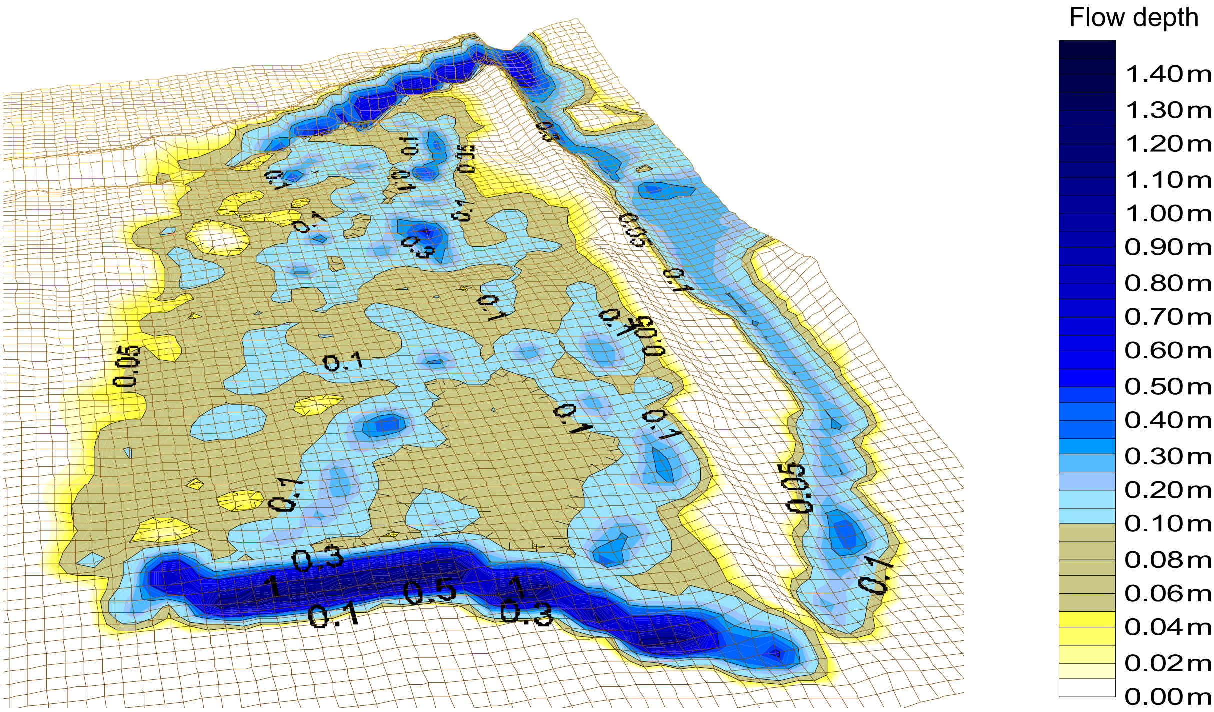


Figure 6 Flow depth contour lines, land use scenario 1, Grid size 20 m, infiltration rate 150 mm/h

From the simulations it became obvious that the infiltration has the most prominent influence on the results. Other factors like grid size and land surface pattern showed relatively small influences on the simulation results. The simulation was performed with an input hydrograph only, no additional precipitation was considered. Based on the formerly obtained results, the infiltration rates are varied in a realistic range to the effect on the flood peak reduction. The infiltration was varied from 0 to 200 mm/h, the results are summarized in Table 5 and Figure 7.

Table 5 Simulation of flood retention

Scenario	Inflow (m^3)	Outflow	retention (m^3)	Infiltration (m^3)	total retention (m^3)	Peak dis. (m^3/s)
grid20 inf 0	126454	118981	7474	0	7474	13,50
grid20 inf 10	126453	113664	5691	7098	12790	12,99
grid20 inf 20	126451	108715	4900	12836	17736	12,69
grid20 inf 50	126454	96143	4052	26260	30311	11,38
20 inf 80	126454	86728	3847	35879	39726	10,03
grid20 inf 100	126455	82755	3779	39920	43699	9,44
grid20 inf 120	126450	80070	3731	42649	46380	8,79
grid20 inf 150	126455	76879	3677	45900	49577	7,82
grid20 inf 180	126454	74341	3631	48483	52114	7,02
grid20 inf 200	126454	73077	3612	49767	53379	6,50

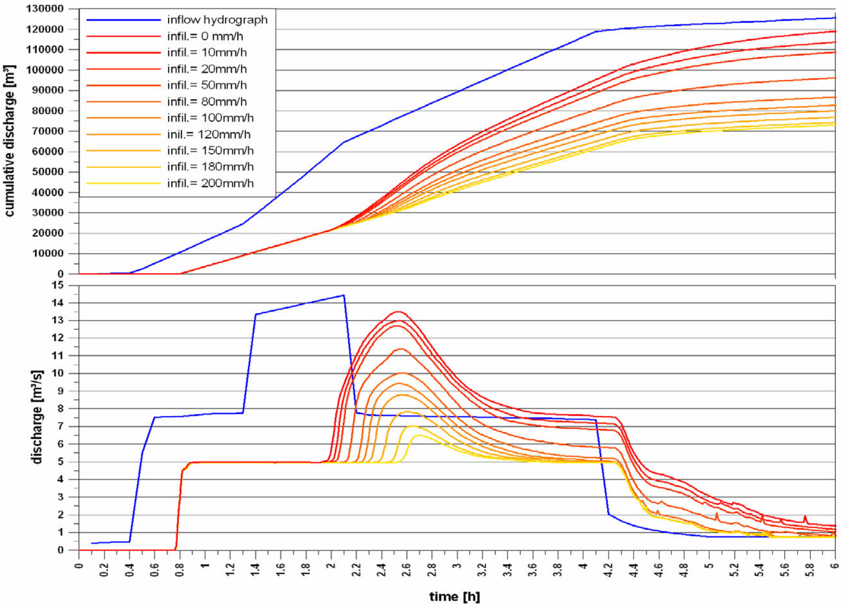


Figure 7 Outflow hydrographs for different infiltration rates (150 years return period)

4. Summary

The multidisciplinary approach proved to be quite successful. The requirement for flood protection and environmental concerns could be matched to a high level. The results obtained at the test sites are very valuable for the ongoing flood management in the region. The hydraulic scale model investigation resulted in the optimisation of the diversion structure and the adjacent diversion channel. Insight into sedimentation and logging by wood could be obtained. Simulation of the infiltration capacity, based on soil physical investigations, could provide a good insight into the system performance of the flood plains. The complex soil system could be characterised for the hydraulic simulation and a realistic input parameter could be provided. Finally, the hydraulic simulation could show, how the measures are effecting flood management. Retention and infiltration, together resulted with the required reduction of the flood peak. It is obvious, that without infiltration study (retention only), the project goal could not have been achieved.

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Aggregation of Reservoirs in Alpine Regions

Cedomil Josip Jugovic, Georg Schuster, Hans-Peter Nachtnebel

Abstract: All the natural streams transport certain amounts of non-cohesive soil particles. Thus, the water discharge and the sediment load of a river change with time, since both natural and man-made factors vary the water and sediment supply within a river basin. Due to the human interaction in the water body by installation of hydraulic structures, the flow conditions alter. These variations cause changes of river phase which also result in flow reach aggradation and degradation. Based on a simplified scale model, some basic principles in formation of sediment deposits at the mouth of the reservoir in a schematised river basin are being discussed.
Keywords: scale model, sediment transport, delta

1. Introduction

Dams are designed to a finite life, as it is typically done by engineering structures. Life expectancy of reservoirs is usually limited by accumulation of sediment, reducing useful storage. The solid phase transported within a stream, begins to deposit at the flow profile where the backwater influence starts. When a stream enters an impounded reach, flow velocity decreases. Following, the sediment transport capacity reduces, deposition starts in the reservoir headwater area. This process forms a delta in this region, that extends further into the reservoir (Fig. 1). With time, the delta can essentially fill the reservoir with deposited sediments. Thus, the benefit for which the dam has been built is eliminated.

Yet, once a project has reached the end of its useful life, it is not possible just to replace the dam. Dams can not easily be re-built at other locations: attractive dam sites, from the technical point of view, are scarce. Nevertheless, the sediment issue will still have to be addressed. The solids that have already been deposited may also degrade, move and even be replaced. For such a process, on contrary, some decrease of the flow depth is needed. An increase of the flow velocity and thus also of the sediment transport capacity follows.

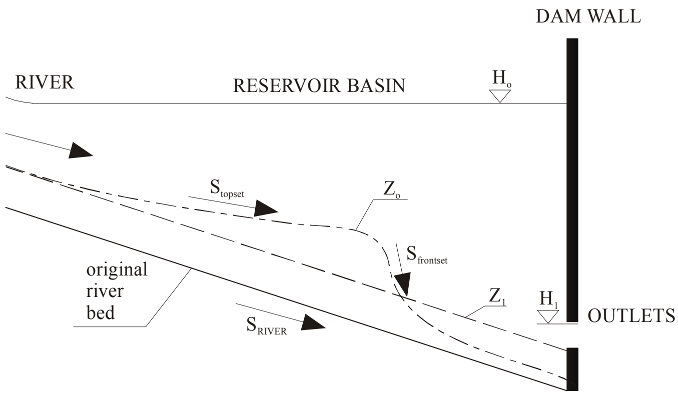


Fig. 1 Typical sediment deposits in a reservoir

Insight in the reservoir sedimentation processes and practical methods of reducing sedimentation are thus vital for designing new and managing existing dams and reservoirs.

2. Material and methods

2.1 Stream-bed aggradation and delta development

Temporal and spatial scales of reservoir deposition are usually so large that the field observation of the deposition process is difficult. Therefore, laboratory investigations on a hydraulic model were carried out, to provide an insight into the processes of sediment deposition and degradation in a reservoir.

When a river enters a reservoir, sediments begin to deposit due to changed flow conditions. The largest sediment particles settle at further positions upstream and are followed by progressively smaller particles downstream in the reservoir. The bed of an aggrading channel rises continuously. From the continuity equation for bed-material transport in an open channel, it follows that

$$\frac{\partial z}{\partial t} + \frac{1}{B(1-\lambda)} \frac{\partial Q_s}{\partial x} = 0$$

Aggradation means that $\frac{\partial z}{\partial t} \geq 0$, hence, it is necessary that $\frac{1}{B(1-\lambda)} \frac{\partial Q_s}{\partial x} \leq 0$. Since $\frac{1}{B(1-\lambda)}$ is always a positive value, $\frac{\partial Q_s}{\partial x}$ must be negative. Thus, the bed-material load decreases in the downstream direction, or, in other words, a certain amount of the bed-material load is deposited to build up the bed elevation during channel aggradation. Aggradation generally involves a decrease of channel bed slope and water-surface slope in that direction, or $\frac{\partial S_0}{\partial x} \leq 0$ and $\frac{\partial S_w}{\partial x} \leq 0$, where B is width of the channel, Q_s is sediment discharge, Q_w is water discharge, S_0 is channel-bed slope, S_w is water-surface slope, Z is bed deviation from the reference datum and λ is porosity (Chang, H., 1982). In the initial stage of the delta development, the deposition of sediment occurs, due to a sudden enlargement of the channel cross-section, and thus, the reduction of the sediment carrying capacity. As the delta stream carries sediments and deposits them at the mouth of the stream, the stream elongates. If the base-level of the reservoir, the water discharge Q_w, the sediment discharge Q_s, and the grain size d, remain unchanged, elongation of the stream diminishes the stream slope. The stream tends to maintain its original slope and thus arises the stream-bed aggradation. The same process may also be initiated by overloading the water body with sediment, by a decrease in water discharge, or by a rise in the base level. The delta gradually moves downstream and eventually approaches the dam. Further, inflow of sediment may even pass over the dam.

A graded stream is a one, which has, over a periode of time, reached its equilibrium. This means, that its slope assures the velocity nedeed to transport the available sediment (bed load and suspended sediment). In other words, a channel is in equilibrium, when the energy avaiable due to discharge and slope is just sufficient to carry the charge, without any tendency for the channel to change its shape or slope. If one parameter is changed, the whole system changes, until it ist again in equilibrium. The equilibrium of a channel is a perfect condition, which the stream always tries to develop. Due to seasonal fluctuations of discharge, sediment and the water surface elevation in the nature, the equilibrium is never fully achieved.

3. Set up of the model - delta development

At the BOKU – University of Natural Resources and Applied Life Sciences, Vienna, the experimental investigation of the reservoir sedimentation has been undertaken. The aim was to document the water-sediment transport processes and a delta forming at the mouth of the reservoir. One simple hydraulic model, consisting of a rectangular reservoir and a straight incoming channel with quadrate cross section was constructed. An “infinite” reservoir water volume has been simulated by an overflow along all three sides of reservoir. Thus, the influence of the concentrated reservoir outflow on the delta formations could be avoided. The steady dosing of the uniformly narrow-graded sediment (quartz sand) took place at the beginning of the inflow channel. Namely, such a range of sediment grading allows us to observe the ongoing aggradation processes (which for alone are quiet complex), without taking into account the gravel sorting (Jugovic, C. J., 2003). Under these circumstances, only one critical velocity v_{cr} exists, which is needed to transport the sediment. In the nature, by the transported coarse material, out of different grain sizes, one prevails, which is representative for a certain flow reach. At the same time, a range of other grain sizes exists, starting from suspended load until the gravel and cobbles. Thus, some sediment transport always takes place.

Depending on the flow velocity and grain size, a part of sediment will be transported. If the transport rate of particular size is known, by the superposition, it is possible to conclude about the total sediment transport, whereas the hiding effect and the development of the bed armouring layer should be considered.

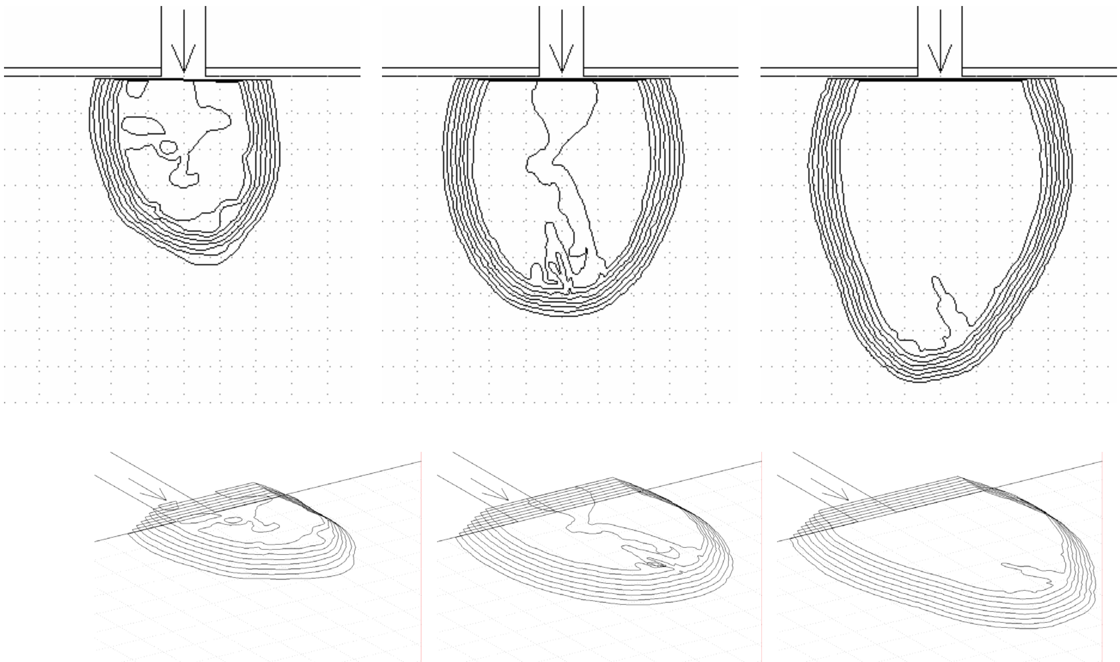
The start of the experiment was always without the sediment in the 5m long inflow channel. In the first phase of the experiment, the sedimentation in the inflow channel occurs, until the conditions for its transport (velocity, slope) are obtained. Then, the progressive movement of the sediment, in the flow direction towards the reservoir, starts.

Photograph 1 Beginning of the sedimentation in the channel (flow direction from the right to the left)

Photograph 2 Movement of sediment through the inflow channel (flow direction from the right to the left)

The self formed flow depth in the incoming channel depends on the discharge (flow velocity) and the sediment grain size. This means, that by the same sediment, after some time, the same flow velocity occurs. This velocity corresponds to the v_{crit}, the velocity needed to transport the coarse material. Slope formation depends on the sediment concentration and the length of the flow section. The flow section is those length, that the sediment has to pass, until it deposits in the reservoir. This length rises continuously during the test performance, because it lasts until the total sediment is deposited.

Once the deposition reaches the reservoir, the delta begins to form. The method of the Moirè topography made it possible, to record spatially the development of this deposition, simultaneously, during the tests performance. This optical method of 3-D acquisition of the surfaces has already been applied by Müller (2001). The experimental basin is being covered by a grid and illuminated in such a way, that the shadow pattern occurs. Under specific assumptions, the Moirè pattern presents the isolines. Those are recorded, digitised and distortion is being annulated. Due to the fact that we are using an optical, non-invasive method, the experiment is not being disturbed. The result is exact 3-D delta form that has been developed under the water surface.



Figures 1 Delta development – 3-D digitized delta form

The development of the delta formation occurs in several phases and may be described by a dimensionless form factor (Jugovic, C. J., Saenyi, W. W., 2002). The form factor is the ratio between the maximal delta length and the maximal delta width. In the first phase, depending on the water discharge, an intensive growth of the delta in the longitudinal direction may be observed. This growth results with the maximal form factor. After that, a distinguished growth of the delta width continues, until also here, depending on the water discharge, a minimal form factor is achieved. These phases alter permanently. After some time of performance, they can not be distinguished any more. The development of the form factor is similar to a damped sinusoidal oscillation. So, the form factor depends also on the discharge and the deposited sediment quantity.

$$L/B(t) = f\left(Q_w, \int_{t=0}^{t} Q_S \cdot dt\right)$$

where Q_w is the water discharge and the Q_S is the sediment transport in [g/s].

In the following diagram (Fig. 2), the dependency of the form factor L/B on the discharge may be observed. It may be noticed, that the form factor L/B depends at every time point on the total mass quantity $\int Q_S \cdot dt$ of the deposited material, until the moment of observation. The time axis for the form factor development is being transformed in relation to the total sum of the until given moment deposited material. The experiments with different sediment dotation Q_S, by same water discharge, result with the same form factor characteristics L/B. A higher dotation of the sediment into the water flow, results just with an, with dotation proportionally faster (temporally dependent) process. The discharge influences directly the delta form.

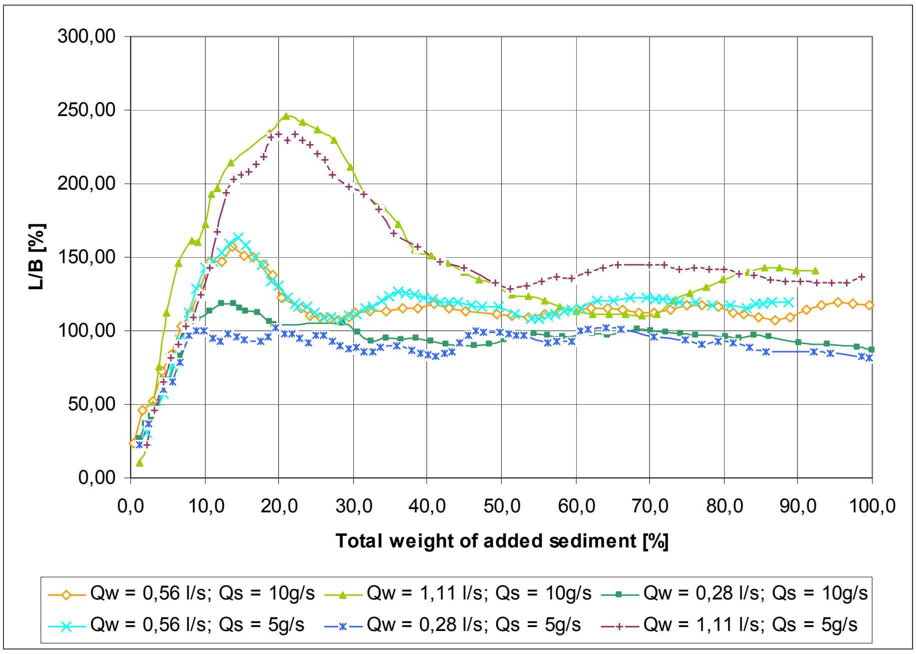


Figure 2 Form factor L/B as a function of water discharge Q_W and sediment mass concentration Q_S

The next interesting relation occurs between the slope of the inflow channel and the delta form. The slope is, as former mentioned, primarily depending on the sediment concentration and rises with the duration of the experiment. At the same time, the flow distance prolongs (along with the flow resistance). In the following diagram, the relation between the slope and the sediment concentration may be seen.

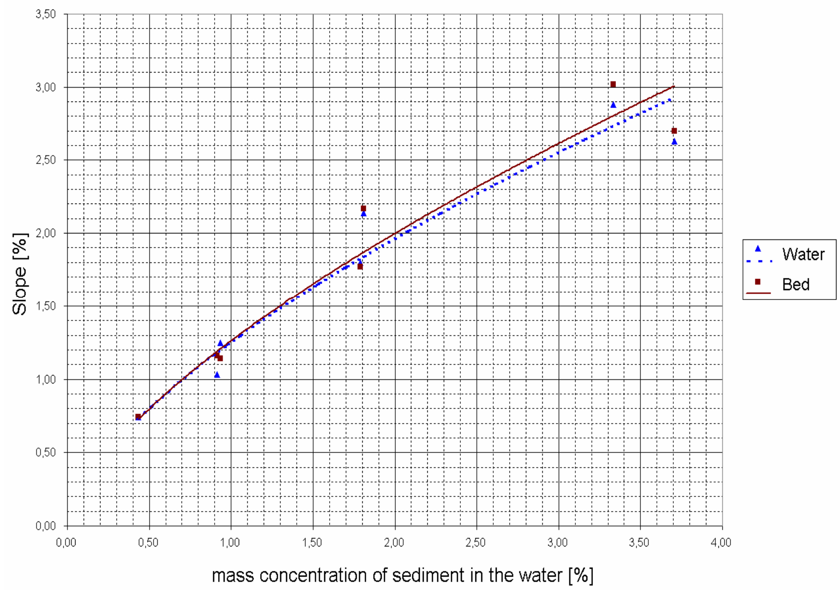


Figure 3 Incoming channel slope as a function of the sediment mass concentration

Consequently, a stronger correlation may be observed in the Fig. 4., where the relationship between the incoming channel slope and the ratio between the sediment mass concentration of and the form factor are presented.

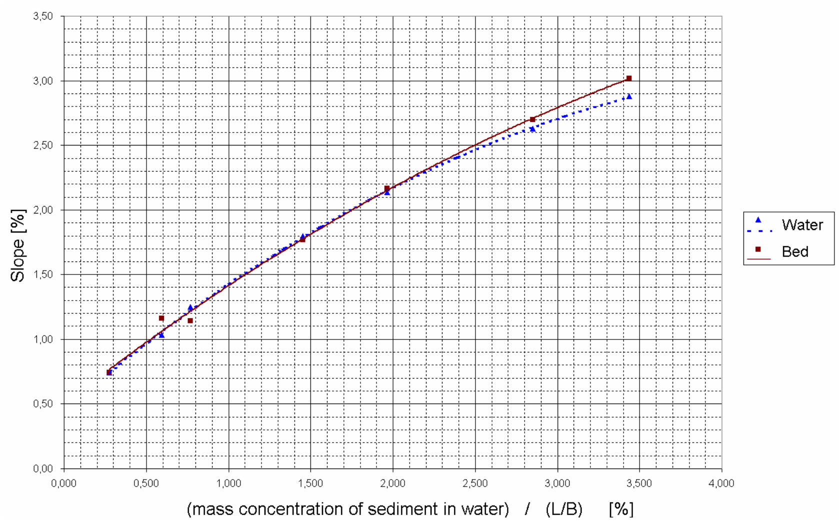


Figure 4 Incoming channel slope as a function of the ratio between the sediment mass concentration and delta form factor

The delta form influences the flow resistance. Flow resistance and sediment mass concentration are the determining factors for the channel slope.

4. Summary

Silting of the reservoirs, as a consequence of building dams in the rivers, leads mostly to irreversible changes in riverine systems (hydromophological alterations). That is why this processes have to be thoroughly investigated and, where possible, mitigated.

This experimental study contributes to a better understanding of the delta formation processes, by silting in the reservoirs. It has been observed, that the delta form depends primarily on the discharge in the incoming channel. The slope in the inflowing channel is effected by the sediment concentration, but also some influence of the delta form has been observed. The delta size influences directly the channel slope: the slope rises along with the growth of the delta in the longitudinal direction. This may be explained by the increased flow resistance.

These experimental results are also a valuable input information for the numerical model studies.

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Some Aspects of Physical and Numerical Modeling of Water Hammer in Pipelines

Apoloniusz Kodura, Katarzyna Weinerowska

Abstract: Numerical modelling of water hammer phenomenon in pipelines is still an open problem. Although the matter has been widely analysed by many authors, the proper mathematical description of all aspects of this phenomenon is still not recognized. It is known, that the pressure characteristics obtained by computations are usually significantly different from the results of experiments. However, experiences show that thanks to some special mathematical and numerical treatment one can obtain results of sufficiently good correspondence to observations. Unfortunately this kind of approach proved to be effective if the single pipeline of constant diameter is analysed. For more complicated cases the proper representation of the phenomenon is much more difficult, as many different factors influence and distinctly modify water hammer phenomenon run. In the paper the results of chosen experiments on water hammer phenomenon (for simple positive water hammer run in pressure pipeline of different diameters, water hammer run in pressure pipeline with the local leak) are presented. The results of experiments and numerical analysis of the phenomenon are presented. The possibility and efficiency of numerical simulation of the water hammer phenomenon are discussed and the conformity between calculated and observed (measured) pressure characteristics are analysed.
Keywords: water hammer, diameter change, local leak, measurements, numerical errors

1. Introduction

Rapid changes of flow velocity in pipelines, caused by sadden valve operating (opening or closure), abrupt changes in work of the pumps, pump failures, mechanical vibrations of some elements and other reasons, result in violent change of the pressure value, which is then routed in the pipeline in a form of a rapid pressure wave. The celerity of such disturbance may exceed 1000 m/s, and the values of pressure oscillate from very high to very low, often causing noises and serious damages in pipelines, different forms of cavitation and other negative consequences. The phenomenon is widely known as water hammer and has been the subject of various analyses since the end of the nineteen century. Although those first analyses were of the preliminary character, the basic theoretical problems were discussed, the formula for the wave celerity was derived and the description of wave transformation and the influence of pipeline junctions and fixtures were taken into consideration. Since then many other works were presented (e.g. Parmakian 1955, Streeter and Wylie 1967 and others), and a vivid advance in the phenomenon description was observed. However, in spite of a big progress in mathematical modeling and measurements, water hammer is still one of the most interesting problems of pipeline hydraulics, and the subjects of numerous publications (e.g. Brunone et al. 2000, Covas et al. 2004 and others), as one of those problems which still is not recognized in sufficient way. The possibility of measuring and recording the observed values of pressure enabled to compare the measurements with the results of calculations, which proved to be significantly different. Since then the attempts to improve the classical description of water hammer and to recognize the main factors influencing the phenomenon run have been carried out. However, the problem is still the open question.

The phenomenon of water hammer may be considered in two ways. From the practical point of view, the main problem is to know the extreme values of the pressure, which are usually considered as the peak of the first amplitude of the pressure wave in pipeline. As it is known, the pressure wave is attenuated in a pipeline due to many reasons, from which one of the most important is flow resistance. The duration of the phenomenon depends on the material of the pipe, initial value of the flow, kind of the medium in pipes and other factors but each time it is usually a question of a few seconds. Thus, for some researchers the basic and most important task is to recognize the highest values of pressure, as the oscillation period, attenuation intensity and other characteristics are of less importance. As the maximal value of pressure wave during simple water hammer run may be calculated from the theoretical formula, which usually leads to the values close to the observed ones, problem of water hammer is sometimes considered to be solved. However, the situation is relatively easy if the simple case of a single pipe of constant diameter is considered. However, in most practical cases the situation is usually more complicated and additional factors influencing water hammer run occur. If the pipeline network is considered, not only the maximal value of pressure can not be easily calculated from the simple formulas, but also the place of its occurrence is not easy to recognize with simple theoretical approach. The pressure wave may be routed in complicated network of pipes, waves may superimpose or reflect from obstacles and the prediction of the extreme values of the pressure is not easy. Thus, no matter if the problem is considered from practical point of view only or from theoretical point of view as well, there are still some aspects that are not recognized in sufficient degree.

From the theoretical point of view the coincidence of the first amplitude of pressure wave is not enough for proper description of the phenomenon. To achieve this, the consistency of the calculated and measured values of pressure should be obtained for the whole duration of the water hammer run. From scientific point of view it is important to model the phenomenon in the proper way, to obtain the result close to observations and to recognize the factors modifying the simple water hammer in pipeline. It is not only interesting from cognitive point of view, but also important for more detailed analyses, in which the observations and prediction of pressure characteristics may be applied to solve practical problems of different nature, e.g. localization of leaks in pipeline. However, in most cases it is very difficult or impossible to obtain good coincidence between numerical solution and measurements, and the calculation results are often significantly different from observations, not only as to the values of pressure of next amplitudes of the wave, but for the oscillation period and the time of the phenomenon duration as well.

2. Main factors influencing water hammer modeling

The problems of the inconsistence between calculated and observed pressure characteristics are of various nature. The main aspect is connected with mathematical description of the phenomenon. As the significant difference between calculations and measurements is observed for most cases of water hammer – especially as to the intensity of wave attenuation, wave smoothing and the duration of the phenomenon, many researchers focused on the improvement of the form of the governing equations. The traditional description (Chaudhry 1979, Streeter and Wylie 1967, and others) of the phenomenon run:

$$\frac{\partial H}{\partial x} + \frac{1}{gA}\frac{\partial Q}{\partial t} + R_o Q|Q| = 0 \text{, where } R_o = \frac{8}{g\pi^2}\frac{\lambda}{{D_w}^5} \tag{1a}$$

$$\frac{\partial H}{\partial t} + \frac{c^2}{gA}\frac{\partial Q}{\partial x} = 0 \tag{1b}$$

(where Q is a rate of discharge, H – water head, g – acceleration due to gravity, a – wave celerity, A – cross-section area, D_i – internal pipe diameter and λ is the linear friction factor), was often replaced by more complicated description, in which the main emphasis was put on the modification of friction term in momentum equation. One can find many different approaches in which the friction factor is increased in relation to steady friction term on different ways, from the simplest idea of multiplying it by some constant (even up to 10 and more), to more complicated ones, based on introducing to the friction formula additional terms dependent from space and/or time velocity derivatives.

The difference in calculations and observations is particularly vivid for pipes made of polymers. The reason of this fact is the viscoelastic behavior of this material as the reaction on the stress. The equations (1a,b) describing elastic model may be applied for steel pipes and for preliminary calculations for plastic pipes. If more accurate calculations are needed, it is necessary to develop the form of mathematical description with taking into account viscoelastic character of pipe walls deformations (Brunone et al 2000, Covas et al. 2004).

Examples presented in literature may suggest the problem of water hammer can be effectively solved by applying the approaches mentioned above. However, as the experiences of Mitosek and Szymkiewicz (2005) prove, this way can not lead to successful solution, from formal point of view. As long as the mathematical description (1a,b) is improved by modification of the friction term only, the results of calculations will not be consistent with the observations. The problem is connected with the nature of the phenomenon and the type of the equations. The observations, especially the shape of the pressure waves, prove that the phenomenon run is influenced by some additional mechanism of diffusive character, which should be represented in mathematical description. As long as the set of equations (1a,b) or its modified version is of hyperbolic type, the results of calculations will differ from the observations. Thus, the modification of the friction term may influence the values of pressure during the phenomenon duration but it will not lead to the characteristic smoothing of the wave shape observed in measurements. This can be done only by a term of diffusive type.

Many authors present the results of the calculations obtained for the models with modified friction term, which seem to be very close to the observations. This is often connected with another question, which is numerical aspect of mathematical modeling. Unfortunately, some features of the pressure characteristics obtained from calculations are the result of pure numerical effects. This may lead to the wrong interpretation of the quality of calculations and mathematical description. The very good coincidence between calculations and observations is very often the result of numerical errors introduced to equations with the numerical scheme applied in solution. The most vivid example is numerical diffusion, which is nearly always introduced to the scheme and which leads to smoothing of the solution. This smoothing is of unphysical character and the coincidence of such solution with observations should not be interpreted as properly solved water hammer problem.

The problem with proper mathematical description of the phenomenon and numerical effects influencing the solution are not the only aspects making the solving of water hammer problem difficult. The remarks presented above were connected with single pipeline of constant diameter, but the situation becomes even more difficult if more complicated systems of pipes are taken into consideration. In many practical cases additional factors should be taken into account, such us changes of pipe diameters, changes of the pipe material, junctions of pipelines and local leaks in pipeline. As the universal way of proper solving of water hammer problem for single pipeline is still not recognized, it is obvious that it is impossible to consider big complicated systems of pipelines. However, some simple examples of different situations in pipeline systems may be analyzed in order to recognize the main factors and effects influencing the accuracy of the solution. Thus in the paper some simple examples of observed and calculated pressure characteristics for different scenarios are presented. The aim of the research is to compare measured values of pressure with those calculated with use of the simple water hammer model (1a,b), and to recognize the difficulties in modeling and main factors affecting the results.

3. Experimental research

Experiments were carried out in the laboratory of Warsaw University of Technology, Environmental Engineering Faculty, Institute of Water Supply and Water Engineering. The physical model is schematically shown in Fig.1. The main element is the pipeline (single straight pipe of the length L , extrinsic diameter D and the wall thickness e or the pipeline consisted of sections of varied parameters (4)). The pipeline was equipped with the valve (1) at the end of the main pipe, which was joined with the closure time register (2). The water hammer pressure characteristics were measured by extensometers (3), and recorded in computer's memory. The supply of the water to the system was realized with use of hydrophore reservoir which enabled inlet pressure stabilization.

The experiments were carried out for four cases:

- simple positive water hammer for the straight pipeline of constant diameter (4); the measured characteristics were the basis for estimation the influence of the diameter change and local leak on water hammer run (Fig.1a);
- positive water hammer in pipeline with single change of diameter: contraction and extension (Fig.1b);
- positive water hammer in pipeline with local leak (Fig.1c) in two scenarios: with the outflow from the leak to the overpressure reservoir and with free outflow from the leak (to atmospheric pressure, with the possibility of sucking in air in negative phase).

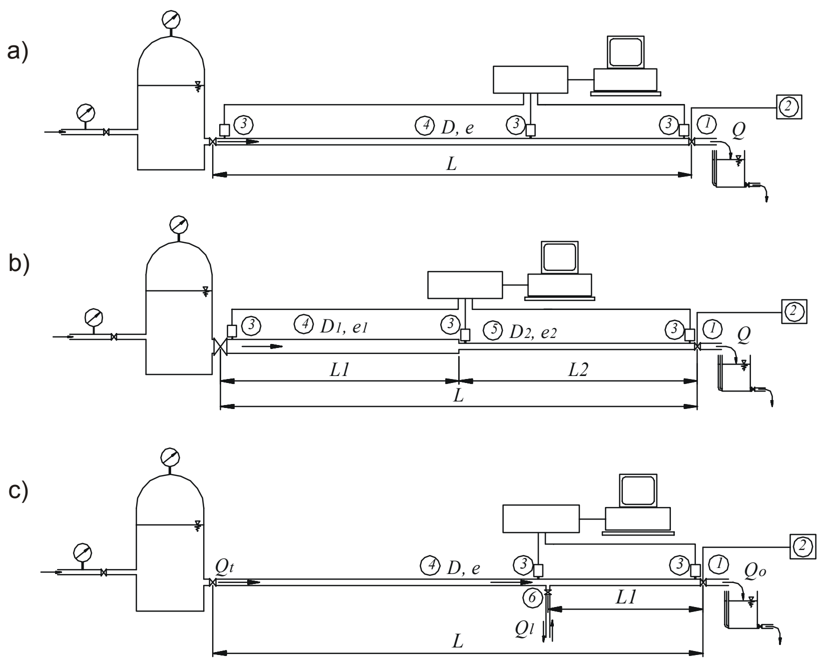

Fig. 1 Scheme of the experimental equipment
simple pipeline, b) pipeline with diameter change, c) pipeline with local leak

The experiments for each scenario were carried out in two steps. During the first step the values of steady flow discharges in the outflows were estimated (by volumetric method) and in the second step the unsteady flow pressure characteristics were measured. Next, the analysis of registered pressure characteristics was carried out, with estimation of the characteristic parameters, such as: water hammer wave period, pressure amplitude, number of oscillations, duration of the phenomenon. The purpose of this analysis was to indicate the factors that may influence the energy dissipation in water hammer in pipeline and to compare the run of the phenomenon for different cases. The pipeline parameters for different scenarios are presented in Tab.1.

Tab.1 Pipeline parameters for different experimental scenarios

Type of phenomenon	Experimental series	Type of pipeline	Pipe length	External diameter	Pipe wall thickness	Theoretical wave celerity
			L	*D*	*e*	*a*
	[-]	[-]	[m]	[mm]	[mm]	[m/s]
1	*2*	*3*	*4*	*5*	*6*	*7*
Simple water hammer – single pipe	S1	steel	41.00	48.0	3.00	1260
	S2		77.80	60.0	3.35	1369
	P1	PE80	36.00	50.0	4.60	256
Pipeline with variable diameter	SS1 (c)	steel	26.50/24.60	48.0/27.5	3.00/3.00	1324/1366
	SS2 (e)		18.40/26.50	27.5/48.0	3.00/3.00	1366/1324
	PP1 (c)	PE80	24.25/25.00	50.0/25.0	4.60/2.30	256
	PP2 (e)		25.00/24.25	25.0/50.0	2.30/4.60	256
Pipeline with local leak	LL	PE80	36.00	50.0	4.60	256

4. Numerical simulation of the phenomenon

For the needs of analyses presented in the paper, the simplified description of the phenomenon, in the form of equations (1a,b), was assumed as a mathematical model. To solve the problem, the Preissmann's scheme (Cunge 1980) was applied as numerical method. Computational time step was matched in a way enabling to obtain Courant number close to unity for each scenario. Unphysical oscillations were reduced by appropriate choice of the value of θ parameter in Preissmann scheme. For each of the considered scenarios, in the first step of calculations the values of λ and preliminary values of wave celerity a were estimated, which were then corrected in the further phase of calculations. Eventually, the main calculations were carried out for each considered scenario. In calculations the possibility of wave celerity changes due to density variations caused by pressure changes was taken into account, and changes of the friction factors due to velocity variations were allowed. Also the influence of the changes in different parameters were on the calculation result was analyzed.

As a consequence, the solution in the form of pressure characteristics was obtained. The examples of observed and calculated pressure characteristics for the chosen scenarios are shown in Fig.2 and Fig.3. More detailed analysis of the case of pipeline with diameter changes may be found in the work of Malesinska (2002), and of the case of pipeline with local leak – in the paper of Kodura and Weinerowska (2005). In the presented paper some examples and general remarks are presented.

5. General remarks and conclusions

The physical and numerical experiments carried out in the presented research and the comparison of observed and calculated pressure characteristics enable to draw some general conclusions and suggest directions of future analyses that could be developed to improve the water hammer modelling.

For single pipeline, the experiments confirmed the well known problem of obtaining the coincidence between observations and calculations if the traditional mathematical description in the form if (1a, b) is applied and if no numerical effects are observed in the solution. Thus it is important to introduce to the equations additional factors which can better model water hammer in pipeline, especially the nature of energy dissipation and pipe wall deformation.

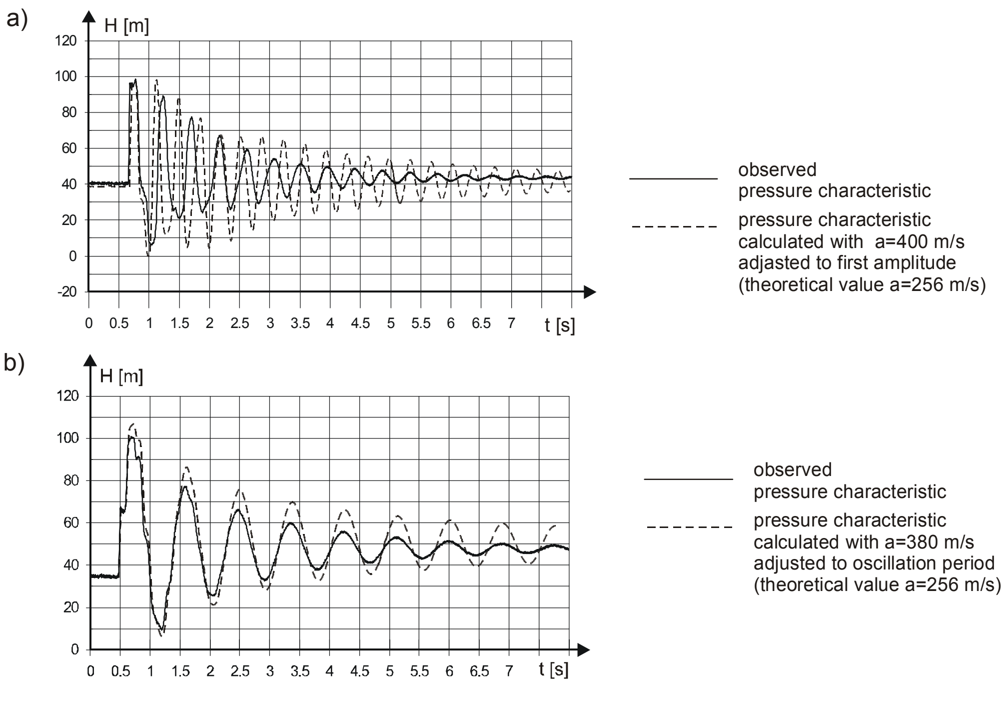

Fig. 2 Example of the measured and calculated pressure characteristics for the pipeline with change of diameter: a) contraction (PP1), Q=0,454 dm^3/s, b) extension (PP2), Q=1,059 dm^3/s.

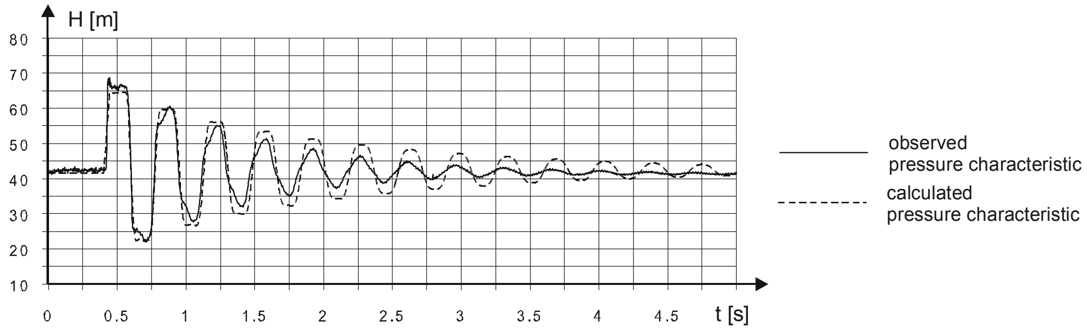

Fig. 3 Example of the measured and calculated pressure characteristics for the pipeline with local leak (LL), Q_t = 0.735 dm^3/s, Q_l =0.08 dm^3/s.

Very detailed analysis of the results obtained for the case of pipeline with change of the diameter can be found in the work of Malesinska (2004). On the basis of this work and the research presented in the paper one can draw the following conclusions:

- for the case of diameter extension in the pipeline:
  - the vivid and regular envelope of the pressure oscillations can be observed; however, the maximal value of pressure is not always the value of the first amplitude and it is usually higher than theoretical value obtained from Zukowski's formula;
  - the 'equivalent' value of wave celerity *a*, chosen to obtain the best fit between calculations and observations, is less than value of *a* for each pipe separately;
  - it is possible to obtain good coincidence between observations and calculations if numerical dissipation is introduced to the scheme; however, it is not possible to achieve such consistence for plastic pipes. For polymers in this case it is impossible to obtain simultaneously the fit of amplitude and frequency of oscillations;
- for the case of diameter contraction in the pipeline:
  - less regular envelope of the pressure oscillations can be observed; however, the maximal value of pressure is the value of the first amplitude and it is comparable with theoretical value obtained from Zukowski's formula;
  - the 'equivalent' value of wave celerity *a*, chosen to obtain the best fit between calculations and observations, is higher than value of *a* for each pipe separately.

If the pipeline with local leak is considered, the phenomenon run is influenced by some additional factors. Detailed conclusions drawn on the basis of experiments and calculations for the pipeline with a local leak are presented in the paper of Kodura and Weinerowska (2005). The most important effects observed are:

- lack of the influence of the ratio of discharge from local leak to total discharge in the pipeline to the values of period of oscillations, and – in a consequence – to the value of wave celerity if the outflow to the overpressure reservoir from the leak was imposed;

- no influence of the rate of discharge from local leak on the maximal value of pressure,
- consistence between observed values of maximal pressure in first amplitude and corresponding values calculated according to Zukowski's formula, irrespective of the rate of discharge from the leak;
- significant influence of the rate of the discharge from the leak on the vivid decrease of duration of the water hammer phenomenon, what suggests the possibility of utilization of this fact to the pipeline leaktightness assessment, especially that the duration time decreases with the increase of the outflow from the leak.

The results of the computations show that in some cases it is possible to obtain relatively good consistence with observations. However, it seems advisable to develop the model in order to improve the mathematical description of the phenomenon and to take into account the factors particularly important for each of the cases, modifying the run of simple water hammer.

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Analytical and Numerical Investigation of Sea Water Exchange in Marinas

Goran Lončar
Vladimir Andročec

Abstract: Analytical solution of sea water exchange process within some marina is very simple but excludes some important influences such as spatial position and shape of marina entrance, magnitude of sea currents outside of the marina and bottom slope inside the marina. 2D numerical model investigations are used to get insight into the relevance of those parameters on the rate of sea water exchange.
Numerical simulation with different boundary conditions are made for cases with flat and inclined bottom, different ratio of entrance/marina width, trapezoidal, triangle and rectangular marina oblique, and diurnal sea levels elevation with amplitudes of 0,25m and 0,75m which are characteristic for Mediterranean sea.
Values of used hydraulic parameters in numerical simulation such as bottom friction are valuated through the comparison of currents magnitudes within marina with values obtained on physical model investigation of some other authors.
Keywords: sea water exchange, numerical simulation

1. Introduction

Simple "one equation" analytical model of sea water exchange gives results dependent only on a ratio between water elevation amplitude and mean marina depth. Nature specifications such as spatial depth variation, entrance width and position, intensity of sea currents inside and outside the marina are not included. Beforehand and general conclusion about underestimated result obtained with analytical solution on temporally dynamics of sea water exchange is not always correct. Numerical 2D model has been used to get better insight and general remarks on overestimating or underestimating obtained with analytical solution.

Values of calibration parameters such as friction factor used for 2D numerical simulation are obtained through comparison with results on physical model investigation.

Characteristic dimensions of all investigated marinas are “mean” width L =435m and lenght also 435m. Marinas investigated forms of horizontal plane view are rectangular, triangular and trapezoidal (picture 1). Analysed entrance widths are d = 45m and 135m, which gives dimensionless ratio d/L = 0,1 ; 0,31. Furthermore, two possibilities of bottom inclination are investigated. Zero inclination which means flat bottom within whole analysed spatial domain inside and outside the marina $H=H_{av}$=6m and spatial depth variation only inside the marina from 10m depth, in the vicinity of entrance, to 2m depth on the coast line. Surface elevations are sinusoidal (diurnal) with amplitudes $\Delta H/2$ =0,25 and 0,75m and period of 24 h.

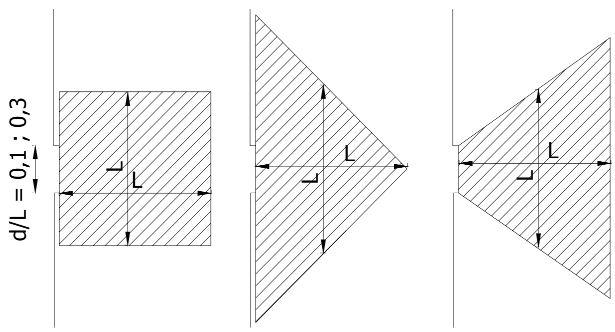


Figure 1 analysed marina plane cross section with same area extent (rectangular, triangular, and trapezoidal)

2. Analytical model

Basic parameter for analytical calculation of sea water exchange is difference between high and low mean sea level z_M. Analysed magnitudes are z_{M1}=0,5m and z_{M2} =1,5m. Area of marina is A = 189225m^2 with sea water volume V=1135350m^3. Volume of exchanged sea water in one period (n=1) of 24h (diurnal) for cases $\Delta H/2$ = 0,25m and 0,75m are 94612,5m^3 and 283837,5m^3 respectively, while not exchanged volume of marina sea water is $V_{1\,i} = V_0 - \Delta V_i = A*(H_{sr}–z_{M\,i})$ or $V_{1\,i} / V_0 = (H_{sr} – zM\ i) / H_{sr}$ = 1- $z_{M\,i} / H_{sr}$. After n-th period volume of „old“-not exchanged sea water will be $V_{n\,i} = V_0 * (1 - z_{M\,i} / H_{sr})^n$ and after that $n = log\,(V_{n\,i} / V_0) / log\,(1\text{-}z_{M\,i} / H_{sr})$. In case of $z_{M\,i}$ = 0,5m and 1,5m one will get values of n_i = 34,6 and 10,4 periods for 95% sea water exchange.

Continuous increases in exchanged sea water volumes inside the analyzed marinas obtained with this easy analytical tool are given on figure 6-9. Comparison with values obtained on 2D numerical model is also given. Obviously, bottom real configuration, general marina oblique, position and width of entrance, roughness, magnitude of sea currents outside the marina etc. don’t make any difference in calculation procedure and final results.

3. Numerical model

2D numerical model is based on finite difference technique. Used space increment are same in x, y (Δx=Δy=15m). Continuity (1) and momentum equations for x (2) and y (3) are:

$$\frac{\partial \xi}{\partial t}+\frac{\partial p}{\partial x}+\frac{\partial q}{\partial y}=\frac{\partial d}{\partial t} \tag{1}$$

$$\frac{\partial p}{\partial t}+\frac{\partial}{\partial x}\left(\frac{p^2}{h}\right)+\frac{\partial}{\partial y}\left(\frac{pq}{h}\right)+gh\frac{\partial \xi}{\partial x}+\frac{gp\sqrt{p^2+q^2}}{C^2\cdot h^2}-\frac{1}{\rho_w}\left[\frac{\partial}{\partial x}\left(h\cdot\tau_{xx}\right)+\frac{\partial}{\partial y}\left(h\cdot\tau_{xy}\right)\right]-\Omega q$$
$$-fVV_x+\frac{h}{\rho_w}\frac{\partial}{\partial x}\left(p_a\right)=0 \tag{2}$$

$$\frac{\partial q}{\partial t}+\frac{\partial}{\partial y}\left(\frac{q^2}{h}\right)+\frac{\partial}{\partial x}\left(\frac{pq}{h}\right)+gh\frac{\partial \xi}{\partial y}+\frac{gq\sqrt{p^2+g^2}}{C^2\cdot h^2}-\frac{1}{\rho_w}\left[\frac{\partial}{\partial y}\left(h\cdot\tau_{yy}\right)+\frac{\partial}{\partial x}\left(h\cdot\tau_{xy}\right)\right]+\Omega q$$
$$-fVV_y+\frac{h}{\rho_w}\frac{\partial}{\partial y}\left(p_a\right)=0 \tag{3}$$

where h water depth (h=ζ-d), d time varying water depth, ζ surface elevation , p and q fluxes densities in x and y direction, C Chezy resistance coefficient, g gravitation, $f(V)$ wind friction factor, Vx and Vy wind speed components in x and y direction, Ω Coriolis parameter, $p(a)$ atmospheric pressure, ρ_w density of sea water, x, y space coordinates, t time, τ_{xx}, τ_{yy}, τ_{zz} effective shear stress. Effective stresses in above conservation momentum equations include subgrid unresolved turbulence effect, and all effects arise due to averaging over whole vertical column. These members are defined due to eddy viscosity concept based on Smagorinsky formulation. Advection-dispersion equation (4) for dissolved or suspended substances in two dimensions is in fact mass-conservation equation.

$$\frac{\partial}{\partial t}(hc)+\frac{\partial}{\partial x}(uhc)+\frac{\partial}{\partial x}(vhc)=\frac{\partial}{\partial x}\left(h\cdot D_x\cdot\frac{\partial c}{\partial x}\right)+\frac{\partial}{\partial y}\left(h\cdot D_y\cdot\frac{\partial c}{\partial y}\right) \tag{4}$$

where c compound concentration, D_x and D_y dispersion coefficients in x and y direction with accepted values 0,1inside the marina and 0,3 outside the marina.

Variation of all investigated marinas geometry, bathymetry, hydraulic parameters and boundary conditions in numerical model are realized due to 24 different set up (table 1). The same space domain with 17500 points (figure 2) is used for all 24 numerical model set up

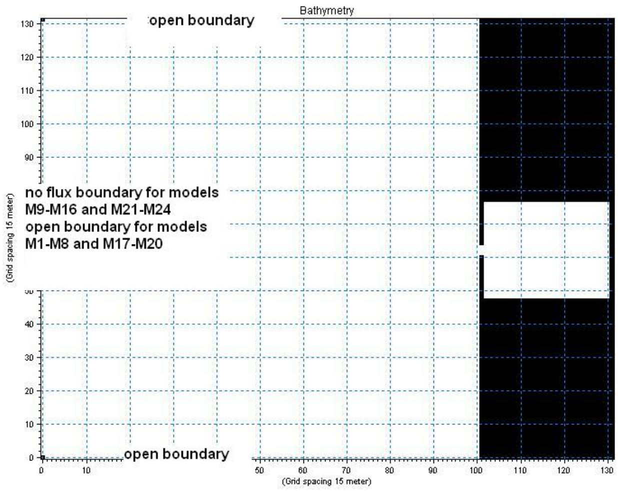


Figure 2 numerical model space domain with open and no flux boundaries (*d*/*L*= 0,1→M1-M4)

Numerical simulations with nomenclature M1-M8 and M17-M20 (table 1) have open boundaries on north, west and south side of space domain (figure 2). Bounadary condition are defined as temporarily variable surfaces elevation in sinusoidal form with amplitude $\Delta H/2$ = 0,25m ; 0,75m about mean sea level and period of 86400s (diurnal). Boundary conditions in numerical simulations M9-M16 and M21-M24 are also sinusoidal surfaces elevation but with phase shift of 26s between north and south open boundaries to gain temporarily variable sinusoidal current speed (0-10 cm/s, average ≅ 5cm/s) and current direction (S→N and N→S) outside the marina. In that case west boundary is no flux boundary.

Calibration procedure is done through comparison with results based on physical model investigation [3]. Physical model has been built with horizontal length scale L_r = 400 and vertical distortion n = 10. Physical model of rectangular marina has ratio d/L = 0,1 uniform bathymetry (H = 6m) and sinusoidal surface elevation boundary condition with amplitude $\Delta H/2$ = 2m and period of 44712 seconds (semidiurnal). Entrance is not centered as in figure 1,2 than appointed on north side of marina.

Comparison of maximum instantaneous *x*-direction current speed in profile of marina symmetry (N to S), averaged in vertical direction, measured on physical model and calculated on numerical model is given on Figure 3. Similarity of results is satisfactory what confirm accuracy of used numerical model hydraulic calibration parameters.

Table 1 nomenclature and conditions of investigated numerical model set up with used values o.s.e. → only surface elevation ; s.e.+ c. → surface elevation + currents (N→S=S→N=5cm/s)

model simulation	*d/L*	Surface elevation	bathymetry	boundary
M1	0,1	0,5	uniform - □	o.s.e
M2	0,31	0,5	uniform - □	o.s.e
M3	0,1	1,5	uniform - □	o.s.e
M4	0,31	1,5	uniform - □	o.s.e
M5	0,1	1,5	uniform - ◁	o.s.e
M6	0,31	1,5	uniform - ◁	o.s.e
M7	0,1	1,5	uniform - ▷	o.s.e
M8	0,31	1,5	uniform - ▷	o.s.e
M9	0,1	0,5	uniform - □	s.e.+ c.
M10	0,31	0,5	uniform - □	s.e.+ c.
M11	0,1	1,5	uniform - □	s.e.+ c.
M12	0,31	1,5	uniform - □	s.e.+ c.
M13	0,1	1,5	uniform - ◁	s.e.+ c.
M14	0,31	1,5	uniform - ◁	s.e.+ c.
M15	0,1	1,5	uniform - ▷	s.e.+ c.
M16	0,31	1,5	uniform - ▷	s.e.+ c.
M17	0,1	0,5	inclined - □	o.s.e
M18	0,31	0,5	inclined - □	o.s.e
M19	0,1	1,5	inclined - □	o.s.e
M20	0,31	1,5	inclined - □	o.s.e
M21	0,1	0,5	inclined - □	s.e.+ c.
M22	0,31	0,5	inclined - □	s.e.+ c.
M23	0,1	1,5	inclined - □	s.e.+ c.
M24	0,31	1,5	inclined - □	s.e.+ c.

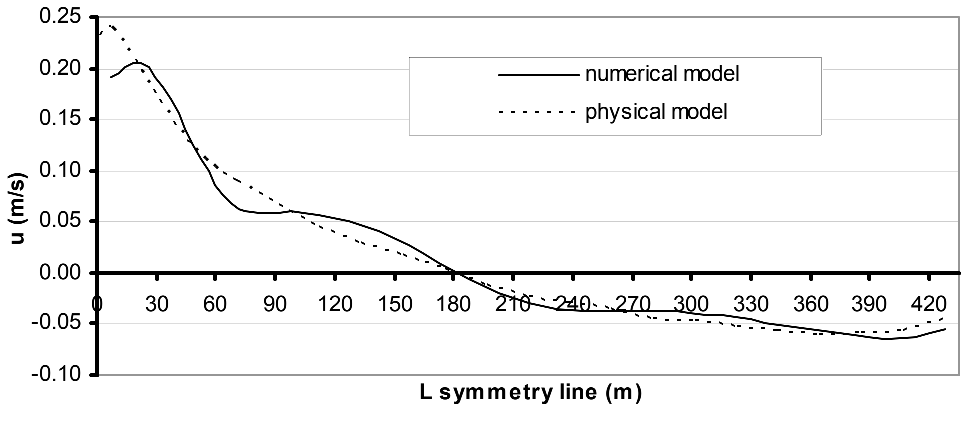
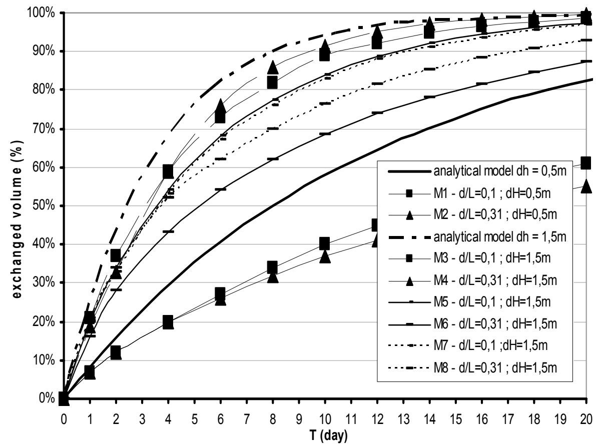


Figure 3 comparison between maximum instantaneous *x*-direction current speeds in profile of marina symmetry (N To S), averaged in vertical direction, measured on physical model and calculated on numerical model

4. Results of numerical simulation

Continuous 20 days increase in exchanged sea water volume ratio calculated with aim of analytical and numerical model is given on figures 4, 5, 6, 7 (M1-M24). Temporally variation of current speed *u* and *v* in *x* and *y* direction at mid point of marina entrance is given on figure

8. Sketched values represent 24 hours extraction from whole simulation period of 20 days (M6, M8, M14, M16-table 1). Values +*u* have W-E orientation and vice versa. Values +*v* have S-N orientation and vice versa.



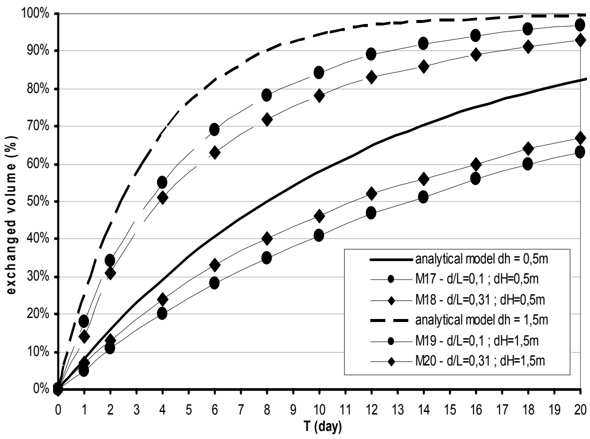
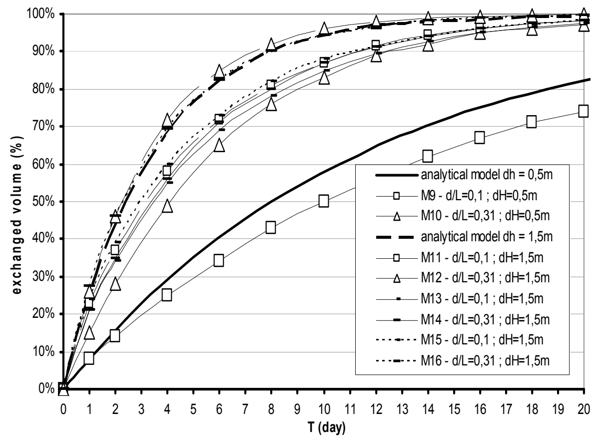


Figure 4, 5 increase in exchanged sea water volume ratio calculated with aim of analytical and numerical model (M1-M8 (4); M17-M20 (5))



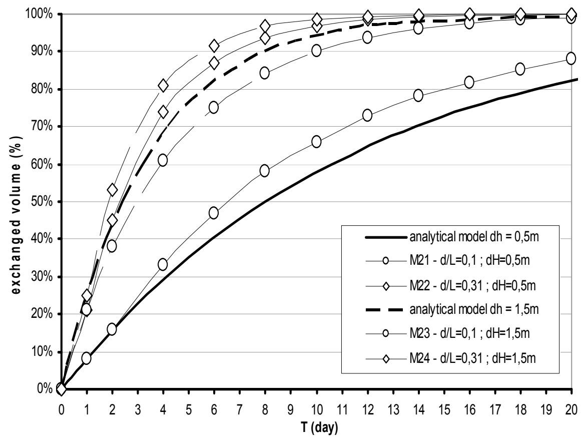


Figure 6, 7 increase in exchanged sea water volume ratio calculated with aim of analytical and numerical model (M9-M16 (6); M121-M24 (7))

Sea currents spatial distribution for M12 in vicinity of marina (extracted from whole numerical model space domain) and in time point when sea surface swell trough neutral-mean sea level is given on figure 9.

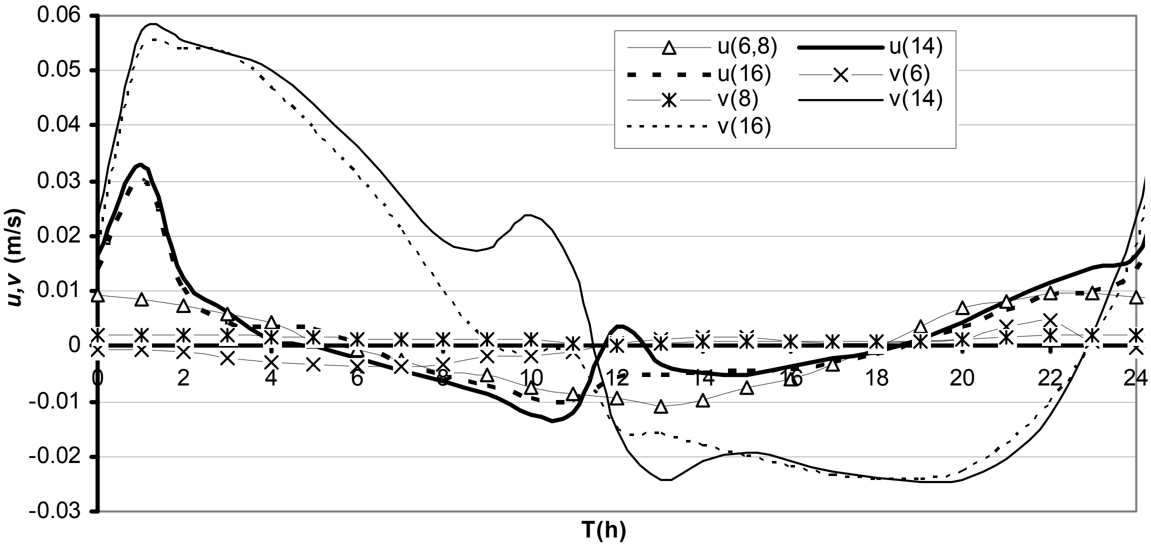


Figure 8 Temporally variation of current speed *u* and *v* in *x* and *y* direction at mid point of marina entrance (M6, M8, M14, M16)

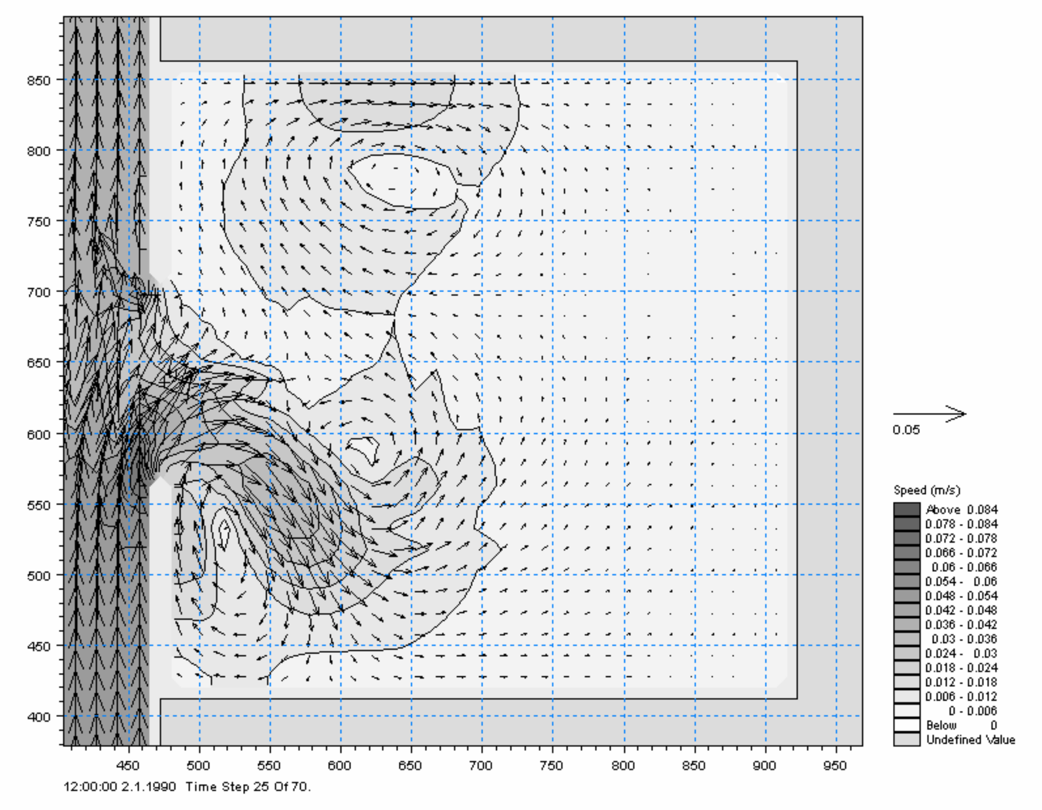


Figure 9 Sea currents spatial distribution for M12 in vicinity of marina (extracted from whole numerical model space domain) and in time point when sea surface swell trough neutral-mean sea level

5. Conclusion

When the sea currents outside the marina have small magnitude analytic “one-equation” model overestimates rate of sea water exchange and gives “optimistic” estimation. Overestimation is more pronounced in case of surfaces level oscillation with amplitude ΔH = 0,5m then for ΔH = 1,5m. Influence of bottom inclination is more pronounced in case with wider entrance (d/L = 0,31). In case of ΔH = 1,5m and flat bottom, rate of sea water exchange is higher then for inclined bottom in marina and vice versa in case of ΔH = 0,5m. Marinas with rectangular and trapezoidal plain section have slower water mass exchange in comparison with rectangular one. Furthermore for triangular and trapezoidal marina, lesser ratio d/L results in higher rate of exchange. Even though analytic solution overestimated the rate of exchange (figure 4, 5).

When the sea currents outside the marina have relatively high magnitude (time average 5cm/s) and ratio d/L = 0,31, analytic “one-equation” model underestimates rate of sea water exchange independent on amplitude of tidal elevation ΔH and bottom inclination. For marina with ratio d/L = 0,1 with flat bottom and ΔH = 0,5m as well as with inclined bottom with ΔH = 1,5m analytic “one-equation” model still overestimates rate of sea water exchange. Marinas with triangular and trapezoidal plain section have similar rate of sea water exchange which is slower then in case of rectangular one. Analytic solution overestimates rate of sea water exchange unless d/L = 0,31 and marina has triangular plain section (figure 6, 7).

From ecological point of view it could be advised not to built marina entrance in ratio d/L less then 0,3 for cases where sea currents outside the marina are low, bottom is flat and difference between low and high mean sea levels ΔH are less then 0,5m. Furthermore, for same conditions, ΔH=0,5m and nearly flat bottom, entrance to marina width ratio d/L should be more then 0,1 even though sea currents with averaged magnitude less then 5cm-s outside the marina are present.

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Physical Model Investigation on optimum Design of "U" shaped Weir in combined Sewer

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Abstract: Either due to reason of spatial restriction or economical optimization a weir in combined sewer often can't be carried out through classic form with one relatively long said weir. One of geometrically alternative solution is performing "U" shaped weir which enables augmentation of its relative length. Insufficiency of globally documented data related to hydraulic characteristics of such shaped weir, set demand for hydraulic model investigation on whole diversion plant with "U" shaped weir that is originally set for implementation to Zagreb City waste water discharge channel. Tests on physical model were focused on measuring speed and water levels at significant points in incoming and outgoing channels and diversion plant, with and without downstream inlet influence, and dependently on characteristic discharges of inlet channel at dry discharge to extreme discharge. Dynamic pressures were measured at weir wall and at stilling basin of overflow construction. Based on measurements, discharge coefficients and energy losses coefficients were calculated. The comparison between the velocities measured on physical model and one calculated on numerical model is also appended as like as comparison of discharge coefficients obtained on "U" shaped weir and classic side weir with similar dimensions. Certain modifications resulted with optimization of overflow functionality. Further more, analyses of the scam board removal efficiency of floating materials was carried out as same as their influence to upstream water surface elevation and decrease of discharge coefficient.

Keywords: "U" shaped weir, combined sewer

1. Introduction

During rain event with relatively high intensities treatment plant is not capable to accept whole expected discharged quantity (Q_{max}) which comes from incoming channel. Diversion plant with performed "U" shaped weir (figure 2), will separate incoming discharge into overflowing quantity which further flows through overflowing channel (Q_{max}-2Q_{dry}) to the final downstream recipient and discharge which goes to the treatment plant (2Qdry). Widely used shape of weir in combined sewer diversion plant is in form of relatively long side weir. Lack of investigation, measurement and results data about hydraulic efficiency for such "U" shaped weir, also intended to be built, set demand for physical model investigation. Schematic sketch with disposition of new and main combined sewer system elements already built in city of Zagreb-Croatia is given on figure 1.

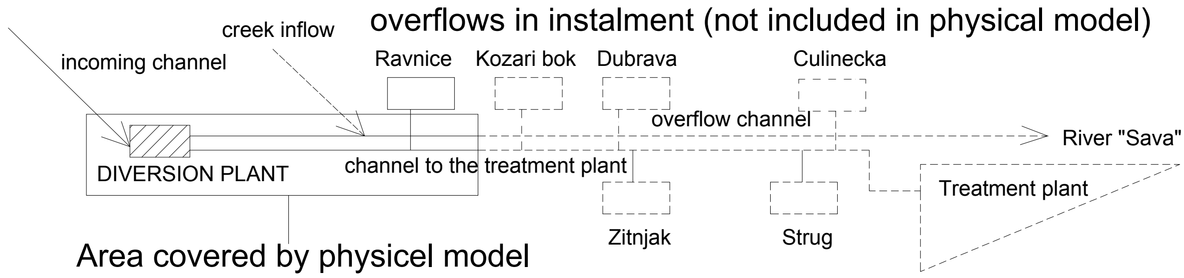


Figure 1 Schematic sketch of main combined sewer system elements already built in city of Zagreb

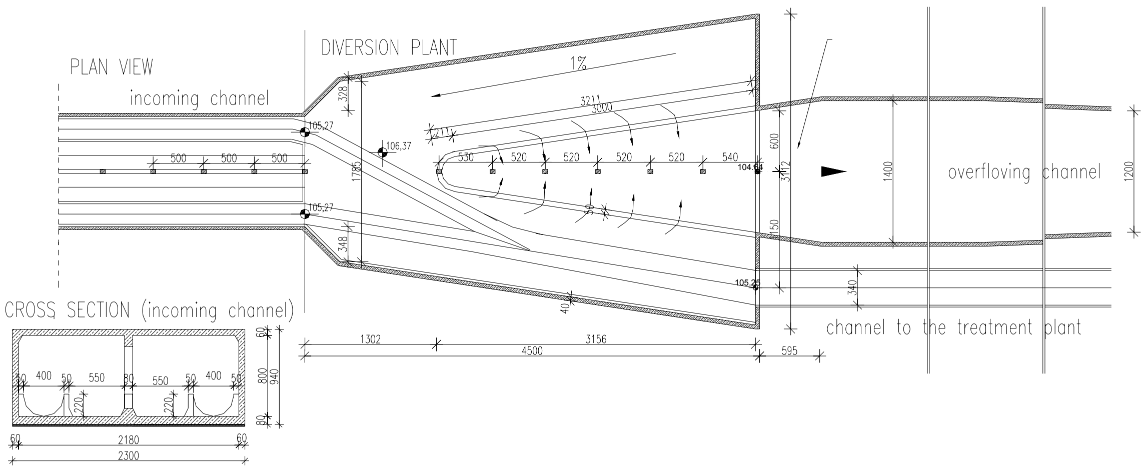
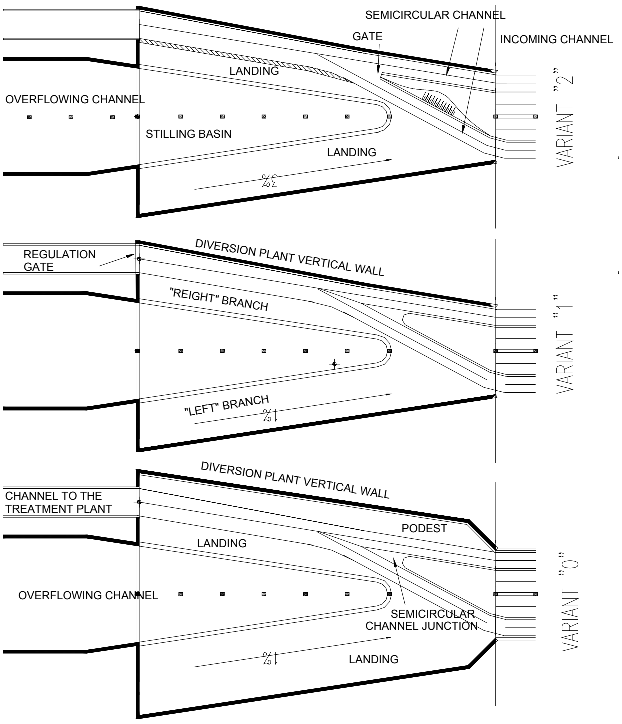


Figure 2 diversion plant in variant solution «0» and positions of incoming, overflowing and channel to the treatment plant

Investigation on physical model gave insight in main hydraulic characteristics connected with "U" shaped weir placed within diversion plant. After only three main geometry changes (variants "0, 1, 2, FS" – figure 2, 3abc, 4ab), executed on physical model, functionality of whole diversion plant was optimized in sense of its decreased extent and sustained discharge coefficient. Furthermore optimum geometry and position for planed scam board were determined.

Results of measurement obtained on physical model variant "0" showed that all hydraulic criteria demands were satisfied and that highest water surfaces level in incoming channel is below in project documentation defined channel ceiling. During the extreme discharge event overflowing at weir is drowned. One has concluded, that further improvement in sense of hydraulic functionality and decreased structure extent could be done. In variant solution "1" width of diversion plant has been decreased for 4,7m due tu približiti/primaknuti of vertical walls (figure 3b).



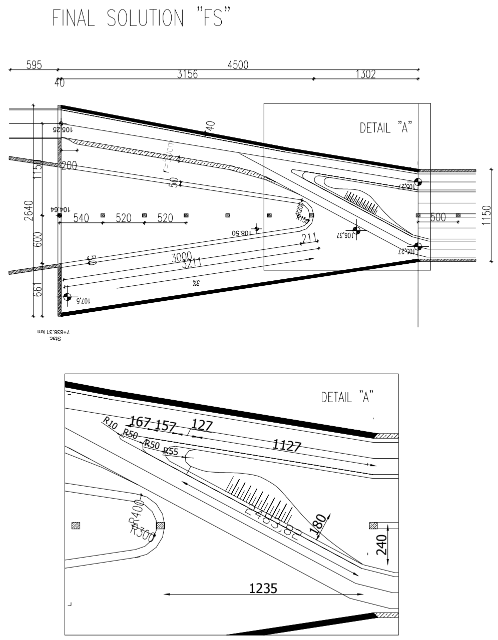


Figure 3a,b,c plan view for variant solutions "0", "1", "2" of diversion plant

Figure 4a,b plain view for variant solution "FS" of diversion plant

Analysis and comparison of measurements results on variant "0" and "1" give insight in minor change in hydraulic functionality. In mine time new demands on cross section of overflowing channel such as change to rectangular cross section with columns against old solution in trapezoidal form without columns should take place. Furthermore on position of semicircular kineta junction, gate for sediment removal should take place (variant solution "2" – figure 3c). Slope of landing has been increased from 1% to 3% to avoid higher rate of sedimentation. Intensive vorticityes on position of sharp semicircular channel top edges decreased after their smoothing with radius of 0,5m in vicinity of landing in diversion plant (figure 3c). With aim of flow visualization on variant solution “2” intensive vortices on position of gate for sediment removal has been observed. Thus gate was finely placed on the side of semicircular channel junction in the diversion plant and junction has been designed same as in variant solution “0” and “1”. This last investigated final solution has been named “FS” (figure 4a,b).

2. Physical model

Physical model with 4 in details different solution “0, 1, 2, 3, FS” has been built in Hydraulic laboratory on Faculty of Civil Engineering-Zagreb, Croatia. Model length scale was 16,7 without distortion and the model followed Froude similitude. Sketch of channels in and out from diversion plant with position of used measurement equipment is also given on figure 6. Photos of diversion plant, overflowing channel and downstream inflows built for model variant solution “0” are given on figure 5a,b. Investigated physical model overflowing heights at weir crest ware greater then 3cm, what is main condition for elimination of surface tension scale effect [1, 2].

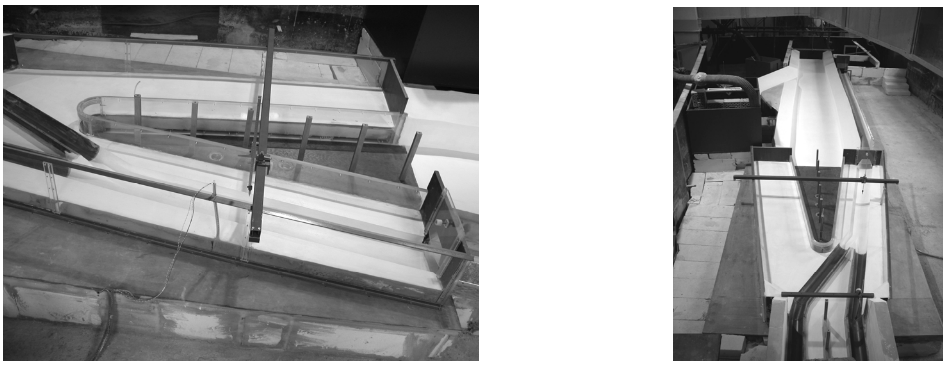


Figure 5a,b diversion plant with incoming and outgoing channels built on physical model variant solution “0”

3. Physical model tests and results

3.1 presure

Nomenclature of measurement conducted on all four variant solutions with six different boundaries –discharge conditions in incoming and overflowing channel is given in table 1.

Table 1 boundary conditions – discharges used in particular test

VARIANT 0, 1, 2, FS	Discharge condition in channel (m3/s) – test No.					
	1	2	3	4	5	6
incoming channel	3,6	5,8	7,2	65	65	83,7 (0;1) ; 78,0 (2;FS)
overflowing discharge	0	0	0	57,8	57,8	76,5 (0;1) ; 70,8 (2;FS)
inflow 1, 2	0	0	0	0	21 ; 14	21 ; 14

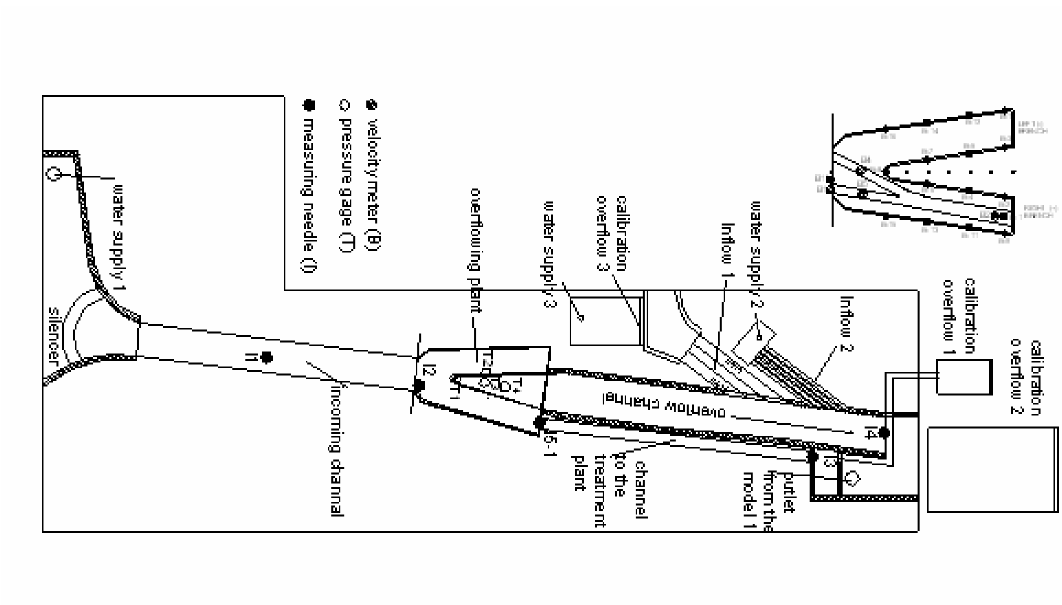


Figure 6 Schematic sketch of physical model extent with positions of used measurement equipment

3.2 Depths

Values of depth at measurement points I1, I2, I3, I4, I5-1 (see figure 6) for all experiments set up after table 1 are also compared with numerical model prediction (MIKE SHE). Depths obtained with aid of numerical simulations for same boundary conditions and at the same points in channels are less then values obtained on physical model tests in range 2-7 % are dependent on investigated variant solution and used boundary conditions. Only in test No 2, measured and on numerical model calculated heights ware practically the same for all variant solutions. In flowing condition connected with test No. 4 and 5 minimum water depths values have been reached in variant solution "FS". It was possible because the absence of drowned overflowing although overflowing channel was rectangular one with columns in the symmetry line. On contrary, in tests No.6 drowned overflowing occurred and water level heights ware higher in case of variant solutions "2" and "FS" then in "0" and "1". Main reason for that are added columns in overflowing channel with decreased cross section area.

3.3 Pressure in stilling basin

Investigations on pressures during stationary overflowing at points T3, T4 at the bottom in stilling basin for tests No. 4, 5, 6 and for all variant solutions have shown that dynamic pressure component increased static component for 25-30%. Average amplitude of pressure fluctuation is close to 0,5 m.

3.4 Water levels in diversion plant

Shape and values of water levels heights over weir crest level and at side walls of diversion plant are given in figure 7. Shown levels are obtained under conditions specified for test No. 4 and for variant solution 0, 1, 2. Distances from beginning of weir to measurement points in left branch have "-" and in right branch "+" mark (see figure 6). As stated earlier no drowned overflowing occurred in test No.4 independent of variant solution.

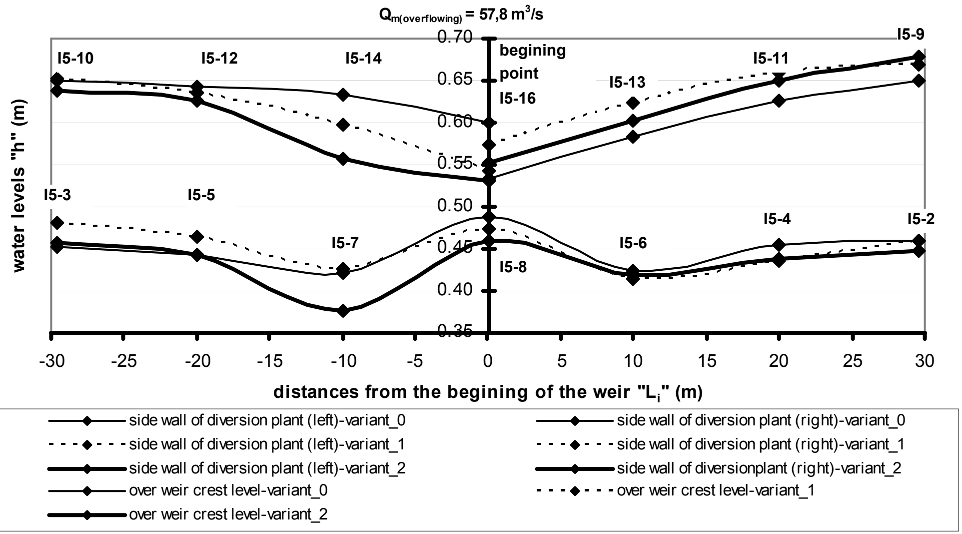


Figure 7 Shape and values of water levels heights over weir crest level and at side walls of diversion plant (test No. 4 – variant solutions 0, 1, 2)

3.5 Discharge coefficient

Value of discharge coefficient for undrowned overflowing over side weir in rectangular uniform channel with heights $H1$ = 0,57m ; $H2$ = 0,65m (figure 8), that are similar to values obtained at investigated „U" shape weir for variant solution „0" is Cd_0 = 0,595 (Cd = 0,375+0,251 ($H1/H2$)) – Kremeneškin [3].

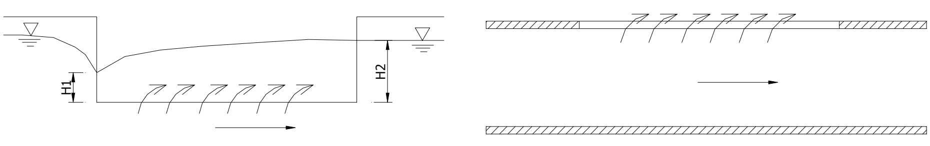


Figure 8 boundary water levels H1 and H2 for udrowned overflowing over side weir in rectangular uniform channel similar to those obtained at "U" weir in diversion structure (test No. 4 – variant solution 0)

Discharge coefficient Cd_i obtained from tests No. 4, 5 (undrowned) and No. 6 (drowned) for all variant solutions 0, 1, 2, FS are divided with Cd_0 and calculated nondimensional values are shown in table 2. Values of discharge coefficients Cd_i for “U” weir in all variant solutions has been calculated from calibration equation:

$$C_d = \frac{3Q}{2b\sqrt{2g}H_0^{3/2}}$$

where Q discharge over weir, b weir length, H_0 overflowing height at measurement point I5-10 (see figure 6).

Table 2 nondimensional discharge coefficients Cd/Cd_0 for variant solutions 0, 1, 2, FS obtained during tests No. 4, 5, 6

Cd/Cd_0	NEPOTOPLJENO	POTOPLJENO	
	Qoverflowing =57,8 m3/s	Qoverflowing =76,5 m3/s	Qoverflowing =70,8 m3/s
VARIANT 0	1,050	0,869	-
VARIANT 1	1,024	0,827	-
VARIANT 2,FS	1,076	-	0,632

3.6 Scam board influence

Four different lining and submerge depth of scam board in division structure have been examined (figure 9). These have influence on buoyant sediment removal efficiency and discharge coefficient of investigated “U” weir. Nondimensional ratios of water depths measured at points I1, I2, I3, I4, I5 and calculated discharge coefficients on variant solution FS under the flow condition No.4 with (index FS+S_i) and without scam board (index FS) is given in table 3.

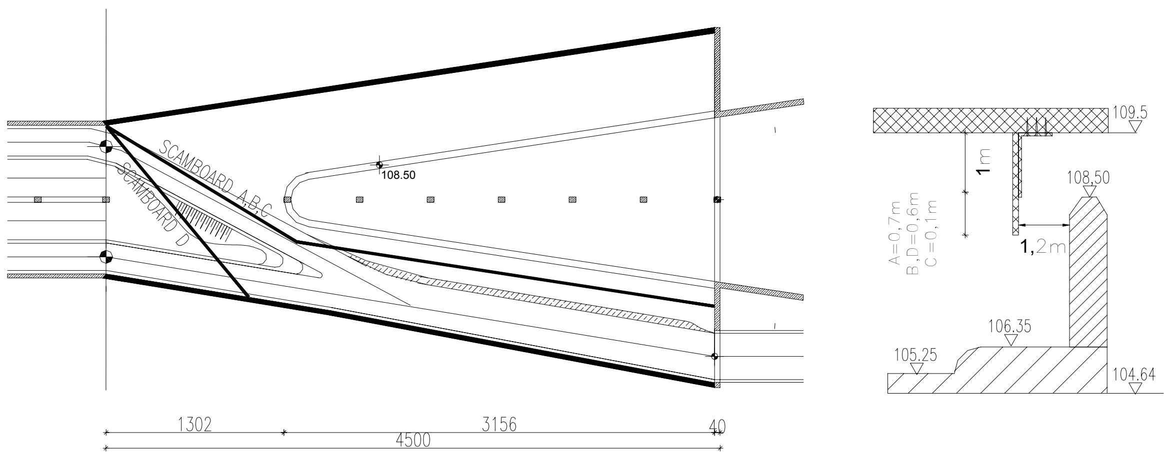


Figure 9 geometry and lining of examined scam boards in division structure (variant solution FS)

Table 3 comparison of depths and discharge coefficients dependency on used scam board type

TEST 4	h_{FS} / h_{FS}+Si					Cd $_{FS}$+Si / Cd $_{FS}$
Variant	I1	I2	I3	I4	I5-1	
FS+S_A	1,03	1,03	1,00	1,03	1,01	0,908
FS+S_B	1,02	1,02	1,00	1,03	1,01	0,930
FS+S_C	1,01	1,01	1,00	1,02	1,00	0,975
FS+S_D	1,07	1,07	1,00	1,03	0,99	0,953

4. Conclusion

Four variant solutions of division structure with “U” shaped weir and all incoming and outgoing channels included in project documentation has been investigated on physical model which followed Froude similarity conditions. Six different flow conditions in incoming channel as part of boundary condition have been established on all variant solutions. After acquisition and analysis of measurements data recordings of velocity, pressures and depths on each variant solution, geometry of particular modeled part was changed what consequence either in decrease of building cost or in improvement of hydraulic functionality. Influence of four different scam board geometry placed into the division structure of final solution “FS” were also investigated.

Hydraulic loss coefficient of whole division structure in final variant solution “FS” calculated on base of equation $\Delta h = \xi_{diversion\ plant} *(v^2/2g)$ was $\xi_{diversion\ plant} = 11$ where v represents average velocity in incoming channel. For comparison, on first investigated variant solution “0”, calculated values of hydraulic loss was $\xi_{diversion\ plant} = 22$. Moreover, division structure in final variant solution was averagely 4,7m narrower then in first investigated variant solution “0”.

Scam board type B with submerge depth 60cm under the weir crest level caused minimum and accepted water surface elevation in whole modeled area. Decrease of discharge coefficient in comparison with the same variant solution without scam board was 7%. Floating material used in model tests ware also permanently hold on that type of scam board.

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Highway Drainage System Efficiency

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Dražen Vouk

Abstract: Starting from 1980, all highways in ecologically sensitive areas in Croatia, have specially designed drainage systems. That includes system of closed or open conduits for runoff collection with oil-grit separators before the final disposal. In more sensitive areas, different BMPs are added after oil-grit separators, to improve overall treatment efficiency. Overflow structures are allowed on big watersheds to avoid large pipe diameters. While overflow structures in urban sewage systems are designed upon German ATV – Rules and Standards, water authorities in Croatia, for the same type of structure proscribe more stringent criteria, namely critical rain intensity from 15-25 l/s/ha depending on ecological sensitivity of the recipient. At the same time, the mentioned criteria are not correlated to local climatic conditions i.e. duration of dray and wet weather, and mean annual rainfall. The question is how much that stricter criteria, which have impact on final cost of highway drainage, contribute to overall drainage efficiency.
On the example of two Croatian cities with considerably different climatic characteristics, the runoff volumes and masses of pollution for different overflow criteria are modeled on the same hypothetic section of highway. Using input data from the case studies on pollution buildup and washoff undertaken in USA, results of simulation with USEPA SWMM-5, are analyzed and compared with overall highway drainage efficiency. As a final conclusion some recommendations regarding design criteria are given.
Keywords: highway drainage, pollution buildup, pollution washoff, SWMM-5

1. Introduction

Since 1980s, in the Republic of Croatia there have been special regulations imposed for the drainage design, while building the highways. The aim of those rules is the protection of underground and surface waters and soil, from the adverse impact of the highway runoff.

The drainage system consists of water resistant open and/or closed conduits, which collect the runoff, oil-grease separators (OGS) where the floatables and settleables are separated, and the outfall to the ground or the watercourse. If the roads go through the ecologically more sensitive areas, besides the OGS, additional best manage practices BMPs must be used, in order to achieve additional effects in the runoff treatment [1].

At the very large catchment areas, the overflow structures (OS) with immersed baffles are allowed. They prevent the outflow of floatable oils and greases. The criterion for dimensioning of OGS and OS is the critical rain intensity, i.e. critical inflow that runoff produces at the facility site. Critical inflow determines the beginning of overflow at OS, and with OGS, it determines the necessary area between two emerged baffles.

2. Problem definition

The first OGSs built in Croatia were dimensioned according to Swiss regulations [2]. Actually, the directives for designing highway drainage in the Swiss canton of Zurich were completely copied. In these directives, the critical rain intensity i_{kr} = 10 l/s/ha, which is the rain with 15% exceedance frequency. The data are valid for the Swiss canton Zurich, of course.

As time went by, Croatian waters (Croatian Water Resources Management agency) started regulating higher critical intensities, explaining that they want to increase the efficiency and safety of OGS operation, and lessen the polluting impact at the ecologically sensitive area. First figures of i_{kr} of 10 l/s/ha rose first to 15, then 20 and finally 25 l/s/ha, which increased the separator building costs. There is some logic in that, but the question is: Is the higher efficiency of the drainage system worth those higher costs of building?

3. Possible solutions

In order to answer this question, it is enough to obtain rainfall intensity data for the specific location, for the last 10 years. By statistic data processing, it is possible to determine the frequency or duration of certain rain intensities. From the intensity – duration curve, it is very easy to determine area of intensity values, which is interesting from the engineers' point of view. Besides the intensity and duration data, it is possible to determine the ratio of rain volume above and under the critical intensity figures. For performing that, it is enough to have rainfall database and statistic data processing software. The rainfall database should be obtained anyway, for the IDF curve calculation. This kind of analysis could be enough for final defining of critical rainfall.

Computer long-term runoff simulations from the specific drainage area could achieve better solution. The retardation, drainage area shape and slope, flow through open and closed conduits, can greatly influence the formation of runoff hydrograph. In this way, the ratio of volume of water running through OGS or OS structure, and the volume released trough overflow could be calculated more precisely.

Both above mentioned possibilities are actually based on the hypothesis that relatively small quantities of untreated water will be released into the environment, and that most of the water will undergo the treatment. In the whole this procedure, however, it is not the quantity of released water that matters, it is the mass of pollution released, suspended or dissolved in that water. In order to determine the balance of pollution, it is necessary to know its dynamics of buildup and washoff at the drainage area.

3.1 Pollution buildup and washoff

This study does not go further into the history of theoretical and practical development, but states the following: Dust and dirt (DD) buildup on the road or urban catchment is generally non-linear phenomenon. At clean catchment, DD buildup at first has linear characteristics, but after some time the unit increment starts diminishing, and stops at some maximal value. That means that the local wind and the turbulences of the vehicles cause a certain quantity of pollution to “get lost” from the controlled catchments of the road. The quantity of the loss grows with the time between two rain events. That is the reason why the drainage system in the drier areas, will take less pollution through the BMP, than those in the wetter regions.

Pollutant buildup that accumulates within a land use category is described by either a mass per unit of catchment area or per unit of curb length. The amount of buildup is a function of the number of preceding dry weather days, and can be computed using different functions [3] [4].

The dynamics of the pollution washoff depends on numerous factors, but mostly on: rainfall energy, its duration, slope and texture of the catchment area. The rainfalls of greater intensity on steep and smooth surfaces will wash out more quickly and with more pollution from the catchment area, so that the concentration and mass of the washed out pollution will be greater than in the cases of slight rainfalls over flat and rough surfaces. Besides, it is important to say that different kinds of pollution have different dynamics of washing out, which mostly depends on the size and thickness of particles, and their connection with inorganic particles – carriers.

The dynamics of the pollution buildup and washoff from the catchment has been described by different mathematical models, that take account of the whole range of different variables, but the reliability of the simulation results is very little, if there are not enough data on the dynamics of the processes themselves. In that sense, there hasn’t been done anything in the Republic of Croatia. An analysis done at the illustration level has been done here. The intention was to show that the initial standards for drainage facilities designing, for the same level of protection, change according to the locations with different climate characteristics.

Well-known US-EPA software SWMM5 [5] was used as a tool, since it can do long term simulations of the runoff, estimating at the same time the buildup and the washoff from the catchment. While doing so, it is possible to calculate the concentration of pollution in time, and the total mass in the time intervals wanted.

Two weather services in Croatia were taken as examples. The first one was in Slavonski Brod, which is in the Continental part of the country, with the average rainfall of 700 mm, and the other one is situated in Parg, a small place in the region of Gorski Kotar, with the average rainfall of 1200 mm. Just the parallel sequence of rainfall data for three years was analyzed. The data time step is 5 minutes. It was supposed then, that a classic highway with exactly the same dimensions (length 1 km, width 20 m and slope 0,5%) was built at the areas of weather services. At the end of the section there is OS, from which the water flows to BMP, and the excess is relieved without treatment, directly in the environment.

It was supposed that DD build-up follows the saturation function [3]. Buildup B begins at a linear rate that continuously declines with time, until a saturation value is reached,

$$B = \frac{C_1 \cdot t}{C_2 + 1}$$

where C_1 = maximum build-up possible (kg/ha) and C_2 = half-saturation constant (days to reach half of the maximum build-up).

Half-saturation time of 5 days was chosen, and with maximal value of build-up of 80 kg/ha. DD was the only indicator of pollution because it is the best representative of the total pollution.

Pollutant washoff from a given land use category, occurs during wet weather periods and can be described in different ways [4]. In this specific case, exponential washoff function was used. The washoff load (W) in units of mass per hour is proportional to the product of runoff raised to some power and to the amount of buildup remaining, i.e.,

$$W = C_1 \cdot q^{C2} \cdot B$$

where C_1 = washoff coefficient, C_2 = washoff exponent, q = runoff rate per unit area (mm/hour), and B = pollutant buildup in mass (kg) per unit area or curb length.
In the case we are discussing, C_1 value was chosen to be 0,1, and C_2 value was 1.

4. Simulations

The sequences of rain intensities were analyzed first, and the intensity – duration calculation was done. The total quantity of rainfall in the period monitored was 1609 mm for Slavonski Brod and 3184 mm for Parg. The results for i_{kr} from 10 – 25l/s/ha were calculated, and can be seen in the Table 1.

The table clearly shows that the intensity of 10l/ha covers the presumption from the Swiss rules.

In the long-term simulation of the outflow from hypothetic catchments, the kinematic wave routing method with 5-minute time step was used, and the results are given in the Table 2.

Table 1 Exceedance frequencies of rain intensities for Slavonski Brod and Parg in period 2001-2003.

i_{kr} (l/s/ha)	Sl. Brod Exceedance frequency (%)	Parg Exceedance frequency (%)
10	11	16
15	7	11
20	3	5
25	2	3

Table 2 The ratio of treated and non treated volume of runoff and mass of DD

i_{kr} (l/s/ha)	Slavonski Brod		Parg	
	V_n/V_t (%)	DD_n/DD_t (%)	V_n/V_t (%)	DD_n/DD_t (%)
10	16,2	16,2	22,4	17,4
15	10,6	10,8	14,5	10,6
20	7,7	8,0	10,7	7,3
25	6,0	6,1	8,4	5,3

V_n non treated runoff
V_t total runoff volume
DD_n non treated mass of DD
DD_t total mass of DD

The results from Table 1 and 2 are shown graphically in Fig. 1

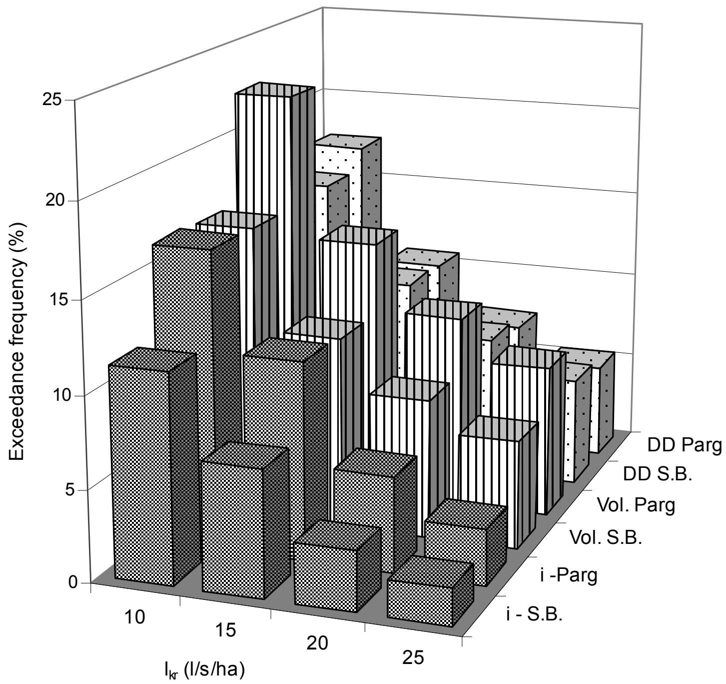

Fig. 1 Comparison of exceedance frequencies of rainfall, runoff volumes and mass of DD, for chosen critical rain intensities.

If daily build up of DD is supposed constant (t=1 day) in the period of 3 years, as the simulation period is that long, 14600 kg of DD settles on the road surface.

Table 3 shows the efficiency of the drainage system before entering BMP. If we suppose that the efficiency of BMP is 80%, we can get the overall efficiency of the drainage system.

Table 3 Ratio of treated and total DD, and overall efficiency of the drainage system.

i_{kr} (l/s/ha)	Sl.Brod		Parg	
	DD_{tr}/DD_t (%)	Overall eff.(%)	DD_{tr}/DD_t (%)	Overall eff.(%)
10	44	35	51	41
15	47	38	55	44
20	49	39	57	46
25	50	40	58	46

DD_{tr} treated DD

For the 2,5 times increase of useful surface of OGS, we get only 5 – 6% increase of overall efficiency of the drainage system.

5. Comment on the results

The presumptions for the choice of critical intensity of 10 l/s/ha taken from Swiss regulations, would be valid in Croatia, but with higher deviations for the areas with average annual rainfall above 1000 mm.

The results show that the increase of the i_{kr}, does not correspond with equal improvement of the highway drainage system efficiency, but it only increases the cost of building. This can be proved with hydrologic and hydraulic analysis, as well as with the analysis of pollution buildup and washoff, and especially by calculating the overall efficiency.

Additional analysis by changing the catchment geometry can prove the changes caused by the size or slope of the catchment, and the same thing could be done with change of the pollution buildup and washoff dynamics.

Summary

The present criteria for dimensioning of OGS and OS within the highway drainage system in Croatia are not satisfying. The critical intensities of rainfall above 15l/s/ha are generally unreasonably high and never elaborated. According to the possibilities so far, the rainfall data supply or the purchasing computer software is not a problem, and neither should be the minimal hydrologic rainfall analyses, from which an optimal critical intensity can be chosen. It should be enough that the water authorities regulate the minimal duration and minimal volume of the rainfall that must be let through the treatment system, before the final release.

The pollution buildup and washoff dynamics estimate gives the best indicators of drainage system efficiency, and it is a real challenge for the researchers. In that sense, the first field researches should be finally done.

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Predictings Sediment Loading into Masinga Reservoir and its Storage Capacity Reduction

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Abstract: It is estimated that the annual loss in storage capacity of the world's reservoirs due to sedimentation is around 0.5 – 1.0%. For many reservoirs, however, annual depletion rates are much higher and can go up to 4% or 5%, such that they lose the majority of their capacity after only 25 – 30 years. The Masinga reservoir, one of the main reservoirs in Kenya, designed for hydropower generation, public water supply and irrigation is faced with severe sedimentation. The designed sediment load into this reservoir in 1981 was estimated to be 3.0 x 10^6 m^3 per year (about 1% per annum reservoir reduction). By 2000, annual sediment loading had increased to over 11.0 x 10^6 m^3, nearly four times, thus reducing the designed capacity by more than 15%. As land degradation has become more evident with increasing land use change within Masinga catchment over the years, the operation and life span of Masinga reservoir is thus under imminent danger from erosion and sedimentation. There is need therefore to quantify spatially soil erosion and sediment yield reaching the reservoir with a view to reducing the sediment delivery. In this paper, a comprehensive procedure to predict spatial sediment yield and overall mean annual sediment volume delivered to Masinga reservoir is presented. Geographical Information System (GIS) technology as a tool to support soil erosion and sediment models is employed. Simulations of different land use and management scenarios are performed and their corresponding sediment yields estimated. Predictions show annual sediment loading into the reservoir of about 14.0 x 10^6 m^3 for land use practices in 2003. By simulating the best feasible management practices (BMPs), the achieved results show that the sediment volume reaching the reservoir could be reduced to about 6.0 x 10^6 m^3 per year.

Keywords: Reservoir, sediment yield, GIS, soil erosion modelling, catchment management

1. Introduction

Without careful planning, design and operation, the economic life of reservoirs can be shortened and thus the goods and services for which the project was constructed may not be sustained over the desired design period. The results of such a failure can impact local and regional economies and lead to considerable disruption. It is estimated that the annual loss in storage capacity of the world's reservoirs due to sedimentation deposition is around 0.5 – 1.0% according to World Commission on Dams (WCD) (2000). For many reservoirs, however, annual depletion rates are much higher and can go up to 4% or 5%, such that they lose the majority of their capacity after only 25 – 30 years.

Kenya's power generation is dominated by hydropower, which accounts for approximately 70% of the generation capacity. The *Seven Forks* hydropower system on the Tana River Basin provides most of this capacity and Masinga reservoir, which provides upstream regulation storage, is therefore critical for the smooth operation of the cascade system. Masinga reservoir acts as a regulating scheme for the lower dams in the cascade and any loss of storage capacity increases the risk of failure to meet the design objectives in dry periods. Although the emphasis was that the development of these multipurpose reservoirs would be the best measure of meeting Kenya's water demand by the year 2020 (Ongweny et al., 1993), it is evident that environmental problems such as soil erosion and silting of dams could curtail these efforts.

While the issue of Masinga reservoir sedimentation has been of interest in recent years, very little work has been done to estimate the spatial variability of sediment transport from the catchment. The key issue with reservoir sedimentation reduction lies within a proper catchment management.

1.1 Objective

The objective of this study is to apply a spatially distributed sediment delivery model in a Geographical Information System (GIS) environment to Masinga catchment with a view to predicting spatial sediment yield and mean annual sediment volume reaching the main Masinga reservoir.

2. Materials and methods

2.1 Study area

The Masinga catchment area (figure 1) is some 6,255 km^2 in extent, lying to the east of the Aberdare Mountains and south of Mount Kenya. It lies between latitudes 0° 7'S and 1° 15'S and longitudes 36° 33'E and 37° 46'E. The elevations range from 900 to 4000 m (a.m.s.l). The catchment falls within five agro-climatic zones ranging from semiarid in the east to humid in the western side. The mean annual rainfall vary from about 600 to 2000 mm with mean annual temperatures ranging from 21 to 31 °C. The catchment has an estimated population of 2 million people (Opiyo, 1999). The agricultural and grazing activities take about 86% of the total catchment area (Mutua, 2005).

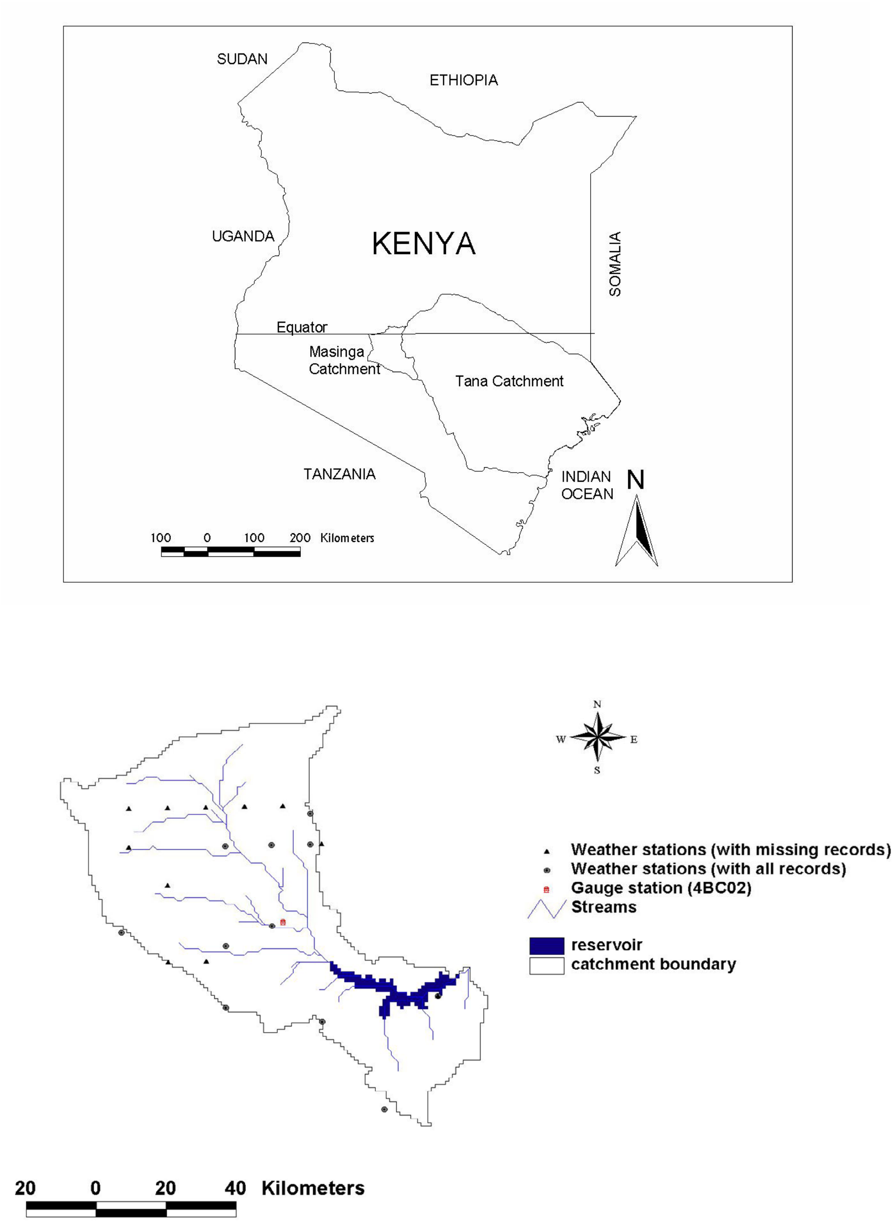

Figure 1 Location of study area on the map of Kenya

2.2 Predicting soil erosion rates

In this study, the Revised Universal Soil Loss Equation (RUSLE) was used to estimate the mean annual soil erosion. The RUSLE model was chosen in this study because its data requirements are not too complex or unattainable, it is relatively easy to parameterise, and it is compatible with GIS. When used in conjunction with raster-based GIS, the RUSLE model can isolate locations of erosion on a cell-by-cell basis, determine the role of individual variables on the rate of erosion, and identify the spatial patterns of soil loss within a catchment (Millward and Mersey, 1999).

In a raster GIS, the mean annual gross soil erosion was calculated at a cell level using six factors, which are composite factors of many others. The RUSLE model is given as:

$$A_i = LS_i \ R_i \ K_i \ C_i \ P_i \tag{1.1}$$

Where subscript i is the i^{th} cell; A (ton ha^{-1} yr^{-1}) is the estimated average annual soil loss; LS is the combination of the slope steepness and slope length factors; R (KJ mm m^{-2} h^{-1} yr^{-1}) is the erosivity factor; K (ton ha^{-1} KJ^{-1} mm^{-1} m^2 h) is the soil erodibility factor; C is the cover and management factor and P is the support practice factor.

Five primary data themes were required to generate the RUSLE factors. These were the digital elevation model (DEM), the climatic data (precipitation), soil data, land use coverage and conservation support practices. The DEM was required to derive the slope length (L) and the slope steepness (S) factors. The climatic data was required to develop the rainfall erosivity (R) factor. The soil type coverage was required to develop the soil erodibility (K) factor and the land use coverage was used to develop the crop management (C) and conservation practice (P) factors.

One major improvement made by using the RUSLE in this study was the application of upslope area contributing method in determining the slope length and steepness factor, which made the model to act on a semi-distributed form. The use of time series of remote sensing imagery and daily rainfall to incorporate the effects of seasonally varying rainfall intensity, and use of new digital maps of soil and terrain properties allowed the estimation of spatial seasonal erosivities for Masinga catchment.

2.2.1 Simulation scenarios

Four simulation scenarios were performed in this study to estimate the spatial soil erosion within the catchment. Scenario 1 was based on land use/cover and management practices for 2003 and formed the benchmark scenario. Scenario 2 was run by changing the conservation practices while maintaining other factors as in benchmark scenario. Scenario 3 was run by changing the land use and cover types but keeping the other factors constant as in benchmark scenario. The formulation of the new database for scenarios 2 and 3 was done in reference to different slopes, climatic zones, soil properties and the viable management practices for each sub-catchment. Scenario 4 was run by combining the new data sets formulated for scenarios 2 and 3. Predicted soil erosion rates for all scenarios were compared with the tolerable erosion rates for Masinga area. The scenario that gave the least erosion rates was taken to have the best management practices (BMPs) for Masinga.

2.3 Predicting spatial sediment yield

Sediment yield is usually not available as a direct measurement and it is estimated using a sediment delivery ratio (SDR). Erosion rates estimated by RUSLE are often higher than those measured at catchment outlets. Sediment delivery ratio (SDR) is thus used to correct for this reduction effect.

There is no precise procedure to estimate SDR, although the USDA-SCS (1972) published a handbook in which the SDR is related to drainage area. In this study, an attempt was made to develop and apply a spatially distributed sediment delivery model in a GIS environment to Masinga catchment. The developed model is known as the Hillslope Sediment Delivery Distributed (HSDD) model. To apply the model, the catchment was delineated and discretized into morphological units (i.e., areas of defined aspect, length, steepness). The morphological units were then aggregated into seven major sub-catchments (figure 2) based on the pour points (outlets) of the delineated stream network.

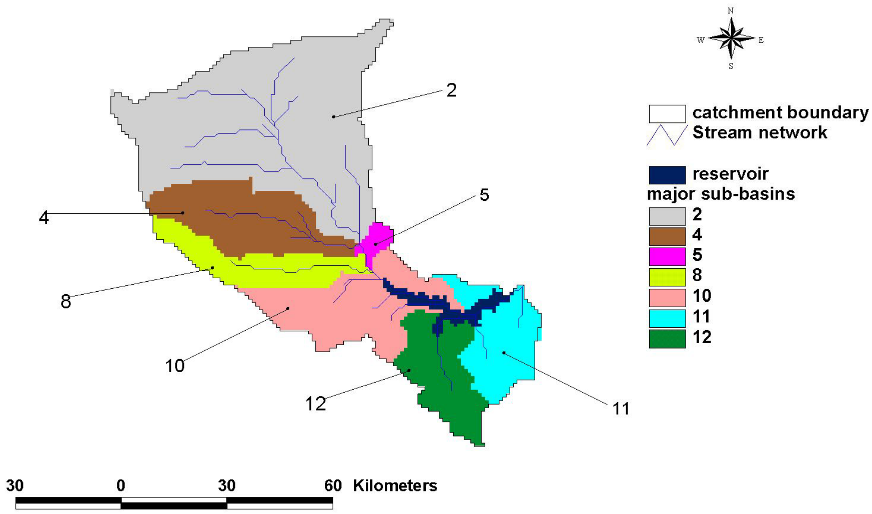

Figure 2 Major sub-catchments of the study area

The main spatial physical properties for each sub-catchment were averaged. Table 1 presents the summary of the average physical properties for the discretized sub-catchments.

Table 1 Main average attributes of the discretized sub-catchments

Basin ID	Soil *WHC (mm)	Soil Depth (cm)	Area (Km^2)	Hlength (m)	Hslope (m)	UpArea (km^2)	Elevation (m)	SCS Number CN	Max Cover	Manning Coeff.
2	117.178	94.3787	2758	21776.3	1.6912	2757	2143.9	76.4	0	0.065
4	126.942	101.006	821	23002.3	1.9787	820	1897.4	73.3	0	0.045
5	63.8816	102.938	76	5312.6	0.6324	3654	1198	79.8	0	0.025
8	108.168	97.9412	506	34384	1.865	505	1802.5	73.1	0	0.035
10	77.8595	121.901	918	16939.1	0.901	5078	1309.9	76.9	0.00106	0.035
11	112.868	195.415	597	13419.6	0.874	6261	1121	73.9	0.00147	0.075
12	88.6997	150.232	586	18397.8	0.9661	585	1213.9	75.4	0.00106	0.055

*WHC: water holding capacity; Hlength: hillslope length; Hslope: hillslope (m/100m); UpArea: upslope contributing area; SCS: soil conservation service curve number; Max cover: maximum % cover of land that is impervious.

A physically distributed hydrological model, the Stream Flow Model (SFM) was used to generate the sub-catchment response and flow velocity layers in a spatial domain. The SFM was developed using the “C” programming language. The user interface for the SFM was developed using the avenue script and loaded as an extension to the normal ArcView GIS graphical user interface.

Using the land use/cover in conjunction with soil information, rainfall incident on each sub-catchment was partitioned to separate surface runoff from water infiltrating into the soil. The land use/cover and soil data were also used by the SFM to calculate response function of each sub-catchment. The response function described how excess precipitation was routed to the outlet of each sub-catchment.

A relationship between the sediment delivery ratio (SDR) and the sediment travel time expressed as a function of the overland and channel flow, and sub-catchments' responses based on rainfall, evaporation, land cover and soil properties was established in this study. The relationship between SDR and the sediment travel time by the HSDD model is given as:

$$SDR = \exp(-\beta T_{ic}) \tag{1.2}$$

Where β is sub-catchment response coefficient, T_{ic} (hr) is the sum of the overland flow travel time t_o and the shallow concentrated flow travel time t_c of the sediment. It was assumed that the sediment that reaches the stream network takes the same travel time as the runoff.

The time for runoff water to travel from one point to another over the catchment was determined using the flow distance and velocity along the flow paths. This is expressed as:

$$t_i = \sum_{i=1}^{N_p} \frac{l_i}{v_i} \tag{1.3}$$

Where t_i (hr) is the travel time for cell i, l_i (m) is the length of segment i in the flow path based on the flow direction, v_i (m s^{-1}) is the flow velocity for the cell i and N_p is the number of cells traversed by runoff from cell i to the nearest channel. For a cell i, the cumulative travel time was estimated by summing the travel time along its flow path.

The surface runoff (excess rainfall) was estimated using the SCS curve number method. This was based on the relation:

$$Q_i = \frac{(P_i - 0.2 S_i)^2}{P_i + 0.8 S_i} \tag{1.4}$$

Where subscript i is the i^{th} cell, Q_i (mm) is the daily runoff, P_i (mm) is the daily rainfall and S_i (mm) is the retention parameter estimated using the relation:

$$S_i = 254\left(\frac{100}{CN_i} - 1\right) \tag{1.5}$$

The curve number CN_i for each grid cell was determined using land use/cover and hydrological soil group data. The flow velocity of the runoff was estimated using the Manning's equation based on the coefficient of velocity (equation 1.6). Velocity coefficients for each type of land use/cover were estimated using values given in Table 2 (after McCuen, 1998). The velocity was estimated using the relation:

$$v_i = (\alpha_i \, s_i^{1/2})\, q_i \qquad (1.6)$$

Where v_i is runoff velocity (m s^{-1}), s_i (m/m) is slope of cell i and q_i (m s^{-1}) is specific runoff rate (i.e. runoff rate per unit cell area).

Table 2 Relationship between land use/cover description and velocity coefficient

Land Cover Description	Velocity Coefficient
Urban and Built-Up Land	6.3398
Dryland Cropland and Pasture	0.4572
Irrigated Cropland and Pasture	2.7737
Cropland/Grassland Mosaic	0.3962
Cropland/Woodland Mosaic	0.3962
Grassland	0.6401
Shrubland	0.4572
Savanna	0.4267

3. Results and discussion

After running the model, a spatial SDR map (figure 3) was generated. The SDR varies from 0.11 to 1.0 within the sub-catchments and the overall sediment delivery ratio averaged for all the grid cells for the catchment is 0.29. The results show that the further away an area is from the stream, the longer the travel time and hence the lower the SDR. It should be emphasized that any two locations that are equidistant from the outlet may not have the same travel time. This means that travel time does not follow concentric zones. Flow velocity in reality is controlled by conditions such as the surface vegetation type and roughness, and elevation changes over the drainage area. In this study, it was established that longer travel time tended to occur in areas with rougher surfaces (vegetated areas) compared with bare and open land surfaces.

Sediment delivery ratio values obtained in this study did not exhibit a clear relation with the type of land use and land cover. This may be explained by the argument that sediment delivery ratio tends to be affected more significantly by the character of the drainage system than by the land use as shown in figure 3. However, the estimation of spatial sediment delivery ratio allows the identification of critical sediment source and delivery areas as well as site-specific implementation of proper management practices within the catchment. The sediment delivery ratio values imply the integrated capability of a basin for storing and transporting the eroded soil.

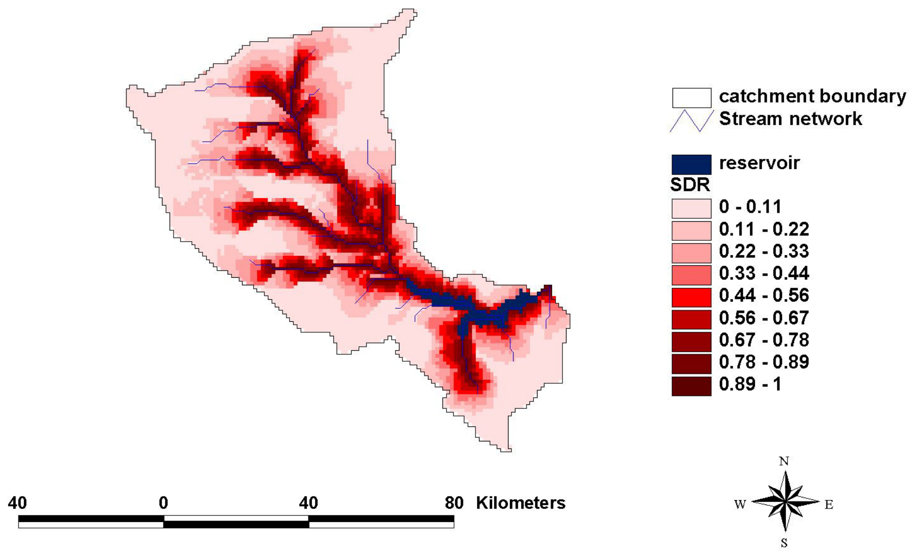

Figure 3 Spatial SDR for Masinga catchment

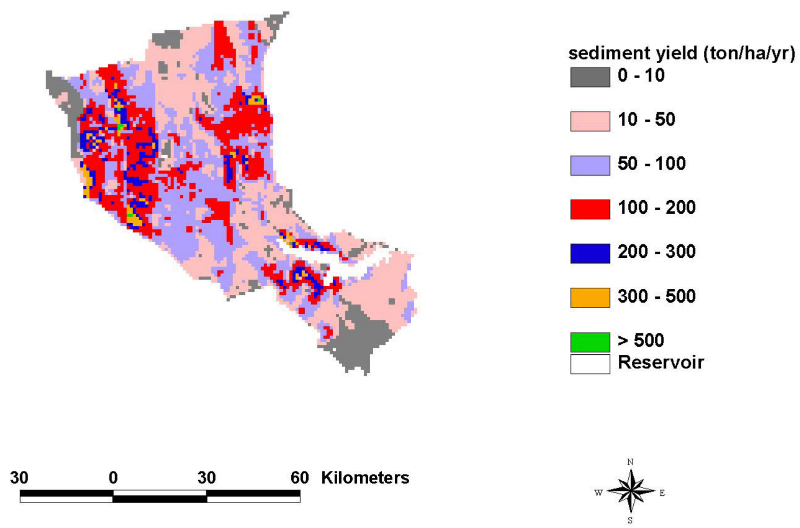

Figure 4 Spatial mean annual sediment yield for Masinga based on 2003 land use and management practices

The SDR map was overlaid with the mean annual soil erosion maps generated in section 2.2. For each scenario described in section 2.2.1, a spatial sediment yield map was generated. Figure 4 shows results of predicted spatial sediment yield based on land use and management practices for 2003. Results of figure 4 show critical source of sediment yield. The results show a great variation in sediment yield within each sub-catchment (Table 3). Such high variations are a result of the diverse land uses and the wide range of land slopes and distance to channels within the individual sub-catchments. The predicted average sediment yields at each sub-catchment outlet show that the sediment yield does not entirely depend on the catchment area but more so on the sub-catchment properties.

In this study, the overall mean annual sediment volume reaching the reservoir is predicted as 14.0 x 10^6 m^3 and by simulating the best management practices (BMPs), the predictions show that sediment loading into the reservoir could be reduced to about 6.0 x 10^6 m^3 per year.

Table 3 Variation of sediment yield within the sub-catchments

Sub-catchment No.	Area (km^2)	Mean annual sediment yield (ton ha^{-1} yr^{-1})	
		Sediment variation within sub-catchment	Mean sediment yield at sub-catchment outlet
2	2758	8.9 - 242.9	77.9
4	821	10.3 – 501.7	84.7
5	76	2.9 – 51.8	9.7
8	506	16.1 – 158.7	50.3
10	918	2.3 – 331.6	30.6
11	597	1.0 – 106.1	17.9
12	586	1.8 – 84.8	14.8

Summary

In this study, a new approach of predicting spatial sediment yield for Masinga catchment is presented. The proposed approach based on the concept of the runoff travel time from individual cells allows for the identification of primary sediment source areas and helps to identify and clarify those critical areas with high potential for sediment transport. It also predicts the spatially varying sediment transport capacity, and ultimately, the sediment yield from each area reaching the reservoir. Simulation results show that the RUSLE and the developed HSDD model integrated in a GIS environment can be used to facilitate fast and efficient assessment of different management alternatives, with a view to reducing sediment loading into reservoirs. Predictions show that annual sediment loading into Masinga reservoir based on land use and management practices for 2003 is 14.0 x 10^6 m^3. By simulating the best feasible management practices (BMPs) for this catchment the achieved results show that the loading rate could be reduced to 6.0 x 10^6 m^3 per annum. However, there is need for further fieldwork research to improve the parameters of the HSDD model especially through calibration and validation.

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Hydraulic Model Investigation of the Evacuation Structures of Dam St. Petka on Treska River

Cvetanka Popovska, Sretko Cvetkovski
Dragan Ivanoski, Violeta Gesovska

Abstract: Modern hydraulic engineering has been and remains to a large extent based on experimental investigations. Mathematical modeling techniques have progressed rapidly due to the advent of high-speed digital computers, enabling the equations of motion coupled with semi-empirical relationships to be solved for complex flow such as pipe network analysis, pressure transients in pipelines, unsteady flow in rivers, etc. There are many cases, particularly where localized flow patterns cannot be mathematically solved, when physical models are steel needed. The hydraulic model investigation of the evacuation structures of dam "St. PETKA" on Treska River has been performed in Hydraulic laboratory at the Faculty of Civil Engineering in Skopje. The model was designed and constructed in scale 1:40. The evacuation structures that have been investigated are: diverse tunnel with intake structure, shaft spillway with spillway tunnel and output portal and bottom outlet with intake structure. The main objectives of the study are: to confirm the disposition and dimensions of the associate structures, to define their capacities, to define the flow patterns, to define the pressure distribution along the tunnels, to define the backwater influence of the downstream reservoir and to investigate the erosion processes in the river bed downstream. The investigations were performed on two stages: fixed-bed model and movable-bed model. On the fixed-bed model were investigated the capacities, pressure distribution and flow patterns in the structures. On the movable bed-model were investigated the erosion processes in the river bed downstream of the output structure. In this paper will be discussed the summary results that have been obtained within the study on physical model and the focus will be on the results on the shaft spillway.

Keywords: hydraulic model, evacuation structures, flow patterns, discharge

1. Hydraulic Similitude

Physical model of dam “St. PETKA” with associate structures was constructed in scale 1:40. The reservoir has been modeled in length of ≈300m and width of 200m. Downstream part of the dam was modeled in length of ≈400m and width of ≈100m. The upstream and downstream boundaries of the model were selected in such manner to avoid their impact on the simulation flows in the structures. The scale of the model enables simulation of similar hydrodynamic conditions and accurate measurements of the parameters.

The model was designed and constructed according to the basic principles of hydraulic similitude, which are: geometric similarity, cinematic similarity and dynamic similarity. Two objects are said to be geometrically similar if the ratios of all corresponding dimensions are equal. The ratios of length (L_R), area (A_R) and volume ($\forall_R$) are 1:40, 1:1600 and 1:64000, respectively.

Two motions are said to be cinematically similar if: (i) the patterns of paths of motion are geometrically similar and (ii) the ratios of velocities of the particles involved in the motions are equal. The computed ratios of velocity (V_R), time (T_R) and discharge (Q_R) are 1:6.32, 1:6.32 and 1:10119, respectively.

Two motions are said to be dynamically similar if: (i) the ratio of masses of the objects involved are equal and (ii) the ratios of the forces which affect the motion are equal. At this point it is noted that while geometric and cinematic similarity can be achieved in most modeling situations, complete dynamic similarity is an ideal which can seldom, if ever, be achieved in practice (Anonymous, 1942). In this case dominant forces are gravity and viscosity. Therefore, dynamic similarity was achieved by Froude number similarity $(Fr)_M=(Fr)_P$ or the ratio $(Fr_R)=1$, and by Reynolds number similarity $(Re)_M=(Re)_P$ or the ratio $(Re_R)=1$. In these relations the subscripts ($_M$) and ($_P$) designate the model and prototype, respectively.

$$Fr = \frac{F_I}{F_g} = \frac{\rho V^2 L^2}{\rho g L^3} = \frac{V^2}{gL} \tag{1}$$

$$Re = \frac{F_I}{F_T} = \frac{\rho V^2 L^2}{\tau V L} = \frac{VL}{\nu} \tag{2}$$

According to the principles of dynamic similarity was determined the scale of Manning resistance coefficient (n_R) and the scale of articles diameter (d_R), as it follows:

$$n_R = \frac{n_P}{n_M} = \frac{(R^{1/6}/C)_P}{(R^{1/6}/C)_M} = \frac{R_R^{1/6}}{C_R} \tag{3}$$

$$d_R^{1/6} = \frac{R_R^{2/3}}{L_R^{1/2}} = L_R^{\ 1/6} \tag{4}$$

2. Investigation Results

2.1 Diverse Tunnel

Diverse tunnel has a length L_P=40 m (L_M=1.0 m), diameter d_P=4.0 m (d_M=0.1 m) and maximum capacity of Q_P=120 m^3/s (Q_M=0.01185 m^3/s). The intake structure has a length of 4.7 m (0.1175 m) and height of 11.0 m (0.275 m). Upstream cofferdam is high 11.0 m (0.275 m). The investigation results of the diverse tunnel can be summarized as it follows. The intake structure of the diverse tunnel is well located. Maximum designed capacity of Q_{100}=120 m^3/s, that is flood with return period of 100 years, can be evacuated at reservoir water level elevation of 330.00 m a.s.l, which means that the designed crest elevation of the upstream cofferdam has to be increased to 331.00 m a.s.l. Regarding diverse tunnel capacity it is recommended to consider the discharge decrease from 120 m^3/s to 100 m^3/s. This means that the crest elevation of the upstream cofferdam can be decreased to elevation of 327.70 m a.s.l. This will certainly lead to financial benefits. The evacuation of Q_{100} was investigated also on the movable-bed model. The objective was to determine the quality and the quantity of the river bed material transport. Maximum erosion of 6.6 m is observed just downstream of the output structure. Maximum deposit of 1.4 m is observed downstream in the river bed on distance ≈25 m of the output structure.

2.2 Bottom Outlet

Bottom outlet is with diameter d_P=3.5 m (d_M=0.0875 m), gate dimensions a/b=3.0/2.4 m (a_M/b_M=0.075/0.06 m) and with the designed capacity of Q_P=136 m^3/s (Q_M=0.0134 m^3/s). The length of the intake structure is L_P=10.0 m (L_M=0.25 m) and its height is 4.0 m (0.1 m). The investigation of the bottom outlet was performed on normal water level in the reservoir that is 357.30 m a.s.l. and with different gate openings: 100%, 75%, 50% and 25%. Bottom outlet capacity is 158 m^3/s when the gates are 100% open, which is 14% increase of the designed capacity. This is partly due to the overestimated minor losses and partly due to the registered negative pressure behind the gates. To avoid the negative pressure the diameter of the aeration pipe has to be increased from d=800 mm to d=900 mm. The maximum pressure of 289.5 kPa is registered on the intake structure. In the section of aeration pipe connection the pressure pulsations are from 11.8 kPa to −22.0 kPa. In the section of connection, the free flow jet impacts the outside wall of the spillway tunnel where the maximum pressure of 93.4 kPa has been registered. On the movable-bed model has been observed maximum erosion of 7.8 m and maximum deposit of 1.1 m downstream of the output structure.

2.3 Shaft Spillway

The most extensive investigations were those of shaft spillway, spillway tunnel and its output portal. The length of the spillway tunnel is L_P=280 m (L_M=7.0 m) and its diameter is d_P=9.4 m (d_M=0.235 m). The spillway tunnel downstream of bottom outlet connection is designed in horizontal curvature with radius of 55 m (1.375 m). Downstream of the horizontal band the tunnel ends with output structure in length of 25 m (0.625 m). The diameter of the shaft is D_P=22.0 m (D_M=0.55 m). The length of vertical shaft is 20 m (0.5 m), and the radius of vertical band is R_P=14,1 m (R_M=0.352 m). For the designed notch geometry of the shaft spillway the maximum flood 1200 m^3/s is evacuated at the water level elevation in the reservoir of 362.50 m a.s.l. that is 0.2 m higher then the designed maximum water level elevation of 362.30 m a.s.l. The flow regime is transitional and the phenomenon of "water shutter" appears up to the elevation of 360.80 m a.s.l. The flow pattern around shaft spillway is circular and the mean velocities are not normal to the spillway sections. To overcome the observed inconveniences some correction of the notch geometry on the model was constructed. In this case the capacity of shaft spillway was increased and the flow pattern in the reservoir has rapidly improved as well. In this case the maximum flood of 1200 m^3/s is evacuated at the designed water level elevation 362.30 m a.s.l. The circulation around the shaft is symmetric and the approach velocities are normal to the spillway sections. The established discharge curves in both cases are presented in Figure 1. The spillway coefficient has the maximum value 0.425 at discharge 1130 m^3/s. The increase of the discharge leads to decrease of the spillway coefficient. In Figure 2 is shown graphically the function C_P=f(Q).

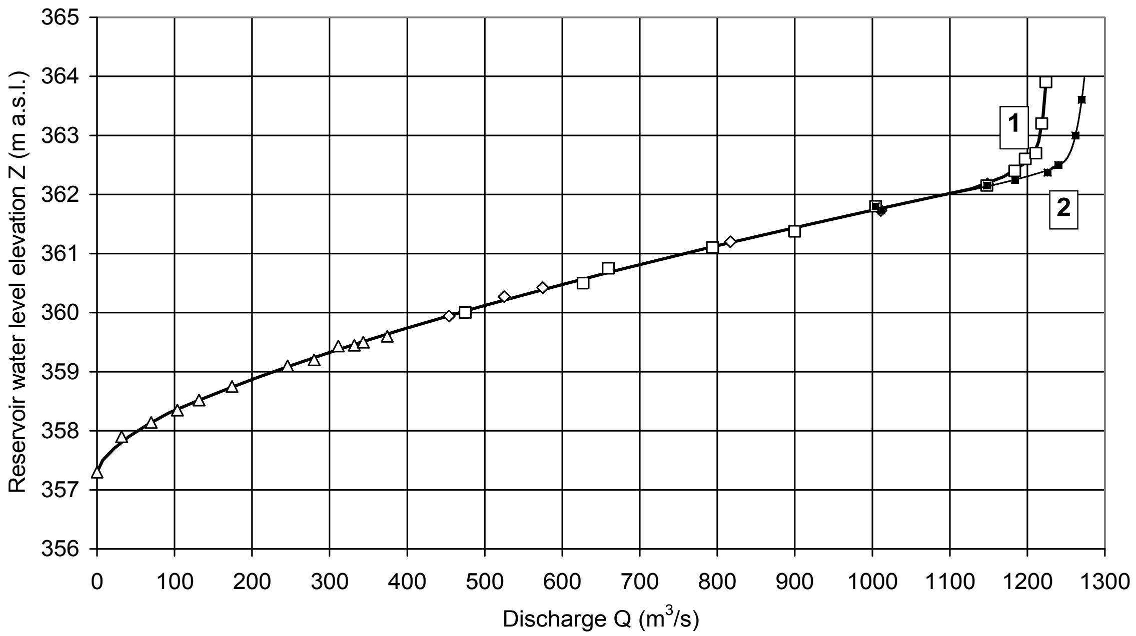

Figure 1 Discharge curves of the shaft spillway: (1) designed notch geometry, (2) corrected notch geometry

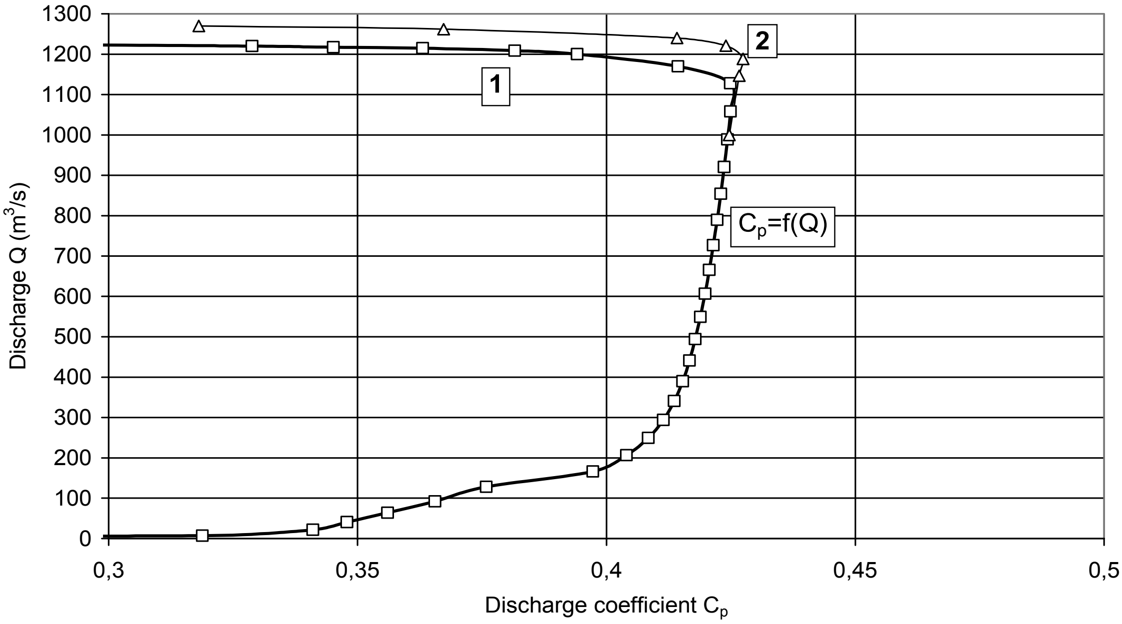

Figure 2 Definition of spillway coefficient: (1) designed notch geometry, (2) corrected notch geometry

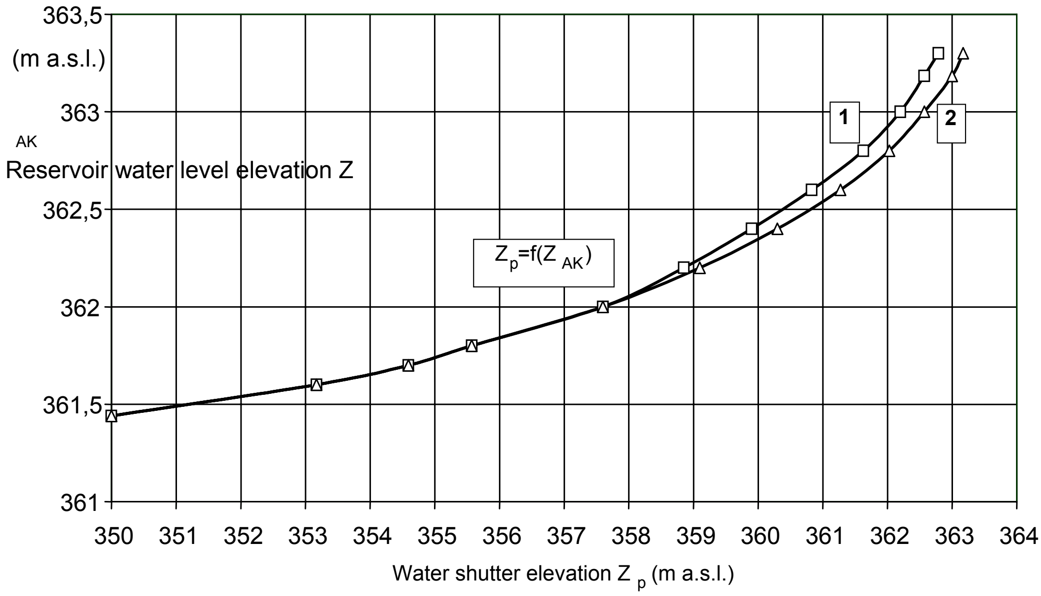

Figure 3 Definition of "water shutter" elevation Z_P as a function of reservoir water level elevation Z_{AK}: (1) designed notch geometry, (2) corrected notch geometry

The phenomenon of “water shutter” appears up to the elevation 359.70 m a.s.l., which is 1.1 m lower than in model case of designed notch geometry. According to the established discharge curve the flow regime for discharge 1200 m^3/s is free surface. On the basis of the model observations and measurements have been constructed two graphs that make possible the definition of the “water shutter” elevation Z_P as a function of the reservoir water level elevation Z_{AK}, Figure 3, and as a function of discharge Q, Figure 4, in both cases: (1) designed notch geometry and (2) corrected notch geometry.

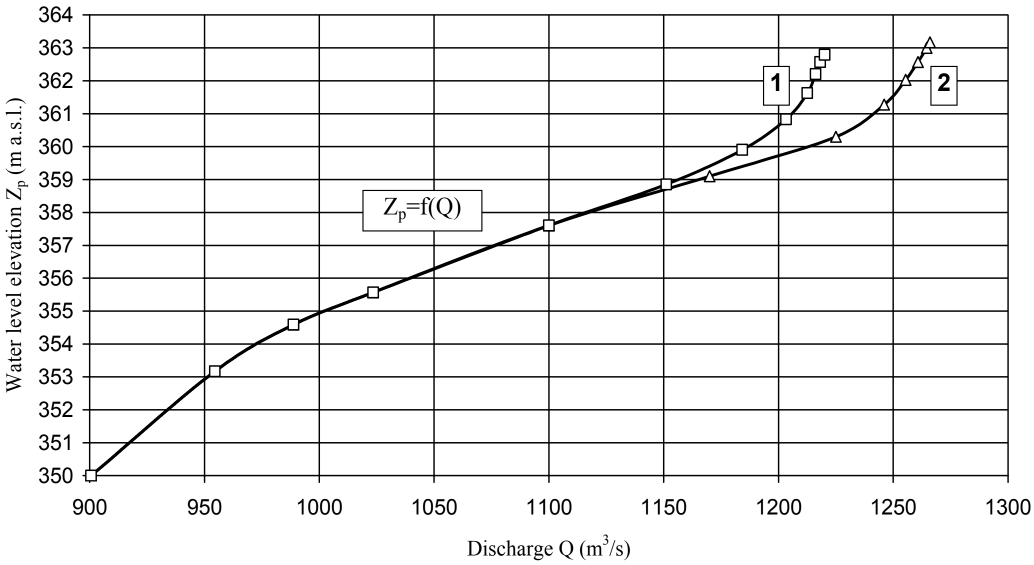

Figure 4 Definition of “water shutter” elevation Z_P as a function of spillway discharge Q: (1) designed notch geometry, (2) corrected notch geometry

The flow in spillway tunnel is free surface and very aerated. The maximum fill of the tunnel is ≈0,7D. Maximum pressure of 271.0 kPa has been measured on the outside wall of the bend. The jet that is leaving the output portal of the tunnel has a great kinetic energy according to the measured velocities of 22 m/s. The left side of the river is attacked on distance ≈100 m. The erosion hole has the following geometry: deepness 11.34 m, width ≈28 m and length ≈180 m. The deposits of 2.54 m and 1.1 m are observed upstream to the tail race and downstream in the Treska River channel, respectively.

The observed and measured flow regime in Treska River on the fixed-bed model is extremely turbulent and without establishment of water level in the river cross sections. The observed flow regime on the movable-bed model is much different. The erosion processes impact the transformation of kinetic energy into potential one, which was manifested with greater depths, low velocities and with establishment of horizontal water level in the river cross sections downstream of the output structure.

3. Conclusions

Hydraulic model investigations of the evacuation structures of dam “St. Petka” were performed to achieve the following objectives: (i) to define the capacities of diverse tunnel, bottom outlet and shaft spillway; (ii) to define the regime flow in diverse tunnel, bottom outlet, spillway tunnel and river channel; (iii) to define the flow patterns in the reservoir around the intake structures; (iv) to define the backwater impact of the water level in downstream reservoir “MATKA”; (v) to define qualitatively and quantitatively the erosion in the river channel downstream of the output structure of the spillway tunnel and (vi) to define the influence on water level in tail race due to flood evacuation.

The investigations were performed through measurements, observations, photos and video records. According to the results some improvements of the dimensions of the structures were proposed, such as: (i) need of increase the height of the upstream cofferdam for 1.0 m in case of design capacity of 120 m^3/s, or decrease of the capacity to 100 m^3/s that leads to a lower height of the upstream cofferdam for 2.3 m; (ii) need of increase the aeration pipe diameter of the bottom outlet from 800 mm to 900 mm to avoid the measured great negative pressure behind the closing gates; (ii) to improve the flow patterns around shaft spillway in the reservoir the aeration pipe has to be displaced from the shaft spillway section where it was planned in the preliminary design; (iii) to improve the flow patterns around shaft spillway and to increase its capacity the geometry of the right side notch has to be corrected according to the proposed measures. The investigations have approved the disposition of the structures and have proposed some improvements that have to be included in the main design. These improvements will certainly result with technical and financial benefits as well.

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Optical Surveying and Mapping in Experimental Hydraulics

Georg Schuster, Cedomil Josip Jugovic, Hans-Peter Nachtnebel

Abstract: By an investigation of the sediment deposition and formation of deltas in reservoirs at the IWHW, Vienna, a new optical surveying and mapping method, the Moiré projection, has been successfully applied. This simple method allows a quick 3-D survey of the model surface.
Keywords: Moirè methode, experimental hydraulics, delta

1. Introduction

In the experimental hydraulics one of the main tasks is to map the surface alterations by the simulated processes of deposition, erosion etc. The classical, mechanical methods are depth pointer gauges, woollen threads and photogrammetry. Mechanical methods have the disadvantage, that they are invasive and time consuming.

On the contrary, optical methods are non-invasive because functioning without disturbance of the water surface. They are also more efficient, because they allow at least partly some automatisation of the processes and less time for the measurement. Thus, also by longer acquisition process, if is even possible to operate without personnel. Decision, which method is the optimal one for a specific task, may be brought respective the following criteria:

- Measurement accuracy
- Time demand for measurement
- Effectiveness
- Quality of the surface to be measured (colour, texture, reflection, form, size)
- Existing means
- Measurement during the tests
- Measurement under water

2. Methods for optical surveying of solid surfaces in experimentalhydraulics

By the optical measurement methods, we may differ between single point measurements and image and image interpretation. Single point measurements are mostly very accurate, delivering numerical results, which may be visualised by expert programs, for example Surfer. These processes are often very slow and do not function simultaneously with the tests. Image interpretation is based on photography. Thus, perspective projection may result with distorted photos. Acquisition works mostly quickly. Nevertheless, a processing is needed. The result can be in form of a photo, graphics and/or 3-D co-ordinates.

2.1 Single-point measurement

The single-point measuring methods run automated. A laser ray measures the level of the predetermined position. It is mostly reasonable to place the measuring points as a raster and to add the borders and extreme points, if possible. Measuring facility is mostly computer driven. Simple systems project a point by a laser orthogonal to the measuring surface. A side positioned camera records the point position and thus is the level determined.

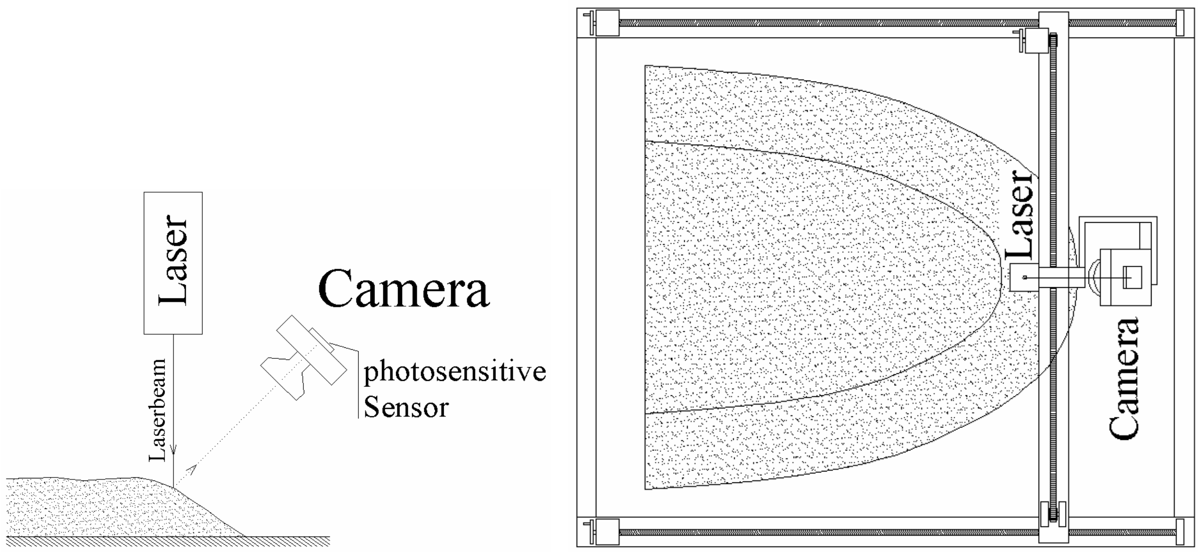


Figure 1 Schematic of a laser scanner

The hydraulic experimental field (HEF) at the IWHW is a multifunctional, computer driven measuring facility (Nachtnebel et. al, 1997). Thus, is it possible to record the level of the chosen position by a laser distance sensor, resulting with a file with 3-d co-ordinates of the measured points. They can be visualised for example by Surfer.

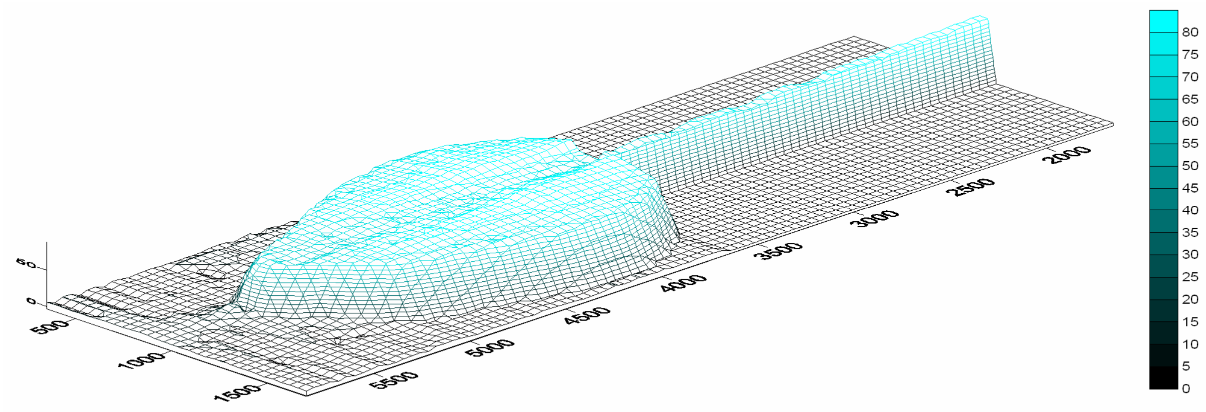


Figure 2 Result of the measuring system of the hydraulic engineering experimentation field

2.2 Image Interpretation

Image interpretation methods are photogrametry and Moirè topography. Here are the photos made that are later being processed. Nevertheless, by this process distortion appears. Firstly, the camera recording includes a central perspective projection. This leads to a spatial distortion. Through orthogonal exposure may the distortion be diminished, yet not eliminated. By orthogonal recording comes to displacements of measuring points positioned on different levels. This is the result of the difference between the perspective an orthographic projection. These distortions may be compensated easily by adequate methods.

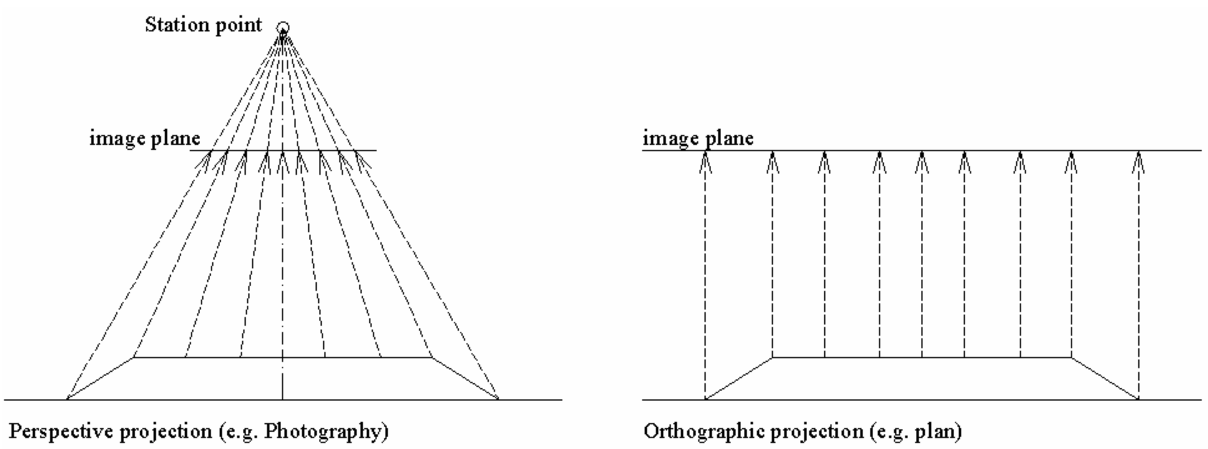


Figure 3 Projective rays of perspective and orthographic projection

2.2.1 Photogrammetry

By this method the measured object is being photographed from different angles. Nowadays, no special metric camera for this task is needed. The objective distortion may be calculated and corrected by the known control points. Yet, the camera position does not be known. Nevertheless, the control point’s positions have to be already known. Having just two distorted photos from different angles, this stereo pair of photos may be three-dimensional viewed and processed. By sufficient contrasts in recording the resulting processing follows, by the computer automatically.

The total processing (measuring control points, elimination of the distortion effects and stereoscopic processing) is very time consuming and does not function under water. Sophisticated software and skilled personnel are needed. But the results are good.

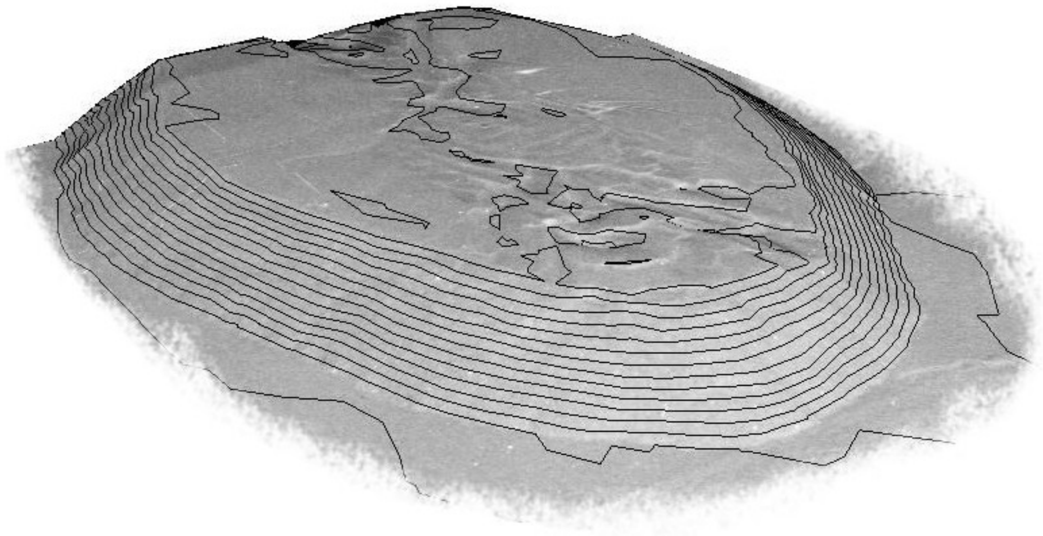


Figure 4 Result of the photogrammetry

2.2.2 Moiré Topography

Three-dimensional acquisition of surfaces in experimental hydraulics by means of Moiré fringe is relatively new. The word stems from 12th century and holds for, in Arabic culture known, material woven out of Mohair-wool. Through the production process it gets a, for Moiré-silk typical pattern (Neugebauer H., 1998). The same phenomena may be noticed by chequered material by TV or by folding of thin curtains etc.

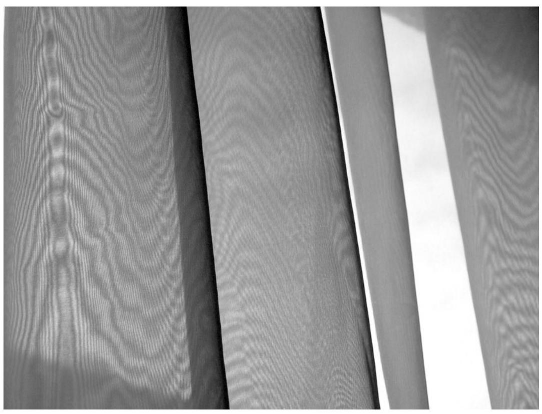


Figure 5 The moiré effect at thin curtains

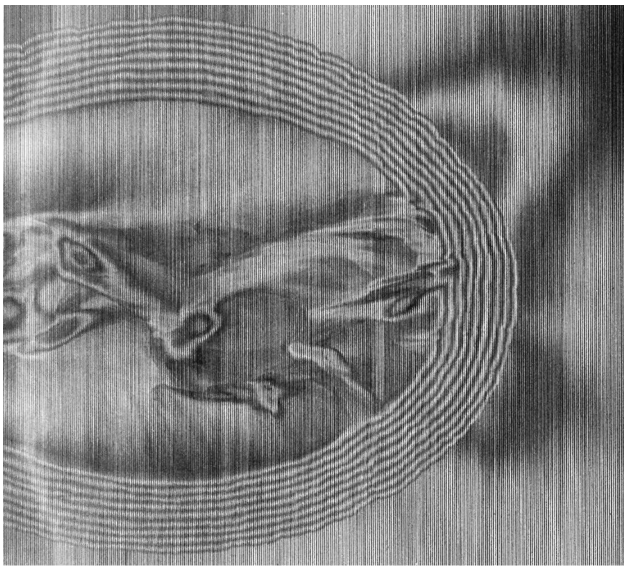
Lord Rayleigh gave scientific explanation in 1874. When two similar periodic structures overlay, it comes to a new, rougher stripped pattern. 50 years later, for the first time, a mirror surface has been optically proved by the Moiré method. In 2000, Müller (Müller et al., 2001) made the first experiments on scour by a bridge pier using Moirè projection.

The Moiré topography is an optical method of the image interpretation, with intention to get the information of the 3-D form of an object. In contrary to the photogrammetry, just one photo is needed. The possibilities of technical implementation are manifold. Here are always two similar periodic systems overlaid, that generate Moiré pattern. A correct description of this effect comes by Cloud (1998). In order to build out this striped pattern of the Moiré fringes, must the projection centres be replaced orthogonal to the lines. This may be done by arrangement of the illumination and of the camera, or through two projectors. By a suitable arrangement of the reference plane (a grid) the resulting Moiré fringes represent the contour lines. These lines are the cutting edge of the object surface and the surface parallel to the base level. This reference plane is, according to the system a grid, a so-called reference image or just a virtual surface, to which the projectors are directed in a specific way.

In order to realise this theoretical basis, Post (1994) introduced various methods how to do this. Either the grid is being projected onto the surface, or the grid is placed directly on the object and casting a shadow on it.

Shadow Moiré

The essence of this method is a flat grid. The object is being illuminated through the grid from the side. The observer sees through the grid the surface that is covered by a Moiré pattern.



Photograph 1 Photo of the shadow moiré effect

This pattern is formed while the dark and bright fields exchange. As bright, appear the sectors that are illuminated through the grid and may be visualised. As dark, appear the sectors that are either not illuminated, or are covered by the grid. Whether a field is dark or bright may depend, by a constant position of the observer, camera and light source only on the distance of the object surface from the grid (the reference plane).

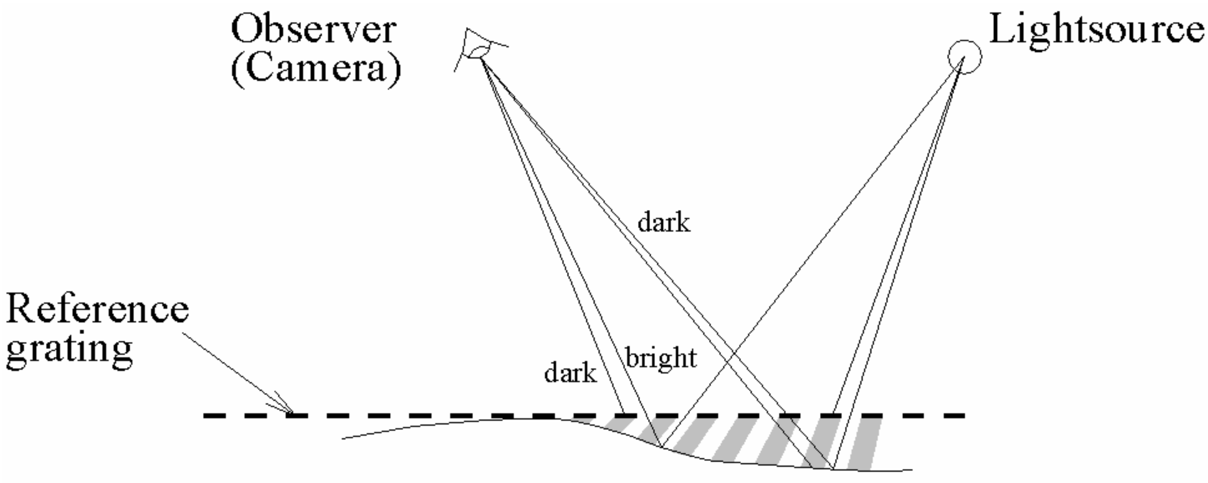


Figure 6 Principle of the shadow Moiré method

The Moiré fringes are actually layers. They are cutting edges between object surface and the planes parallel to the grid. The level difference between the neighbouring Δh_{iso} may be determined by the following equation:

$$\Delta h_{iso} = \frac{L \cdot S}{D} = \frac{S}{\tan \alpha}$$

Where L is the vertical distance between the light source and the camera from the grid, S the reference grating pitch and D is the distance between the camera and the light source. α is the angle between the observer and the light source.

Projection Moiré

The grid projection method is the fringe pattern by one or two projectors, projecting lines to the object. So, is the time consuming building up of the grid unnecessary. Yet, by the grid projection method, it is not possible to produce such a high resolution and such a contrast, as by the grid shadow method.

One Projector Moiré

By this method a stripped pattern is projected on the object. In order to generate the Moiré fringes the double-expose is needed: once without the object and secondly with the object. The two pictures are being superpositoned. The reference plane represents the grid and the layers are parallel with it. The geometrical relations are the same as by the grid shadow method and thus are also the elevation difference between the layers equal to those by the grid shadow method.

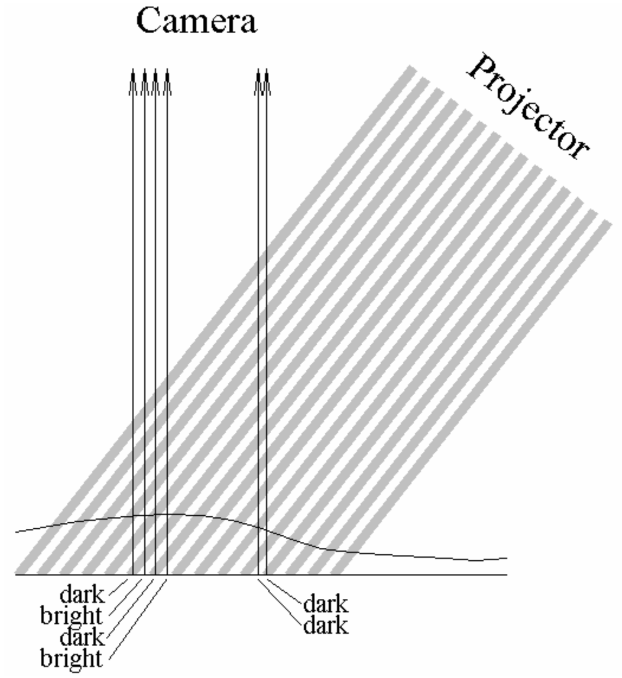
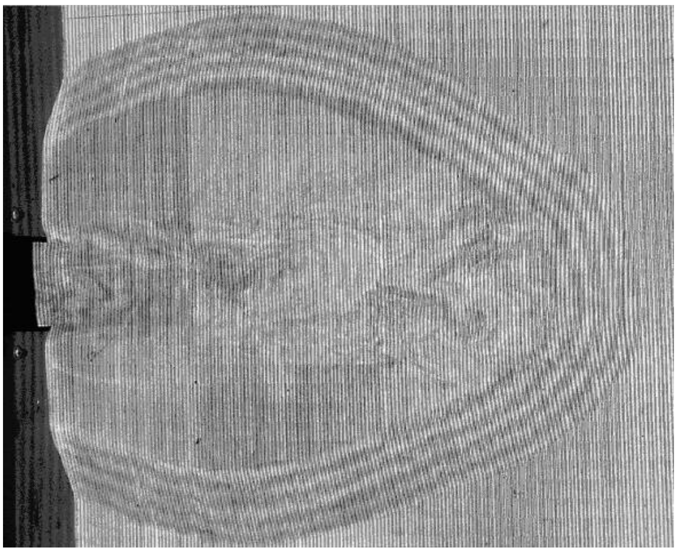


Figure 7 One projector - Schematic of double-exposure projection Moiré

After the superposition of the pictures with object and the one without (reference plane) result is the Moiré pattern.



Photograph 2 Photo of one projector Moiré

Two Projector Moiré

It is also possible to use two projectors, where the Moiré lines are, like by grid shadow method, immediately visible. By this method the two projectors are situated in such a way, that their projection axes form an isosceles triangle. The projectors are adjusted to a (virtual) reference plane in such a way, that their line pattern overlap exactly in this plane.

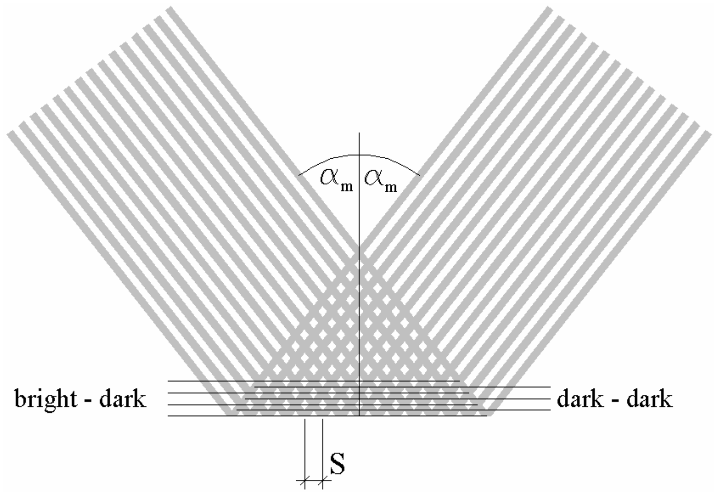


Figure 8 Principle of the two-projector Moiré method

2.2 Measurement under water

Measurement under water, during the test performance, has not been possible till now, by optical methods, although such a permanent monitoring would be of a great value. The methods of the single point measurement do not reach the object under water, because the laser ray reflects on the water surface. Besides, these methods are, due to their time consume, unsuitable for the current measurements. Photogrammetric processing is due to the light reflection not possible. Just by the help of the Moiré topography is a continuos monitoring of an ongoing current investigation under water surface possible. Yet, the consequences of the light refraction have to be considered.

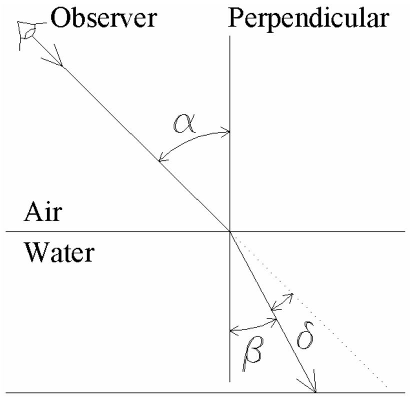


Figure 9. Principle of refraction

Through the light refraction on the water surface, the elevation difference between the Moiré lines is larger, what results with a smaller number of layers. The new distance between the Moiré lines may be easily calculated while taking into account refraction angle δ :

$$\Delta h_{iso} = \frac{S}{\tan(\alpha - \delta)}$$

The distortion effect caused by the refraction may be compensated parallel with the spatial perspective equalisation.

3. Summary

Optical methods of the surface acquisition in the experimental hydraulics are absolutely non-invasive and allow a quick recording of the surface. Although extrem time consuming, the single point measurements are in the same time very efficient, because they follow fully automated. Here, especially the Moiré topography should be stressed. By this simple method, it is possible to record very quickly 3-dimensionaly the whole surface of an object. The measurement may follow relatively independently from the surface and the result may be visualised immediately. Above all, the surface recording during the test performance without any disturbance of the tests is unique and promising. The performance may be “live” followed and documented.

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Paper №: **II.12**

Proposals of Measures for Enhancement of the Safety of Dams

Vlastimil Stara
Miroslav Špano

Abstract: Extreme floods which occurred in the Czech Republic in 1997 and 2002 evoked questions relating to the safety of dams built during the last century. Most of the dams have to be assessed mainly with respect to their safety and the capacity of their existing operational structures. The design of new facilities of dams, designs for reconstruction of the existing structures, and/or both are necessary to enable safe management of extreme water passage. This paper presents specific proposals of measures to enhance the safety of some Czech dams.
Keywords: dam, safety of dams, Znojmo Dam, hydraulic research

1. Introduction

Extreme floods had been affecting most of the territory of the Czech Republic from 1997 to 2002. Most Czech dams were exposed to the highest load so far. As a response to these events and according to a worldwide trend, a Guideline for Examination of the Safety of Dams during Floods [Metodický pokyn (1999)] was created by the Water Protection Section of the Ministry of the Environment of the Czech Republic. The guideline should amend the valid standard specification ČSN 73 6814 "Designing of dams." The purpose of the guideline is to ensure the required safety of Czech dams according to ICOLD recommendations [Metodický pokyn (1999)]. There are 86 dams registered in The World Register of dams and more than 23 thousand small earth dams (ponds) in the Czech Rep. The process of examining the existing dams according to the new guidelines has now been in progress.

2. Comparison of approaches to examination of the safety of dams

The present legislative ([Act No. 254/2001 of Coll. of Laws], also called “The Water Act”) divides hydraulic structures (HS) into four categories (I to IV) from the point of view of Technical and Safety Supervision. Probable life and property losses downstream of the dam play a decisive role for the division. The standard specification ČSN 73 6814 set the N-year interval of the so-called design flood and control flood, and some operational conditions. The maximum N-year interval does not exceed the value N = 1 000.

The Metodický pokyn (1999) guideline divides HS into three categories (A, B, C) by specific survey of losses in case of dam breach. The aspects examined are:

- hazard to life,
- economic losses (on HS and on downstream area),
- environment damage,
- social and economic consequences for the owner, the region, and the country.

Assignment of the required safeguard for each category expressed in N-year control flood is summarized in Table 1. The law category of HS is also included.

Table 1 Required safeguard of hydraulic structures (HS)

HS category (hazard indication)	The law category	Survey of losses	Required safeguard of HS	
			N [years]	P = 1/N
A (VERY HIGH)	I - II	Considerable life losses are expected	10 000	0.0001
	II	Life losses are not probable	2 000	0.0005
B (HIGH)	III – IV	Some life losses are expected	1 000	0.001
		Life losses are not probable	200	0.005
C (LOW)	IV	Losses prevail with third parties	100	0.01
		Owner’s losses prevail, other losses are negligible	50 - 20	0.02 - 0.05

As shown in the table, the present safeguard requirements are much higher compared with ČSN standards (max N = 1 000) especially for higher categories. A new special standard specification TNV 75 2935 “Assessment of dams during floods”, which follows the Metodický pokyn (1999) guideline, has been in force since August 2003.

2.1 Examination of the safety of dams

The following terms are being introduced to evaluate the safety of a dam:

- maximum safety storage elevation (MSE): determined for a specific dam as the highest water elevation in the reservoir which does not endanger the safety of the dam,
- maximum water-surface elevation (MWE): determined by routing the inflow hydrograph of an N-year flood through the reservoir surcharge volume.

A positive result has to satisfy the following formula:

$$MSE \leq MWE \qquad 2.1$$

Many dams in the Czech Rep. do not satisfy this condition. Therefore, searching for ways of correction is required. Redistribution of the reservoir volumes can sometimes suffice. But sometimes reconstruction of the existing and/or construction of new safety structures are necessary. Special roles are played by small earth dams (ponds). Their safety during floods was underestimated. A warning example is the rupture of several ponds on the river Lomnice in southern Bohemia. Flood from breached ponds caused significant losses in downstream towns (e.g. the Metelský Pond) [Jandora, J. (2005)].

3. Examples of dam safety enhancement on some specific dams

3.1 The Skalka Dam – implemented modifications

This 17 m high earth dam is located on the river Ohře close to the town of Cheb in Western Bohemia. It had been built between the years 1962 and 1964. Underestimation of the overtopping danger was expressed in the design of safety spillway with one section gated by a tainter gate. Within the framework of the prepared reconstruction of functional structures it was decided to build a new 17 m wide functional block containing a 7 m wide spillway section gated by a flap gate. The new spillway was proposed to safely handle the flow rate of 277 $m^3.s^{-1}$ (corresponding to Q_{100}) itself and 700 $m^3.s^{-1}$ (corresponding to $Q_{10\,000}$) combined with the old one [Broža, V., Pondělíček, V., Satrapa, L. (1999)].

3.2 The Morávka Dam – implemented modifications

The Morávka Dam is situated on the northern side of the Moravskoslezské Beskydy mountains on the river Morávka (catchment area of the river Odra). It had been built from 1961 to 1967. The earth dam is 39 m high. The upstream bituminous concrete face was damaged during floods occurred in September 1996 and July 1997. The analyses showed the necessity of the dam reconstruction. One of the aims of reconstruction was increasing the bottom outlets capacity. This was achieved by constructing two new bottom outlets situated in the right bank of the reservoir. For a full supply elevation in the reservoir the capacity of all bottom outlets has grown up from 24 $m^3.s^{-1}$ to 60 $m^3.s^{-1}$. The reconstruction had been realized between the years 1997 and 2000 [Morávka dam (2000)].

3.3 The Šance Dam – proposed modifications

An earth dam located on the river Ostravice (catchment area of the river Odra), serving as flood protection and as a drinking and industrial water supply, had been built from 1964 to 1969. The dam height is 56.8 m. The functional structures are composed of a safety spillway and a concurring chute with a design capacity of 70 $m^3.s^{-1}$, water intake tower with two bottom outlets DN 3000 and DN 2200 with a capacity of 113 $m^3.s^{-1}$ with full supply level [Švancara, J. (2002)], and a common stilling basin. The capacity of safety instrumentations was exceeded during the flood in 1997. The spillway, stilling basin and escapade channel were significantly damaged. Therefore, the owner, Povodí Odry, s. p., ordered a study [Švancara, J. (2002)] which deals with proposals of measures for increasing the safety of the dam against overtopping. Safe handling of the control flood corresponding with $Q_{10\,000} = 730\ m^3.s^{-1}$ (a flow rate of approx. 110 $m^3.s^{-1}$ was estimated during the 1997 flood) was required. Seven variants of proposed measures are discussed in detail in the study. Construction of a new spillway, reconstruction, and modification of the old one with respect to current structures are proposed. The choice of the specific proposal will be based on a technical and economic evaluation of the variants presented.

3.4 The Bystřička Dam – proposed modifications

The Bystřička Dam is the oldest dam in the catchment area of the river Morava. This 27.4 m high sandstone masonry dam, located on the river Bystřička near the town of Vsetín, had been built between the years 1908 and 1912. At present it serves mainly as a flood protection. Three pipes of bottom outlets (one inside the masonry and two in the diversion tunnel) have a capacity of 10.85 $m^3.s^{-1}$ for the full supply elevation. The dam almost overtopped during the 1997 flood. The cables anchoring the masonry into the ground were also damaged. A low capacity of the bottom outlets and the escapade channel ($Q = 30\ m^3.s^{-1}$) disabled operative management during the flood. The dam is classed in category A by the Metodický pokyn (1999) guideline. The whole dam reconstruction was chosen as the solution [Bilík, J., Rotschein, P. (2004)]. The masonry reconstruction and enlargement of the bottom outlet inside the dam (from DN 700/500 to DN 1200/1100) are already done. An outlet capacity of 11.7 $m^3.s^{-1}$ is available now (for the full supply elevation). The reconstruction of the safety spillway is planned for the years 2006-2007. It should ensure safety handling of 10 000-year flood with a peak flow rate of 310 $m^3.s^{-1}$ [Švancara, J., Torner, V. (2004)].

4. The Znojmo Dam – proposed modifications

This dam is located on the river Dyje close to the town of Znojmo in southern Moravia. It had been built from 1962 to 1966. This rock-filled dam with a watertight core is combined with a spillway block which comprises two sections of safety spillway gated by flap gates, two bottom outlets, two small hydropower stations (HPS), and water intakes.

The capacity of the safety spillway was exhausted during the 2002 flood. A flood peak of $Q = 379\ m^3.s^{-1}$ was estimated [Kadeřábková, J., Krejčí, V. (2004)]. Therefore, a study [Glac, F., Janků, O., Bubeník, M. (2003)] with variants of technical measures for safe management of extreme water passage was made. The study counts with a design flow rate of $Q = 610\ m^3.s^{-1}$ and with a 10 000-year control flood with a peak of $Q = 732\ m^3.s^{-1}$. To increase the spillway capacity of the Znojmo Dam the variant 2b was chosen as the best. The following modifications were proposed: to sink the existing spillway crest by 1.2 m and to replace the existing flap gates with tainter gates with flaps. The necessary modifications of the stilling basin were designed, including an increased height of basin walls. A breakwater was designed on the dam crest as well as a debris retaining system in front of the spillway ,see Figure 1.

With regard to the importance of the dam and the volume of the planned reconstruction, assessment on a hydraulic model was necessary. The client required:

- verification of the modified spillway capacity and the discharge coefficient evaluation,
- assessment of the spillway surface mainly with respect to the negative pressure occurring during extreme flow rates,
- proposal and design of modifications of the stilling basin at a design flow rate of $Q = 610\ m^3.s^{-1}$,
- specification of the designed modifications if necessary.

4.2 Hydraulic research

The hydraulic research was realised in the Laboratory of Hydraulic Research of the Faculty of Civil Engineering, Brno University of Technology, Veveří 95, Brno, Czech Republic.

A physical model of the Znojmo Dam spillway block was built respecting the rules of Froude’s similarity at a length scale of $M_l = 35$ (Figure 1).

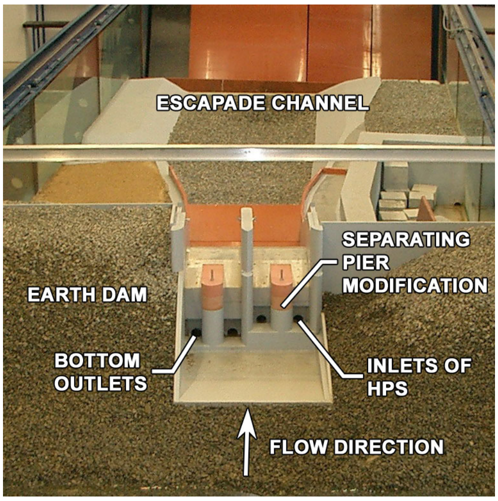

Figure 1 Downstream view of a model of the Znojmo dam spillway block

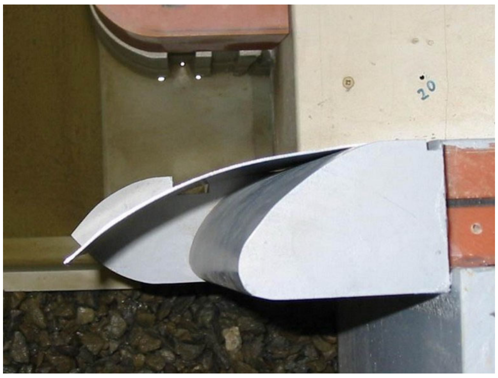

Figure 2 The stream leading plate set on top of the lower part of side pier

4.2.1 Tests of modified spillway capacity

There are 3.3 m wide separating piers with vertical grooves of cofferdam upstream of each section of the spillway (see Figure 1), whose modification considerably affects the spillway capacity. The results of the research led to the decision to sink the piers top elevation from 223.00 m a.s.l. to 221.60 m a.s.l. The influence of the tested piers shape modifications was negligible. The values of the discharge coefficient of the modified spillway surface are summarized in Table 2.

Table 2 Discharge coefficient **m** values as a function of discharge, valid for final modification

Q [$m^3.s^{-1}$]	51	143	282	606	725
m	**0.392**	**0.372**	**0.374**	**0.398**	**0.401**

4.2.2 Regulation of flow conditions

A model test proved that the contractions at the side piers have no effect on the spillway capacity because the side piers are ahead of the spillway surface. Wakes cause a water level increase of approximately (1.0 ÷ 1.1) m at side piers when the discharge is Q = 610 $m^3.s^{-1}$. An improvement of wake effects, i.e. decreasing the wave crest, was necessary due to the maximum gate opening during floods. Therefore several tests of flow improvement and wake elimination were performed by using wing-like steel elements attached to the front part of pier walls. The minimisation of plate dimensions was achieved by taking step-by-step measurements while using different plate shapes and sizes [Stara, V., Šulc, J. (2004)], see Figure 2.

The circular trailing end of the central pier was replaced by the rear part of the NACA profile [Stara, V., Šulc, J., Špano, M. (2005)] with the aim to eliminate the wave formed downstream of the pier, see Figure 3.

No values of pressure sufficiently negative to cause a cavitation effect on the spillway surface had been registered during the model measurements - even when the rate of flow reached Q = 732 $m^3.s^{-1}$.

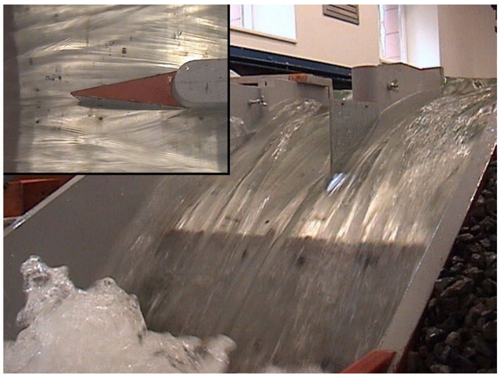

Figure 3 Deformation of level arising behind the end of middle pier and its elimination by the NACA profile when Q = 610 $m^3.s^{-1}$

4.2.3 The stilling basin modifications

The first model experiments have shown that the existent stilling basin of Znojmo Dam (length L = 33.0 m, depth h = 3.5 m) can no longer comply with the requirement of sufficient energy dissipation at a design rate of flow Q = 610 $m^3.s^{-1}$. It was necessary to propose its modification. The client required to keep the elevation of the stilling basin bottom at 207.65 m a.s.l., because the bottom is formed by natural rock. The baffle blocks at the bottom of the basin and the chute blocks at the end of the spillway surface with a combination of basin length were proposed and tested basing on preliminary computations

[Stara, V., Šulc, J. (2004)]. The criterion for finding the optimum stilling basin modification was the extent of deformations of the bottom of the channel downstream of the basin sill.

The use of chute blocks had a negligible effect on the total energy dissipation in the basin. The use of chute blocks considerably increased the aeration of the stream entering the space above the basin and spread a more uniform load on its bottom. The use of the chute blocks by alt. 3 was proved as more suitable (Figure 4). A 48.5 m long basin combined with five baffle blocks ensures low deformations of the channel bottom. Baffle blocks may be laid out in one or two rows depending on the chute blocks used, see Figure 4.

Due to the spray formed by water impinging on the baffle blocks in the basin a full concrete barrier about 1.2 m high was added on top of the basin walls. A heavy riprap 15 - 20 m long with concrete filling in an approximately 5 - 7 m long section was recommended as channel bottom protection.

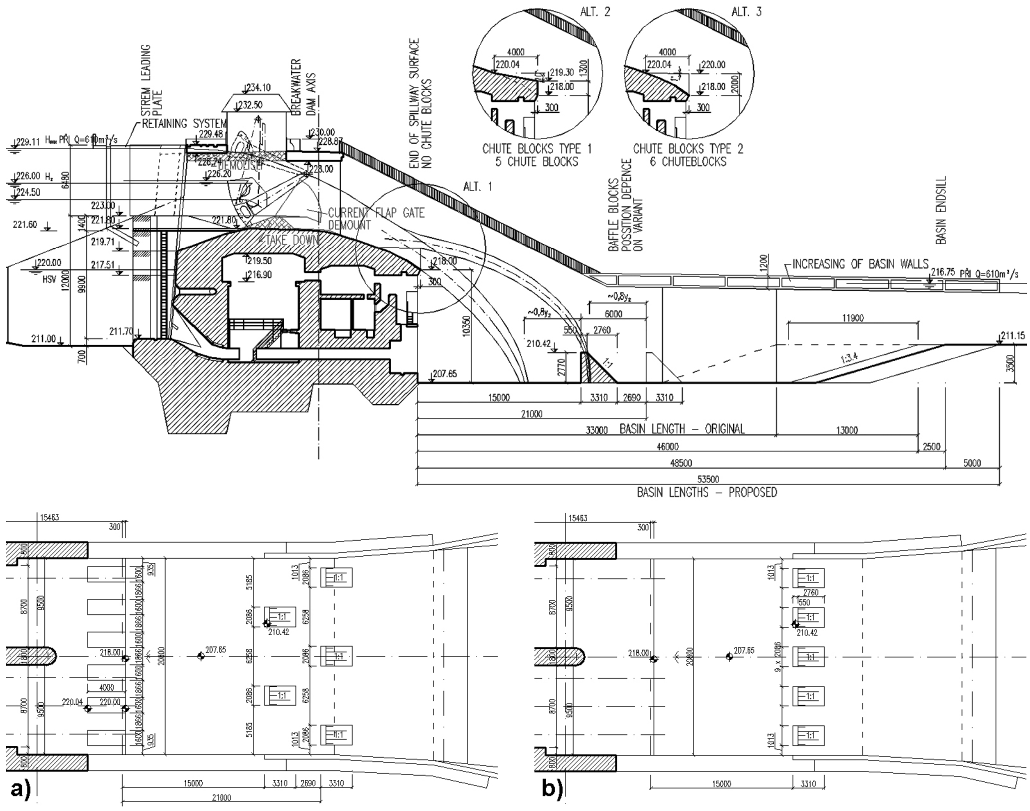

Figure 4 Vertical plane section of operational structure and the possible modifications and ground plan of optimal layout of baffle blocks: a) with chute blocks used; b) with no chute blocks used

4.2.4 Conclusion

The modified and tested safety spillway enables safe handling of the required rate of flow $Q = 610\ m^3.s^{-1}$. If the extreme water elevation corresponds with the breakwater crest level, the capacity of the spillway reaches up to $Q = 730\ m^3.s^{-1}$. Prolongation of the existing stilling basin combined with baffle blocks leads to sufficient energy dissipation and enables safe connection of the river Dyje downstream. Physical modelling was entire part of presented assessments and proposals of suitable modifications of the Znojmo Dam spillway block.

5. Summary

This paper shortly describes proposals for reconstructions of the functional structures of some hydraulic structures (HS) which led to increased safety of dams during floods with special regard to an overflow danger. Special attention was given to Znojmo Dam because the authors participated in the hydraulic research and evaluation of designed modifications. The capacity of the outlet and safety instrumentations of the dams is necessary for safe management of extreme water passage over the HS. The functionality and reliability of valves and gates are also very important. HS as a whole represent a complex organism which needs adequate servicing. The time of construction of large dams is over in the Czech Rep. However, it is still necessary to solve the questions of servicing, operating, modernizing, and enlarging the lifetime of dams. We will probably need to solve the question of safe demolition of some of the dams that are unsatisfactory from both operational and safety points of view.

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Paper №: **II.13**

Small Hydro-Power as a Source of Renewable Energy in the European Union

Roman Wichowski

Abstract: The development of renewable energy sources (RES) is a central aim of the EU's energy policy, because RES has an important role to play in reducing CO_2 emissions. Hydropower contributes one-fifth of the world's power generation and it provides the majority of supply in 55 countries. Its present role in electricity generation is therefore substantially greater than any other renewable technology. Hydropower is the largest and most mature source of renewable energy, with 728500 MW of installed capacity producing over 22% of the world's electricity, i.e. 2744 TWh in 2002 [7]. In the European Union, hydro energy represents about 90% of all EU renewable energy sources (RES) production and supplies about 14% of electricity demand. Small-scale hydro (<10 MW) has high efficiency and potentially low installation costs depending on the size of installation and location and currently represents only 3% of all hydro production. Decreases in head height, variable speed generators, reductions in the cost of equipment and environmental mitigation technologies will enhance the attractions of mini-hydropower. The object of this paper is to present an overview of Small Hydro-Power (SHP) in the European Union. More than 17400 small hydro-power plants were in operation in 1999 in 26 European countries with a total installed capacity of 12.5 GW. The total production was about 50 TWh per year, corresponding to 9.7% of the total hydropower production and 1.7% of the total electricity production. About 94% of the total hydro-power capacity in Western Europe is installed in 8 countries, i.e. Austria, France, Germany, Italy, Spain, Sweden, Switzerland and Norway. The EU White Paper [3] foresees a SHP production increase of 18 TWh in the EU countries between 1995 and 2010, which contributes by 4.7% of the global renewable energy production increase.

Keywords: European Union, hydropower, small-scale hydro, mini-hydropower, development potential.

1. Introduction

Hydropower is the largest source of renewable energy with 728500 MW of installed capacity, producing over 22% of the world's electricity, i.e. 2744 TWh in 2002 [5]. Most assessment consider that already existing plants exploit only about 19% of the world's total viable hydro-potential. In Western Europe hydropower contributed to the production of 568 TWh of electricity in 2002, or about 19% of EU electricity. Despite the present large hydropower capacity, there is still much room for further development for small hydro-power.

In 1999 more than 17400 Small Hydro-Power (SHP) were in operation in 26 European countries with a total installed capacity of 12.5 GW. The total production of SHP is about 50 TWh per year, corresponding to about 10% of the total hydropower production and 1.7% of the total electricity production. About 94% of the total capacity is installed in 8 European countries (Austria, France, Germany, Italy, Spain, Sweden, Switzerland and Norway) [3].

The European Union White Paper [1] foresees that a small hydro-power production increase of 18 TWh in EU countries between 1995 and 2010, which contributes by 4.7% of the global renewable energy production increase. In 1995 approximately 307 TWh of hydro energy was produced in the EU from an overall capacity of 92 GW. Small hydro plants, i.e. plants smaller than 10 MW accounted for 10% of installed capacity (9.3 GW) and produced 37 TWh of energy. An increase of 10% in installed capacity of large hydro (8500 MW) is likely by 2010 if one takes into account projects already planned. An additional installed capacity of 4500 MW of small hydro plants by 2010 is realistic contribution which could be achieved.

2. Renewable energy in the European Union

The European Union (EU) was one of the strongest proponents of the Kyoto Protocol, and assumed for 2008-2012 an emission reduction obligation of 8% below the 1990 level. The promotion of renewable energy has an important role to play in addressing the growing dependence on energy imports in Europe and in tackling climate change. Since 1997, the European Union has been working towards the ambitious target of 12% share of renewable energy in gross inland consumption by 2010. In 1997, the share of renewable energy was 5,4% and by 2001 it had reached 6%. The EU's Renewables Directive has been in place since 2001 [4]. In accordance with Directive 2001/77/EC, all Member States have adopted national targets for the share of electricity production from renewable energy sources. It aims to increase the share of electricity produced from renewable energy sources (RES) in the EU to 22.1% by 2010, thus helping the EU reach the RES target of overall consumption of 12% by 2010. With the enlargement of the European Union, the new Member States are required to adopt the RES-E Directive [4] by 1 May 2004. In the accession treaty, national indicative targets are set and the overall renewable electricity target for the enlarged Union will therefore be 21% of gross electricity consumption by 2010. National indicative RES-E targets for 2010 for all Member States of EU-25 are presented in Table 1 [2,4,12].

Table 1 National indicative RES-E targets for 2010 for Member States [2,4,12]

EU-25 country	RES-E 1997 [%]	RES-E 2010 [%]	EU-25 country	RES-E 1997 [%]	RES-E 2010 [%]
Austria	70.0	78.1	Sweden	49.1	60.0
Belgium	1.1	6.0	United Kingdom	1.7	10.0
Denmark	8.7	29.0	Czech Republic	3.8	8.0
Finland	24.7	31.5	Cyprus	0.05	6.0
France	15.0	21.0	Estonia	0.2	5.1
Germany	4.5	12.5	Hungary	0.7	3.6
Greece	8.6	20.1	Latvia	42.4	49.3
Ireland	3.6	13.2	Lithuania	3.3	7.0
Italy	16.0	25.0	Malta	0.0	5.0
Luxemburg	2.1	5.7	Poland	1.6	7.5
Netherlands	3.5	9.0	Slovakia	17.9	31.0
Portugal	38.5	39.0	Slovenia	29.9	33.6
Spain	19.9	29.4	**EU-25**	**12.9**	**21.0**

2. Small Hydro-Power Plants (SHP) in the European Union

2.1 A brief history of small hydro-power

Small hydro-power (SHP) has been exploited for centuries. First, the energy in falling water was used in mechanical form, e.g. watermills for milling grain. The invention of water turbine in 1827 led to the development of modern hydropower. In 1880s, hydropower turbines were first used to generate electricity for large scale use [7]. In Europe, turbines replaced the waterwheel almost completely by the end of 19th century. Small turbines were increasingly used throughout Europe and North America. With expansion and increasing access to transmission networks, power generation was concentrated in larger units benefiting from economies of scale. This resulted in a trend away from small hydropower systems to large hydropower installation between 1930s and the 1970s [7].

The oil crisis in 1973 re-kindled interest in the development of small hydropower resources. This led to a revival of the industry and with new turbine manufactures. Interest in developing hydropower systems again declined through the 1980s and early 1990s due to the low level of fuel prices and the subsequent dash for natural gas. More recently, liberalization of the electricity industry has contributed in some areas to the development of small hydropower generating capacity by independent power producers.

There is no international consensus on the definition of Small Hydropower (SHP). The upper power-limit varies from 2.5 MW to 25 MW, but 10 MW is becoming generally accepted by the European Commission and has been adopted by ESHA – The European Small-Hydro Association. Common definitions for small hydropower electric facilities are the following [3]:

- small hydropower: capacity of less than 10 MW;
- mini hydropower: capacity between 100 kW and 1 MW;
- micro hydropower: capacity below 100 kW.

The natural factors which affect SHP potential are the quantity of water flow and the height of the head. Flow roughly relates to average annual precipitation and the head depends on topography. The main requirement for a successful hydropower installation is an elevated head, either natural or artificial, from which water can be diverted through a pipe into a turbine coupled to a generator that converts the kinetic energy of falling water into electricity.

Small hydropower can generally be divided into three different categories depending on the type of head and the nature of the plant:

- high-head power plants are the most common and generally include a dam to store water at a higher elevation; these systems are commonly used in mountains areas;
- low-head hydroelectric plants generally use heads up to meters in elevation or simply function on run-of-river; low-head systems are typically built along rivers;
- supplemental hydropower systems are generating facilities where the hydropower is subordinate to other activities like irrigation, industrial processes, drinking water supply or wastewater disposal; electricity production is thus not the prime objective of the plant but often a useful by-product.

During the 20th century, the technology for harnessing water power developed rapidly and turbine efficiencies close to 100% were achieved. Typically, larger turbines have higher efficiencies. For example, efficiency is usually above 90% for turbines producing several hundreds kW or more, whereas the efficiency of a micro-hydropower turbine of 10 kW is likely to be in the order of 60% to 80%.

2.2 General overview of Small Hydro-Power Plants in the European Union

In the former EU-15 there operates about 14.000 SHP plants with an average size of 0.7 MW and there are 2770 SHP plants installed in the EU-10. The average plant size of these categories is 0.3 and 1.6 MW. The SHP plants situated in the former EU-15 are also the oldest, and surveyed countries have the highest share of young SHP plants, especially the candidate countries. The total installed capacity of SHP plants in the former EU-15 is around 9910 MW, i.e. 12 times more than in the EU-10. Electricity generation by SHP in the former EU-15 is nearly 17 times that of the EU-10, i.e. 39397 GW/year and 2329 GW/year, respectively [12].

Installed capacity and corresponding generation is expected to increase from 11% to 30% by the year 2010 and 2015 when compared with reference year 2002 in the former EU-15. About the same rate of increase will be kept for EU-10 (11-40%). Small hydro-power of the former EU-15 plays a far greater role in the electricity production than in surveyed countries. In the latter countries SHP plants contribute only 0.64-0.67% of the total electricity generation, less than half that of the former EU-15. Concerning the total hydropower production (excluding pumped storage plants), SHP shares are almost equal in the former EU-15 and EU-10 (11% and 13% respectively) [7,12]. SHP has many positive effects on nature and society and it replaces fossil fuel based power production.

The former EU-15 has an estimated economically feasible SHP potential of about 110000 GWh/year, or 110 TWh/year. The new member states (EU-10) have economically feasible potential of 775 GWh/year, or 7.8 TWh/year. More than 82% of all economically feasible potential has been exploited in the former EU-15. SHP potential exploitation rate is about 36% in the new EU-10 [8, 12].

Hydropower (large and small) contributes 17% to production of electrical energy in Europe, ranging from 99% in Norway, 76% in Switzerland, 65% in Austria, 51% in Sweden, down to 23% in France, 12% in Czech Republic, 6% in Poland, 4% in Germany, 3% and less in the UK and some other countries. Small hydropower accounts for approximately 7% of total hydro generation in Europe. The present capacity and production for 30 European countries are shown in Table 2 [10].

Table 2 Installed capacity and production of SHP plants (up to 10 MW) in 30 European countries [10]

No.	European country	Capacity [MW]	Energy [GWh]	Number	MW/Plant [MW]
1	*2*	*3*	*4*	*5*	*6*
	EU countries	**10634**	**41906**	**15560**	**0.68**
1	Austria	848	4246	1110	0.76
2	Belgium	95	385	39	2.44
3	Czech Republic	250	677	1136	0.22
4	Denmark	11	30	38	0.29
5	Finland	320	1280	225	1.42
6	France	1977	7100	1700	1.16
7	Germany	1502	6253	5625	0.27
8	Greece	48	160	17	2.82
9	Ireland	32	120	44	0.73
10	Italy	2209	8320	1668	1.32
11	Luxemburg	39	195	29	1.34
12	Netherlands	30	60	7	4.28
13	Poland	127	705	472	0.27
14	Portugal	280	1100	60	4.67
15	Slovakia	31	175	180	0.17
16	Slovenia	77	270	413	0.19
17	Spain	1548	5390	1056	1.47
18	Sweden	1050	4600	1615	0.65
19	United Kingdom	160	840	126	1.27
	Non-EU countries	**1983**	**8729**	**1903**	**1.04**
1	Croatia	30	120	13	2.31
2	Norway	941	4305	547	1.72
3	Romania	44	176	9	4.89
4	Switzerland	757	3300	1109	0.68
5	Turkey	138	500	67	2.06
6	6 other non EU countries	73	328	158	0.46
	Grand total – 30	**12617**	**50635**	**17463**	**0.72**

The total installed capacity of SHP stands at 12617 MW and production is estimated at 50635 GWh. Leading countries in Europe are Italy, France, Germany, Spain, Norway, Austria and Switzerland which combine 86% of small hydro-power capacity and production. The SHP production consists of around 17500 individual power plants with an average capacity of 0.7 MW and a production of 2.7 GWh per year. Average capacity varies widely between countries, from over 4 MW per plant in Portugal and Netherlands, 2.82 in Greece, 2.44 in Belgium, 1.0 – 1.5 for Italy, Spain, France, Finland and UK, down to 200-300 kW in others [10].

2.3 Prospects for Small Hydropower Plants (SHP)

Potential for Small-Hydropower technical capacity worldwide is estimated at 150-200 GW. World hydropower economic potential is estimated at about 7300 TWh per year, of which 32% has been developed, but only 5% (117 TWh) through small-scale sites [8].

In Asia (India, Nepal and China) almost 15% of the potential technical SHP capacity (60-80 GW) has been developed, while in South America only 7% of its potential (40-50 GW) has been realized. In the Pacific and in Africa, less than 5% of the potential (5-10 GW and 40-60 GW, respectively) has been developed. In North America and Europe, a larger share of the technical potential has already been developed than in developing countries [8].

In developed countries there are three key markets for small hydropower with substantial near-term potential: a) new installations; b) restoration and refurbishing of existing facilities and c) addition of SHP plants at dam built for flood control, irrigation and drinking water.

The greatest potential for SHP exists in new installations in developing countries. In rural areas of these countries, energy demand is often moderate and can be met by small or micro hydropower schemes. The plants are frequently operated in isolation or are connected to the local grids. The main competitor to SHP today in these countries is diesel generation. Refurbishment of old sites means the replacement of old equipment with more efficient turbines and/or generators, which would increase power production and/or reduce maintenance costs. The restoration and refurbishment of old sites is one of the most promising and cost-effective ways to increase hydropower generating capacity.

Small hydro-power installed capacity is estimated to grow between 1% and 6% per year over the next 20 years. Developing countries are likely to experience higher growth rates than other countries. The largest increase is expected to be in China. Rapid expansion with significant growth rates of 5% or above are expected on other of Asia, Latin America, the Middle East, and North and sub-Africa. Central and Eastern Europe are expected to increase their capacity at a lower growth rate of 2%, mainly through refurbishment and restoration of old sites. In Table 3 is presented the global growth rates and installed capacity of SHP by 2070 [8]. In Western Europe most of the region's hydroelectric resources have already been developed. In East-Central Europe, hydroelectricity already represents a substantial source of power in some countries such as Albania (96%), Croatia (59%) and Romania (37%). Despite a very large potential for expansion, these countries have found it difficult for a such projects.

Table 3 Global growth rates and installed capacity of SHP by 2020 [8].

Region	Present	Business-as-usual scenario		Accelerated development scenario	
	[MW]	Growth rate	2020 [MW]	Growth rate	2020 [MW]
China[1)]	9500	5%	25000	6%	30500
Europe	12500	2%	18500	4%	27500
South and Central America	3000	5%	8000	6%	9500
North America	5500	1%	6500	4%	12000
Rest of the World	1500	5%	4000	6%	5000
Total	**32000**	**3.2%**	**62000**	**5%**	**84500**

[1)] Higher and different capacity figures (especially for China) are communicated based on different SHP definitions [7]

2.4 Hydropower in Poland

Hydroelectric generation in Poland began at the end of the 19th century, and expanded during the early 20th century. However, the two world wars and subsequent conflicts destroyed existing infrastructure and dampened further development. Of the approximately 6.500 hydropower stations operating in Poland before World War II, only less than half of them survived the war. During the post World War II years, efforts and electrification, urbanization, and industrialization focused on construction of large, state-owned thermal power plants fueled by hard coal and brown coal. Consequently, the conventional hydroelectric power industry received little support from state authorities. In total, Poland currently has about 500 small hydro-power plants of total capacity about 170 MW. All the plants (small and large) are grouped in a few production companies and several distribution companies and are private property. Poland's has approximately 34500 MW of installed capacity (December 31, 2002). Hydropower contributes 2.400 MW or about 7% of the total capacity. Installed hydro capacity breaks down as follows:

- 1350 MW (4.0%) from pumped storage plants;
- 170 MW (0.5%) in small hydro and other renewable stations.

Several thousand sites exist in Poland for small, mini, and macro hydropower stations, while several dozen sites exist where medium sized hydro plants (up to several dozen megawatts) could be built.

Summary

1. Hydropower is a proven, well understood technology based on more than a century of experience. It schemes have lowest operating costs and longest plant lives. Many existing stations have been in operation for more than a half of century and are still operating efficiently.

2. Hydropower plants provide the most efficient energy conversion process. Modern plants can convert more than 95% of moving water's energy into electricity, while the best fossil-fuel plants are only about 60% efficient. Hydropower also has the highest energy payback ratio.
3. Hydropower is the largest and most mature source of renewable energy, with 728500 MW of installed capacity producing over 22% of the world's electricity, i.e. 2744 TWh in 2002. In Western Europe hydropower contributed to the production of 568 TWh of electricity in 2002, or about 19% of EU electricity.
4. In 1999 there were in operation 17400 SHP in 26 European countries with a total installed capacity of 12.5 GW. The total production was about 50 TWh per year, corresponding to about 10% of the total hydropower production and about 2% of the total electricity production. About 94% of the total capacity is installed in 8 countries, i.e. in Austria, France, Germany, Italy, Spain, Sweden, Switzerland and Norway.
5. Small hydro-power has a huge potential, which could allow it to make a significant contribution to future energy needs. SHP makes possible the production of renewable, decentralised, and indigenous energy, which corresponds perfectly to the objectives of both the Kyoto protocol, and the European Commission's White Book, since there are no CO_2 emissions.

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The Impact of Polish Pumped-Storage Plants on Operation of the National Power System on Working Days and Holidays

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Abstract: The analysis of current changes of power demand in the power system is conducted for characteristic days. The investigations are based on daily power balances of the power system made in the national dispatching center. The impact of pumped-storage plants on the load distribution in the national power system is investigated for two basic options: when the national power system operates without pumped-storage plants and when pumped-storage plants take part in the national power system, which is the real situation. The most recent investigations conducted at the Department of Hydro and Marine Engineering Gdańsk University of Technology, have also been focused on analysis of the effect of pumped-storage plants on the curve of daily generation of power and energy in the national power system. The analysis of current changes in demand for power in the national power system will be carried out for characteristic days selected from the period 1996-2003. The following studies have been carried out for the characteristic working and holidays:

- analysis of contribution of pumped-storage plants to increase the minimum base load of the national power system - N_{min} ,
- analysis of contribution of pumped-storage plants to reduce peak demand for power and intermediate load of the system in the power range between N_{max} and N_{min},
- analysis of contribution of pumped-storage plants to reduce intermediate generation of the system,
- analysis of contribution of pumped-storage plants to reduce minimum intermediate load,
- analysis of increase of the load coefficient in the system.

Keywords: Pumped storage plants; Power System; Peak and minimum demand; Load demand of the power system.

1. The effect of pumped-storage plants on the curve of daily generation of power and energy in the national power system

Data found based on readings on the hours throughout the day and two half-hour readings during the evening peak (and every 15 minutes since 2003) extend on the following parameters of the national power system:

- country generation,
- pumping (in all pumped-storage plants simultaneously),
- country generation not including power consumption for pumping,
- generation of hydro power plants of the system (6 pumped-storage plants of: Żarnowiec - 716 MW, Porąbka-Żar - 500 MW, Żydowo - 152 MW, Solina - 138 MW, Czorsztyn - since 1997 – 92MW, Dychów - 80 MW and two water- plants of: Włocławek - 160 MW, Rożnów - 50 MW),
- peak generation of all hydro power plants of the system,
- country generation not including generation of pumped-storage plants,
- normal demand not including power consumption for pumping,
- total generation of thermal power stations.

The results contain:

- total power generation [MW] from all national pumped-storage plants,
- total country power generation minus generation from pumped-storage plants [MW],
- country generation of active energy for half-hour intervals (and every 15 minutes since 2003) throughout the day [GWh],
- country generation of active energy minus generation from pumped-storage plants for half-hour intervals of the day [GWh],
- energy consumption for pumping for half-hour intervals of the day [GWh].

These data form a basis for subsequent calculations within the subject of investigations. Table 1 gathers results of calculations for an exemplary holiday - November 11, 2003 (Independence Day). The calculations were carried out for two main operational configurations of the national power system:

- national power system not equipped with pumped-storage plants (column 3 in Table 1), and
- national power system operating with pumped-storage plants (column 4 in Table).

The results of calculations divided into 9 groups separated by horizontal lines are gathered in Table 1. In the first group (rows 1÷3), there are values of generation in the national power system, with and without energy consumption for pumping, which enables the determination of percentage increase of generation in the national power system due to this consumption.

In the second group (rows 4÷10), values of normal peak demand P_{max} and minimum demand P_{min} (corresponding exactly to the frequency of 50,000 Hz) are gathered, based on which the percentage rates of P_{min} / P_{max} were determined (rows 7 and 8). Also the percentage share is evaluated of maximum power consumption for pumping (loading night lows) $P_{pump.max}$

referred to the minimum demand P_{min} that could take place if not for the pumping (row 10). In group 3 (rows 11÷13), the increase of minimum load in the national power system (minimum country generation) N_{min} owing to pumping in pumped-storage plants is determined.

Table 1 The effect of 6 Polish pumped-storage plants (PSP) on the load distribution in the national power system on November 11, 2003 (Independence Day)

	Parameter of the national power system	Unit	System		Remarks
			without PSP	with PSP	
	1	2	3	4	5
1	Country energy	[GWh]	418,50	425,17	
2	Energy consumption for pumping	[GWh]	-	6,67	
3	increase of energy consumption	[%]	-	1,6	
4	Maximum demand Pmax	[MW]	18295	18295	
5	Minimum demand Pmin	[MW]	13547	14510	
6	Pmax-Pmin	[MW]	4748	3785	
7	Pmin/Pmax	[%]	74,0	79,3	
8	Increase Pmin/Pmax	[%]	-	5,3	
9	Loading night lows (Maximum power consumption) Ppum.max	[MW]	-	1045	
10	Increase of Ppum.max/Pmin	[%]	-	7,7	
11	Minimum load Nmin	[MW]	14999	15768	
12	Increase of Nmin	[MW]	-	769	
13	Increase of Nmin	[%]	-	5,1	
14	Peak load Nmax	[MW]	20 513	20 513	
15	Nmin/Nmax	[%]	73,1	76,9	
16	Increase of Nmin/Nmax	[%]	-	3,7	
17	Nmax without PSP	[MW]	20513	19560	
18	Reduction of evening peak	[MW]	-	953	
19	Reduction of evening peak	[%]	-	4,6	
20	Intermediate load	[MW]	5514	3792	*
21	Intermediate load	[% of peak]	26,9	18,5	
22	Reduction of intermediate load	[MW]	-	1722	
23	Reduction of intermediate load	[%of peak]	-	8,4	
24	Relative reduction of intermediate load	[%]	-	31,2	
25	Basic energy	[GWh]	359,98	378,43	
26	Intermediate energy	[GWh]	58,52	46,74	
27	Share of intermediate energy in the country energy	[%]	14,0	11,0	
28	Reduction of intermediate energy	[%]	-	3,0	
29	Daily-average load Nav	[MW]	17437	17715	
30	Load coefficient m=Nav/Nmax	[%]	85,0	86,4	
31	Increase of load coefficient “m”	[%]	-	1,4	
32	Minimum intermediate load				
33	Nmin.aftern.l. – Nmin.nigh.l.	[MW]	2 572	1 803	
34	% of peal load	[%]	12,5	8,8	
35	Reduction	[%]	-	3,7	

* generation and pumping in pumped-storage plants (PSP) subtracted

The increase of minimum load in the national power system (minimum country generation) N_{min} referred to the peak load N_{max} is determined in group 4 (rows 14÷16).

The reduction of evening peak load owing to generation from pumped-storage plants is determined in group 5 (rows 17÷19).

In group 6 (rows 20÷24), the reduction of intermediate load, being a difference between the maximum and minimum power in the national power system is evaluated.

Group 7 (rows 25÷28), shows results of energy generation in the base of the system (below N_{min}) and in intermediate generation, that is between N_{max} and N_{min} .

In group 8 (rows 29÷31), values of the daily-averaged power and load coefficient for the system are determined.

In the final group 9 (rows 32÷35), the reduction of minimum intermediate load, that is a difference between the minimum load in the afternoon and night low load period ($N_{min.aftern.l.}$ - $N_{min.nig.l.}$) referred to the peak load is evaluated.

Based on the above data, daily graphs of country generation, country generation minus load consumption for pumping and country generation minus power generation from the pumped-storage power plants were elaborated. A sample graph for November 11, 2003, is shown in Figure 1. The picture illustrates selected results of calculations gathered in Table 1. Besides indicating (by arrows) the regions of pumping, the following quantities are indicated and described in Figure 1:

- percentage reduction of power generation in the system during the evening peak due to operation of pumped-storage plants,
- percentage increase of the minimum power generation in the night low due to pumping,
- percentage reduction of intermediate load (difference between the maximum and minimum load throughout the day) referred to the peak load,
- reduction of intermediate generation (generation between the daily minimum and maximum load) referred to the country generation, reduction of the minimum intermediate load (difference between the minimum load during the afternoon low-load period - $N_{min.aftern.l.}$ and minimum load during the night low - $N_{min.nig.l}$) referred to the peak load.

The conducted investigations revealed a number of interesting results and led to a number of conclusions, which will be formulated below (Wróblewski, 2003).

A general principle of loading pumped-storage plants in the national power system follows from the programmed function of these plants in the system. In a daily cycle they are loaded for generation in the period of peak demand for power (morning peak and evening peak) and for pumping in the period of minimum demand for power (night off-peak and relatively rare afternoon off-peak - practically only on Sundays and some holidays). The level of generation and pumping load and its duration depend on the season of the year, atmospheric conditions (temperature, cloudiness, rainfalls or snowfalls), and on the day of the week.

The distribution of load changes between working days, Saturdays, Sundays and holidays, as well as some other days neighboring with holidays (e.g. May 2, Christmas Eve, New Year's Eve). The distribution of load in pumped-storage plants on working days is not the same for all days. Mid-week days – Tuesdays, Wednesdays and Thursdays are different than Mondays and Fridays. Mid-week working days are characterized by the largest demand for power of all weekdays. The peak power usually takes place on Tuesdays or Thursdays.

The difference between Mondays or Fridays and other working days consists in some extension of the period of generation during the evening peak on Fridays, and delay in pumping during the Friday/Saturday night off-peak, and in some prolongation of pumping during the Sunday/Monday night off-peaks and delay in generation work on Monday mornings. Another difference is a deeper Sunday/Monday night off-peak as compared to other weekdays.

The analysis was also extended on variation of maximum power generation and maximum peak demand for power during morning and evening peaks of characteristics days over 12 months of both investigated years. The lowest values of power demand, and country generation, in both peaks occur during summer months: from June to August, the largest power demand is in winter: from November to February. Particular attention is due to the fact that during summer month's values of peak demand during the evening peak are below those of the morning peak.

2. Analysis of contribution of the pumped-storage plants to increase the minimum base load demand of the national power system

It follows from the conducted investigations that the pumped-storage plants considerably change the shape of the daily curve of power generation in the national power system. The power demand during night low-load periods increases. The minimum power demand P_{min} increases thanks to pumping operation of all pumped-storage plants in the country from 73.8% to 80.7% of the peak demand P_{max}, that is 6.9% of P_{max}. The considered increase oscillates during the analysed days between 1.8% (Holiday on November 1, 2003) and 10.6% P_{max} (on June 3, 1999 - Corpus Christi day) (Biliński, Wróblewski, 2001 and Wróblewski, Tararuj, 2005).

Values of maximum loading of the night low due to pumping range between 769 MW and 1577 MW (for 1627.8 MW of total pumping power of the 6 pumped-storage plants), with the average value equal to 1195 MW, which constitutes 8.7% of the minimum power demand N_{min}. The limits of the range are 5.1% and 12.3% of N_{min}, respectively. The discussed percentage increase of the minimum load in the national power system due to pumping in national pumped-storage plants is indicated and quantitatively described in Figure 1.

Owing to the increase of the minimum load N_{min}, the daily ratio of this load to the peak load N_{max} also increases. During the considered days, N_{max} changes between 12617 MW and 24890 MW (the latter value referring to December 3, 1998 - a 'peak' day in 1998). The percentage ratio N_{min}/N_{max} during the analysed days increases. The extreme values of the increase were 3.7% and 9.0% of N_{max}.

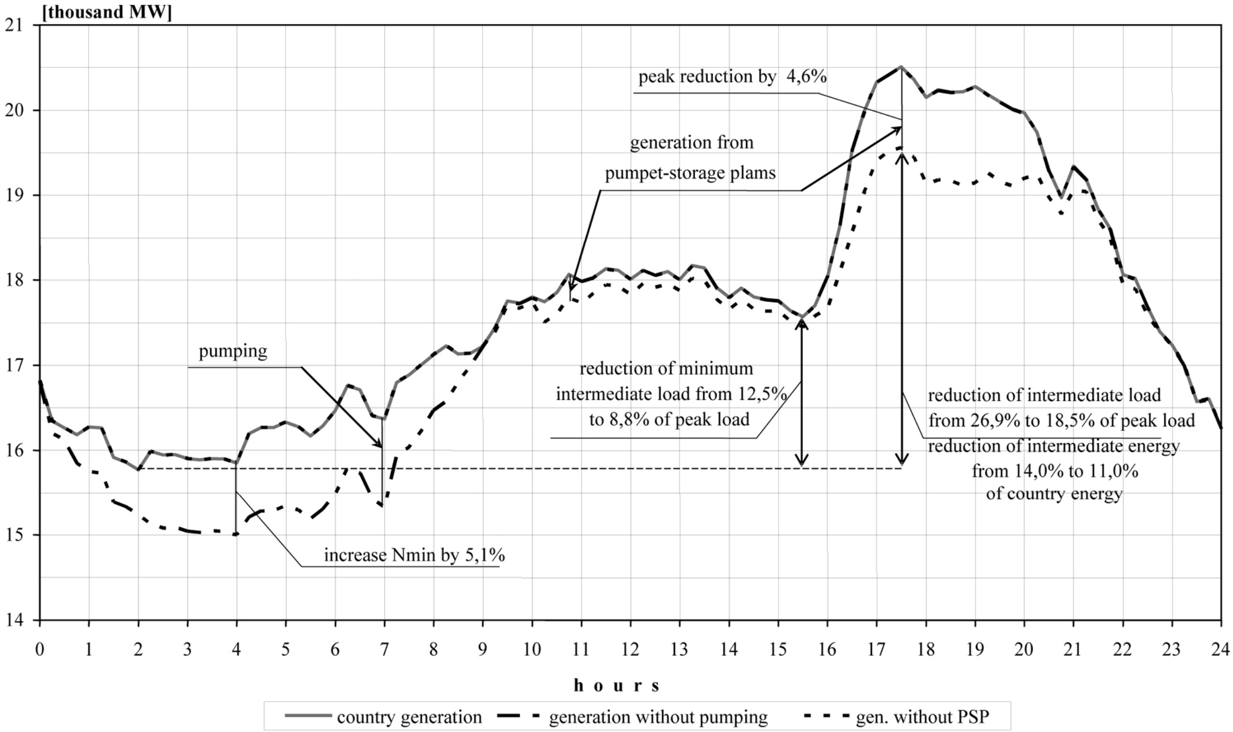

Figure 1 The effect of 6 national pumped-storage plants on the balance of power and energy in the national power system on November 11, 2003 (Independence Day)

3. Analysis of contribution of the pumped-storage plants to reduce the peak power demand and intermediate load of the system

For certain, pumped storage plants not only increase the demand during the night lows, but also reduce the peak demand. The reduction of the evening peak due to generation from pumped-storage plants ranges from 408 MW (August 18, 1998) to 1489 MW (April 4, 1996) (Wróblewski, 1999), which amounts to 2.6% do 9.5% of the maximum load N_{max} (April 20, 2003 – Sunday, Easter). The percentage value of the considered reduction is also indicated in Figure 1.

The increase of the minimum load and reduction of the peak causes that the range of load between N_{min} and N_{max}, called the intermediate load (Adam,Neill, 1984) is reduced twice. The intermediate load (N_{max} -N_{min}) is reduced from about 1400 MW (7.7% of the peak load N_{max}) down to 2866 MW (13.5% of N_{max}). The relative reduction intermediate load oscillates between 24.2% and 55.3% of the intermediate load of the national power system. The value of reduction of the intermediate load expressed in % is indicated in Figure 1.

4. Analysis of contribution of generation from the pumped-storage plants to intermediate generation of the national power system

Integration of the load of the national power system over 24 hours of the day in the limits between N_{min} and N_{max} enables the determination of intermediate generation for each day. Owing to the operation of pumped-storage plants, the amount of intermediate generation is considerably reduced. The percentage value of intermediate generation referred to the total daily generation for both configurations of operation of the system, with and without pumped-storage plants, is also indicated in the figure. Values of reduction of intermediate generation for the analyzed days oscillate between 3.0% and 8.0% of generation in the national power system.

5. Analysis of increase of the load coefficient in the system

The 24 hour-averaged load N_{av} of the system was calculated for each considered day. The operation of pumped-storage plants in the national power system acts to increase the average load of the system. The degree of equalization of the load throughout the day can be expressed by the so called load coefficient of the system m, calculated as a ratio of the average load N_{av} to the maximum load N_{max}

$$m = \frac{N_{av}}{N_{max}} 100\% \quad (1)$$

Hypothetically, in a practically impossible case of constant load over the day, the load coefficient m would be equal to 100% (the average load would equal the maximum load and the daily distribution of load would be a horizontal line). Owing to the operation of pumped-storage plants in the national system, the load coefficient increases on average from 88.4% to 90.0%, that is by 1.6% and maximal 2.7% N_{max}.

6. Minimum intermediate load during off-peak periods

For each considered day, a reduction of the minimum intermediate load, that is a difference between the minimum load during the afternoon low $N_{min.aftern.l.}$ and night low $N_{min.nig.l.}$ ($N_{min.aftern.l.}$ - $N_{min.nig.l.}$) referred to the peak load N_{max} was determined. The minimum off-peak intermediate load due to operation of pumped-storage plants decreases on average from 3000 MW (16.0% of the peak load) down to 1900 MW (9.5% of the peak), that is by 6.5% of N_{max}. The maximal observed reduction was 9.1% of the peak (N_{max}).

7. Increase of energy consumption in the national power system due to pumping

In order to fully illustrate the effect of pumped-storage plants on the operation of the system, it is also useful to determine the daily percentage increase of energy consumption for pumping related to the country generation. Considering numerous advantages for the system coming from the operation of pumped-storage plants discussed in the preceding sections, this seems to be the only disadvantage. The increase of energy consumption for pumping in all 6 pumped-storage plants amounts on average to 1.9%. For the analysed days, it changes between 1.0% and 2.7%.

Summary

Pumped-storage plants considerably reduce the intermediate load. The reduction of the evening peak due to generation from pumped-storage plants ranges from 400 MW to 1500 MW, which amounts to 2.6% do 9.5% of the maximum load $N_{max.}$ The intermediate load (N_{max} -N_{min}) is reduced from about 1400 MW (7.7% of the peak load N_{max}) down to 2870 MW (13.5% of N_{max}). The relative reduction intermediate load oscillates between 24.2% and 55.3% of the intermediate load of the national power system. Values of reduction of intermediate generation for the analyzed days oscillate between 3.0% and 8.0% of generation in the national power system.

The calculations are valuable as they concern real loads that occur in Polish power system.

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Theme III

Design and Construction Works

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Yachts Parameters for Marina Design

Dalibor Carević
Marko Paršić

Abstract The basic dimensions of power and sailing yachts are presented, as well as basic dimensions for two type of berths: Mediterranean ones and finger piers type (characteristicall for US and Australia). The idea was to collect different recommendations data, from different nautical tourism markets: US, Australian, British and Mediterranean, make compare, and give recommendation for Croatia. The other part of work deals with the wind effects on yachts. Wind parameters for wind calculation are defined in different standards. A discussion is presented on the state-of-the-art in marina design to accomplish these parameters from marina design experience in Mediterranean region and worldwide. The result tends to Croatian recommendation.

Fleet of nautical turism

Types

The basic types of boats which are commonly used in engineers practice are:

1. Power boats (M),
2. Sail boats (S),

and other types:

3. Catamarans,
4. House boats.

Design of marina is deriving from parameters of basic types of boats because they are common users of marina and for the other types can be predicted reduced capacity. Which parameters will be used depend of aim of marina and sort of fleet which will be primary user.

Basic boat parameters

PIANC data are presented 1976. from "International commission for sport and pleasure navigation") [1]. Here are also presented parameters from "Australian Standards" [2]. US recommendations come from industry cataloges of boats [3], [4].

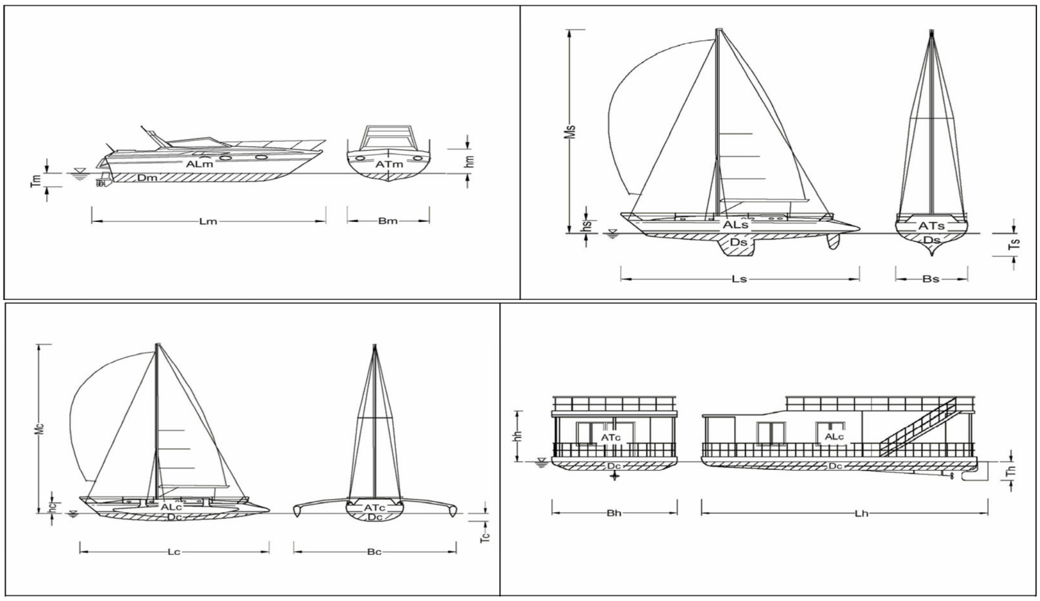

Figure 1 Definition sketch for dimensions of power and sail boat, catamaran and house boat. T[m]-draught; h[m]-stem heigh; L[m]-lenght; B[m]-width; D[m^3]-displacement; AL[m^2]-transverzal section area; AT[m^2]-longitudinal section area; m-power boat; s-sail boat.

All defined data have just orientation purpose in dimensioning of marinas and acquire dimensions depend of specific situations and projects.

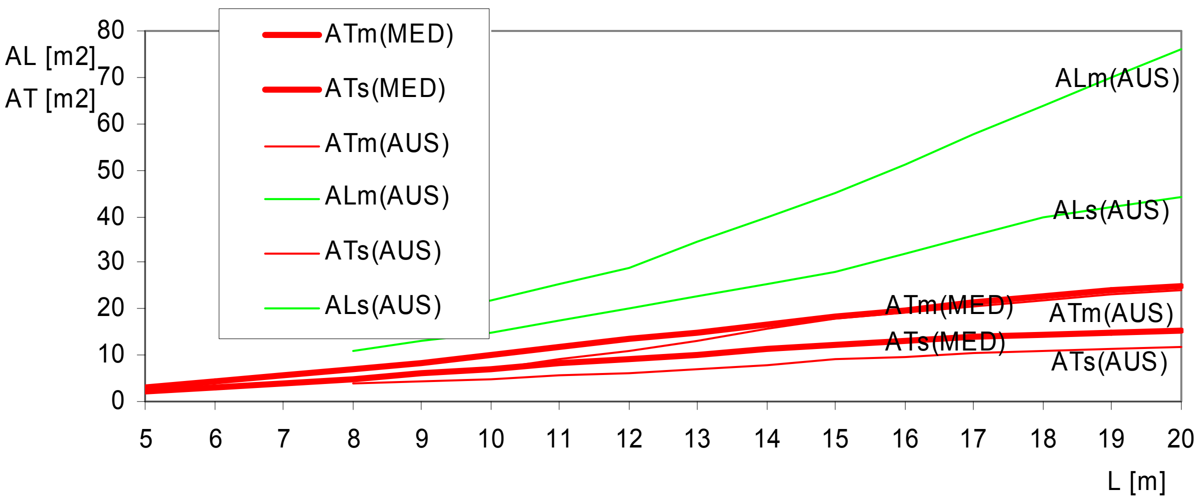

Figure 2 Hull cross sections in relation with length of boat; AL_m[m^2], AL_s[m^2], AT_m[m^2], AT_s[m^2]; m-power boat, s-sail boat; MED-Mediteranian, AUS-Australia.

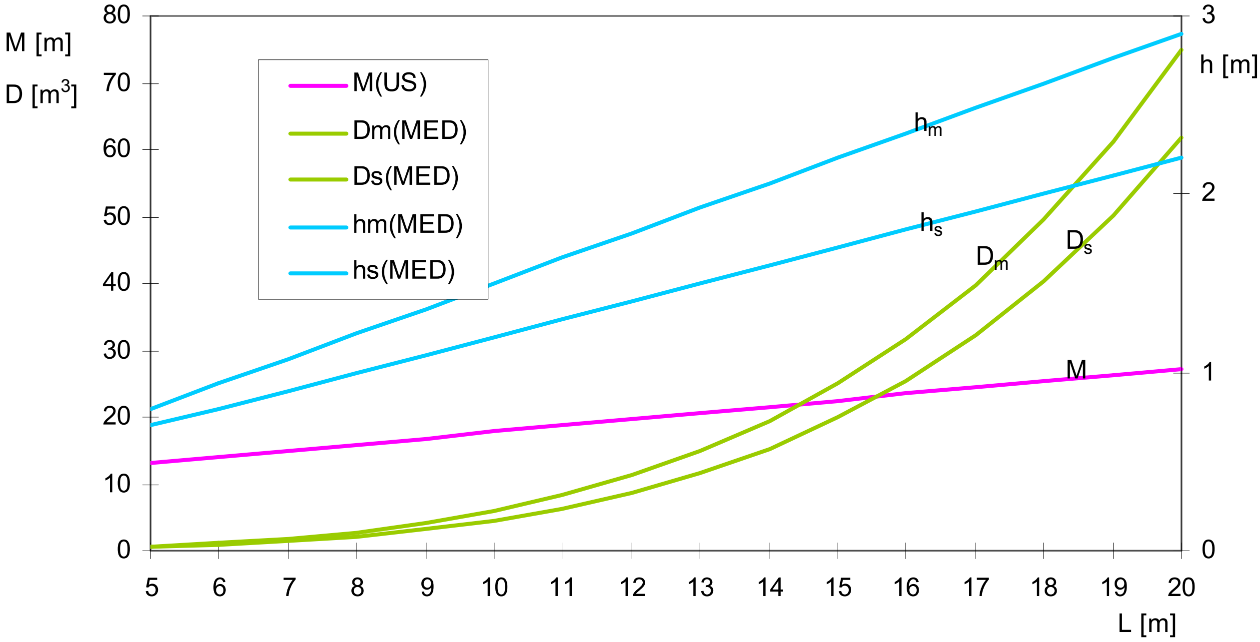

Figure 3 Displacement D [m^3], mast height M[m], stemt height h[m]; m-power boat, s-sail boat; MED-Mediteranian, US-United States.

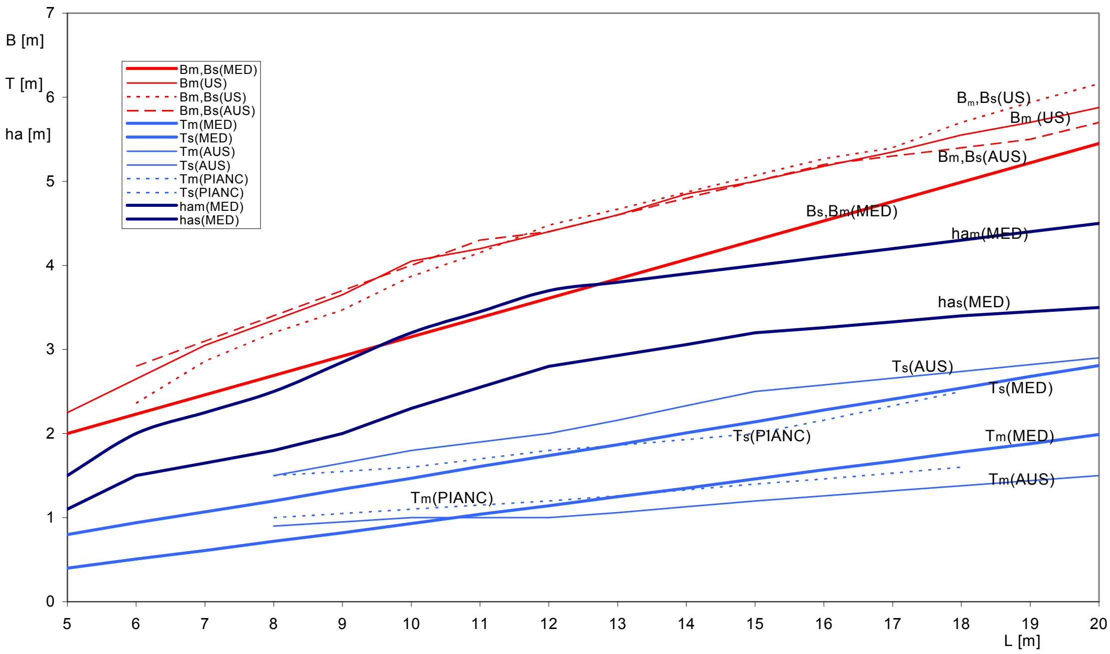

Figure 4 Hull geometry; B_m[m], B_s[m], T_m[m], T_s[m], ha_m[m], ha_s[m]; m-power boat, s-sail boat; MED- Mediteranian, US-United States, AUS-Australia.

Basic types of marinas

Basic berth types are:

1. Mediteranian type (med)
2. Finger piers type (f)

Which type will be selected depend on tradition of building but also on technical and economic aspects of project.

Generally finger type is more expensive variant of berth configuration because it requires more space but providing safetly maneuouvreing and mooring boats. On the other side Mediteranian type provides better usability of space and allowes possibilities of mooring greater boats longitudinal to the pier. Weakness of this type is slimness of berth space which desire greater agility of yachtman.

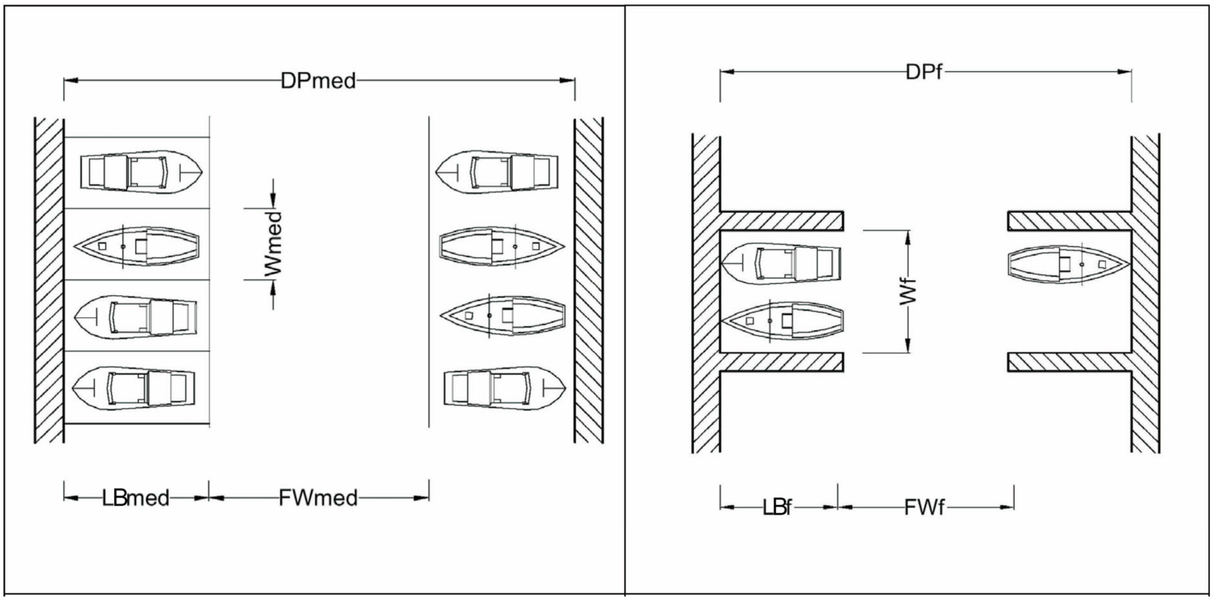

Figure 5 Definition sketch for Mediteranian type and finger type

DP	[m]	space between piers
W	[m]	berth width
LB	[m]	berth length
FW	[m]	fairway between berths
d	[m]	draught

med-mediteranian type; f-finger type

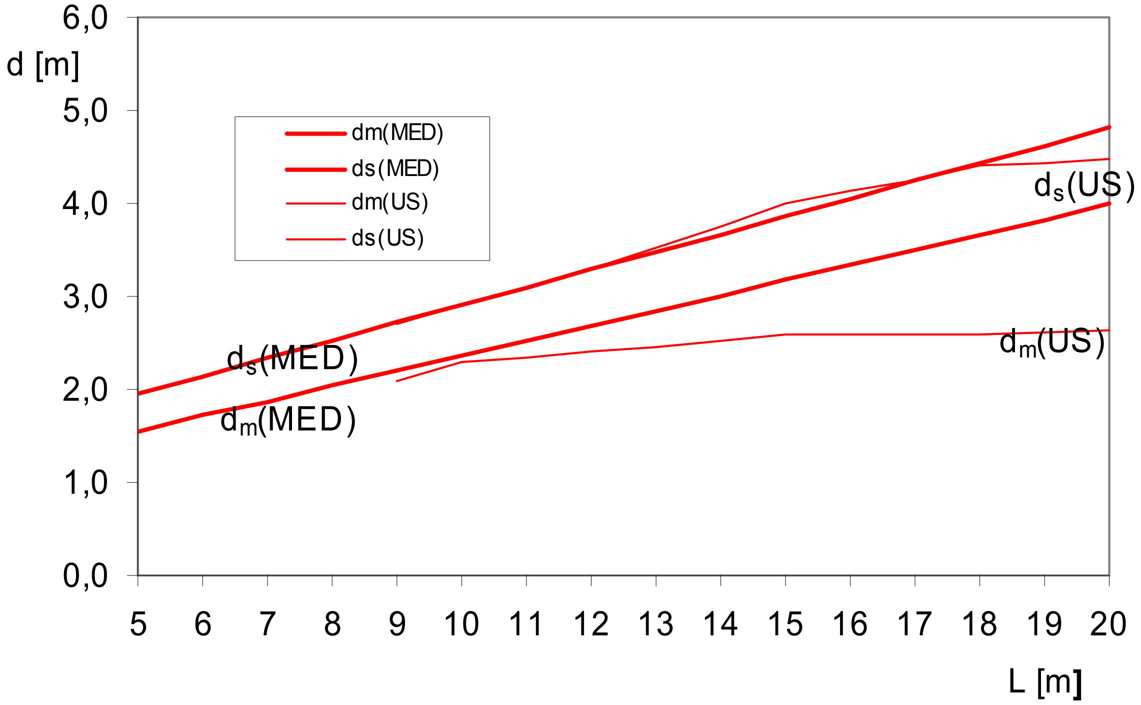

Figure 6 Draught; d_m[m], d_s[m]; m-power boat, s-sail boat; MED-Mediteranian, US-United States

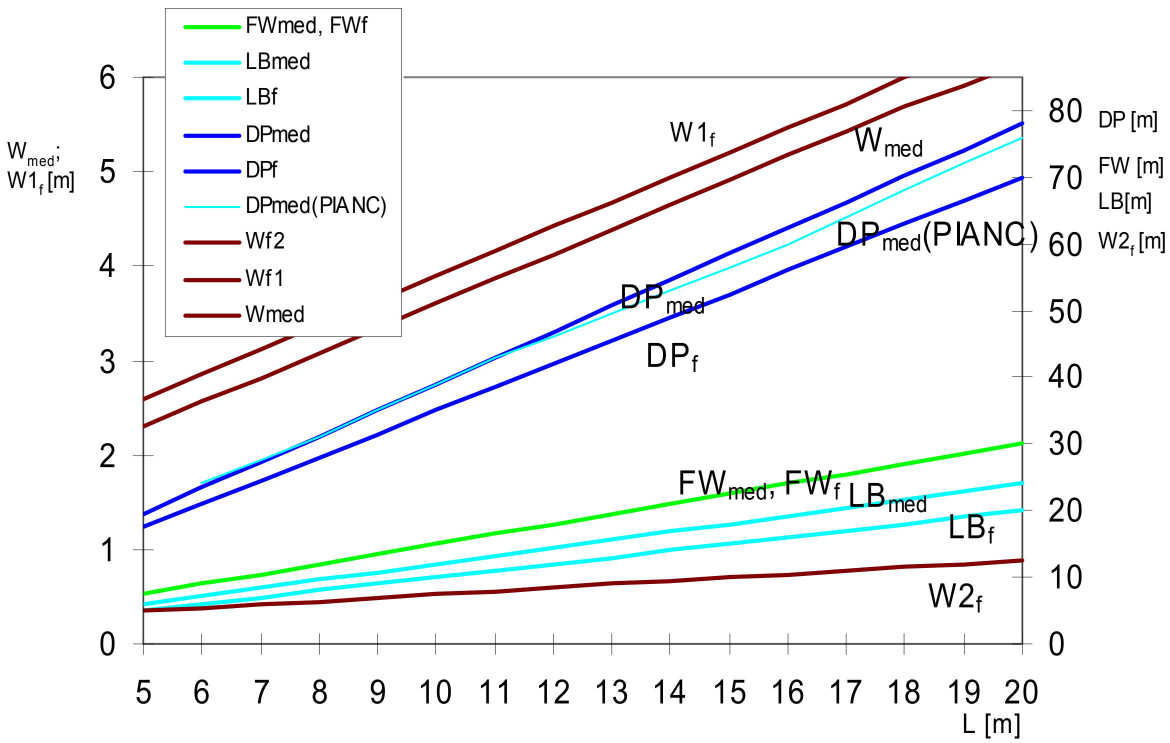

Figure 7 Geometric charachteristic of berths; W1, W2-single and double berth [m], FW[m], LB[m], DP[m; med- Mediteranian type; f- finger type.

Design wind velocity

Australian Standard[2] recommends hourly average wind velocity for design velocity which must be multiplied with gust factor. In that sense for yachts and ships in marinas is required design velocity of 30 seconds duration on 3 meters above terrain:

- $U_{30s}(3m) = c^{G}_{3600}(z = 3m, 30s) \cdot \overline{U_{3600}(3m)}$

$c^{G}_{3600}(z = 3m, 30s) = 1{,}33$	*gust factor*
$\overline{U_{3600}(3m)}$	*average hourly wind velocity 3 m above terrain.*

British Code [3] suggests using 50-year average wind velocity with duration between 1 and 5 minutes. There are no data about height above terrain.

According to Eurocode (1991) calculations of wind loads can be made with 50-year wind velocity with duration of 1 second. Gust factor 10 m above terrain for average velocity of 10 min. duration is $c^{G}_{600}(z = 10m, 1s) = 1{,}39$, and for hourly average velocity is $c^{G}_{3600}(z = 10m, 1s) = 1{,}54$, which gives:

- $U_{1s}(10m) = 1{,}39 \cdot \overline{U_{600}(10m)}$,
- $U_{1s}(10m) = 1{,}54 \cdot \overline{U_{3600}(10m)}$

where:

$\overline{U_{600}(10m)}$	*average 10 min. velocity, 10 m above terrain*
$\overline{U_{3600}(10m)}$	*hourly average velocity, 10 m above terrain.*

Drag factor

Experimental values of drag factor (C_D) [5] for sail boat, power boat and house boat in comparison to Australs, British and USA standards are given in next table.

Angle of wind blowing in table represent: bow to wind (0°), beam to wind (90°) and stern to wind (180°).

	angle of wind blowing	0°	90°	180°
boat type	power boat	0,75	1	0,95
	house boat	0,75	0,9	0,75
	sail boat	0,35	0,9	0,45
standards	AS3962-1991	1,10-1,20	1,30-1,60	1,60-2,00
	PIANC-British Notes	0,20-0,75 (0,45)	1	0,2-0,75 (0,45)
	PIANC-ASCE	0,50-1,20	0,80-1,50	0,50-1,20

Table I Drag coefficient comparison for unshielded boat[5].

Where boats are moored next to each other or on both sides of a pier it is generally accepted that the leeward boats will each attract a reduced wind load because of the shielding effect of windward boats. Recommendation of American Society of Civil Engineers [5] is to use reduction factor for shielded boats ranging from 0,1 to 0,5 of the force on an unshielded boat.

Summary

The design of marina requires careful attention to design parameters to maximize benefit, provide functional use and insure safely berths for yachts. The marina must provide suitable yacht capacity for the defined nautical fleet, provide adequate access from the sea and allow safe maneuvering of yachts inside of marina.

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Geotechnical Aspects of Small Retention Dam Vir in Croatia

Tomislav Ivšić, Eva Ocvirk
Željko Pavlin

Abstract: The small earth dam Vir is designed for the flood control of nearby town Vrbovec in middle Croatia. It should also serve as the embankment for local road connecting two villages. The foundation soil in the valley consists of several loessy silty clay layers representing relatively soft deep profile.
The paper presents the choice of earth materials in zoned embankment in order to satisfy the diverse design criteria such as stringent road construction requirements, maintaining water retention function, and using local loessy clay as much as possible. The paper also present the choice of design foundation soil parameters relying on soil data in order to get more realistic estimate of settlements. The method of Reference Correlation Level was used for integration of all collected data in the unique site sedimentation sequence. The resulting geotechnical model and the range of analyses results are also discussed.
Keywords: earth dam, geotechnics, cohesive soil, parameters, RNK

1. Introduction

The retention Vir is designed for the flood protection of town Vrbovec in middle Croatia. It is realised by a small dam in the valley of creek Zlenin between two villages. The retention controls the catchment area of 2.64 square kilometres; it has the area of 68.000 m^2 and the total volume of 67.000 m^2. It was designed to accept the water wave of 50 years return period by water discharge through foundation outlet only (the capacity 3.0 m^3), and to safely accept the 100 years water wave by combined discharge through bottom outlet and spillway.

The evacuation structure is positioned at the lowest point of creek valley. Both, bottom outlet and the spillway, are united in the unique concrete structure penetrating through the dam body.

The dam is positioned along the local unpaved road, and it is planned to replace the existing valley road to the dam crest. This demand defined the width of dam crest as 7.5m. The length of the dam is about 365m, and maximal height above the terrain surface is about 4m in the centre of the valley.

Figure 1 presents the basic features of the creek valley and the plan of the designed dam.

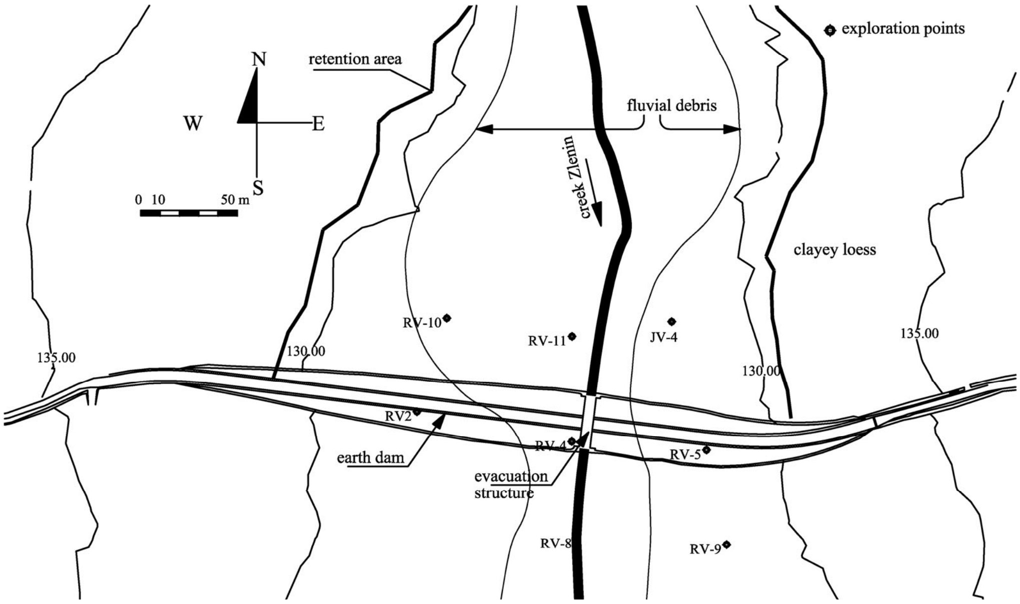

Figure 1 The plan of the dam and the valley with the position of geotechnical explorations

2. Characteristic dam cross-section

The basic functions of dam are hydrotechnical (as a temporary retention for flood control) and transportation (as a road embankment). The road is planned for local traffic of vehicles, trucks and agricultural machinery. This function is fulfilled everyday, and water retention function happens occasionally. From these functional aspects also the basic, fixed costs of construction arise, as the costs of pavement structure and the water evacuation structure. The rest of costs is in a small embankment which has to meet the specifications as a road base and to retain the water during expected floods.

The geotechnical investigations for material borrows showed that in feasible shallower depths the ground consists of useless wet fluvial debris in the creek valley and loessy clay in nearby hills around the valley. The foundation soil of embankment to relevant depths consists of silty clays of medium to high plasticity, which is relatively "soft" profile from aspects of deformation, but sufficiently impermeable base for temporary water retention.

The technical conditions for the foundation base of asphalt pavement structure request the zone of at least two meter thick well compacted layer to form a firm and durable base able to resist traffic loads and vibrations without significant deformations of pavement surface.

Since the majority of the embankment dam is of low height, it was estimated that the compaction criteria for the pavement base could not be easily achieved with the silty clay from valley abutments, and it was decided to construct this pavement base by better material from mid-distant quarry.

The above reasoning from the functional and constructional geotechnical aspects resulted in the zoned dam cross section shown in Figure 2. The characteristic zones form relatively robust embankment to be easily constructed by unknown contractor.

The main parts of the section are: central core of gravel with fine particles (wide granulometric composition) to obtain the firm base for pavement insensitive on wetting, the supporting shell of silty clay from nearby borrows which is realising the water retaining function and the wide chimney drain with downstream drain outlets. The soft debris in the base of central part of dam will be replaced to depth of 2.5m with compacted clay to reduce deformations and achieve water tightness of the creek bed.

The slopes of the embankment are 1V: 3H due to low cohesion of silty clay materials and comply with average slopes of surrounding hills of similar materials. They will be protected from erosion by grass. The ample drainage system should assure the collection and evacuation of water coming from possible cracks or erosion in dam body, eventual seepages through pavement structure or indicated ground water coming along the dam from valley abutments. The selected granular materials in drains are separated by geotextile from surrounding finer particle materials.

Due to large expected settlements it was recommended to monitor the crest deformations and to construct the asphalt layers afterwards, when majority of settlements will realise.

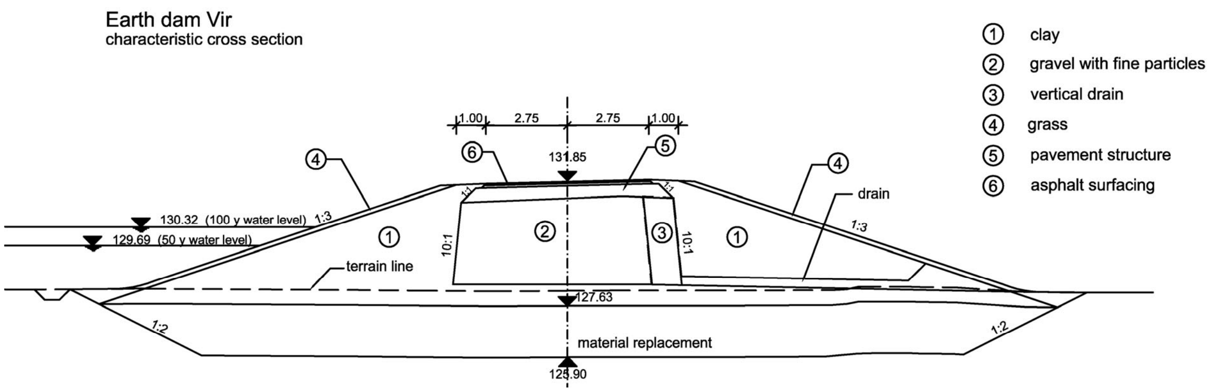

Figure 2 The characteristic dam - road embankment cross section

3. Ground conditions

3.1 Site investigations

The ground conditions at the dam site and possible borrow areas were explored using several investigation techniques: geoelectrical 2-D tomography to depths of 50m, investigation boring to 25m depth with samples and in-situ tests and laboratory testing for classification, index, hydraulic, mechanical and compaction properties of soils. Also the engineering geology reconnaissance of wider area was made.

According to *geological determination* the ground in wider area and to greater depths consists of quaternary sediments, basically fluvial debris and continental non-carbonate loess. Fluvial debris is shallow sediment that was brought by the creek. It is sandy silt material with a lot of clay particles, organic matter and plant roots. Continental loess is material known in this part of country as loessy clay, macroscopically presented as “marble” clay of yellowish-brown colour with irregular parties of grey silty clay. In mineral composition dominate detritic quartz grains with some feldspar and clay minerals. The absence of calcium carbonate is characteristic. In the base of these sediments sublayers or lenses of sandy clay can be found occasionally. The approximate borders of fluvial debris and loessy clay are shown in Fig.1.

The *geoelectric measurement* distinguished the shallow zones of silty sand debris in the vicinity of creek and on the eastern part of valley, possibly indicating former creek bed. The second recognized characteristic media were clay sediments with lowest resistance. These sediments are present in all investigated profiles and are more than 20m thick. The third region is in greater depths, where great lenses of clayey silt and sand could be interpreted from greater electric resistances. The positions of geoelectric profiles coincide with the profiles through exploration boreholes (in Fig.1), so that the crosschecking of results can be made.

The more detailed insight in ground properties was obtained from *exploration borings* and testing of samples, although at relatively distant exploration points. The positions of boreholes (RV) and shallow trench (JV) are shown in Fig 1 for the dam site. The other investigations for material borrows are outside the presented plan.

Geotechnical characterization of fluvial debris samples presents it like silt or clay of medium to high plasticity (the results are close to A-line, both for silts and clay, meaning that it is generally the same material). The clay content varies from 6-13%, while the sand content is usually less than 10%, except at particular locations where more sand was found at the layer bottom. The debris extent is to depths of 2.5 - 3.0 m. The ground water level was found at depths 1-1.5m (along the dam axis). Above the water level debris has stiff to very stiff consistency, but under the water level this material was soft to semi stiff (the consistency indices less than 0.5), even very soft at locations near the creek.

The material geologically determined as continental loess or loessy clay was classified as clay of medium to high plasticity (CI or CH). The variations with depth indicate diverse sedimentation conditions. The consistency of these materials is stiff to firm, as demonstrated by consistency index of 0.8-1.0, relatively high values of uniaxial strengths and overconsolidation ratios from 2 to 5. The permeability coefficients ranged from 10^{-9} to 10^{-10} m/s, which means that the temporary retention has practically impermeable base with thickness more than 20m. At the bottom of some boreholes the lenses of silty sand were found, but it is out of practical engineering interest.

3.2 Geotechnical model

For further geotechnical analyses an idealised model of foundation soil was established, which includes the geometry of the soil strata and the representative values of relevant parameters. The detailed stratification of soil profile with depth was not clearly visible from distant borehole profiles or geoelectric profiles. In order to associate properly the measured parameter values and to reduce the scatter of results, the soil stratification was obtained by Reference Correlation Level method (Ortolan and Mihalinec, 1998; Jurak et al, 2004).

The Reference Correlation Level (RNK - the acronym in Croatian language) is defined as recognizable and visually identifiable bedding plane or any other reference plane within structural feature, in relation to which the altitude of all studied profiles can be unambiguously defined. Such a plane is a part of a single litostratigraphic sequence or geotechnical correlation column. Usually the visually recognizable, distinguishable, layers in soil column are used as RNK. In situation, like at the Vir site, where such layer is not found, the RNK and the geotechnical correlation column are obtained by graphical procedure, provided that there is a sufficient density of of laboratory classification results. Most common is the use of plasticity index, which is correlated to clay content and clay mineralogy composition. The depth distribution of plasticity index then reflects the sedimentation sequence and is a sort of sedimentation “fingerprint” in soil profile. Fitting this index distribution in a unique correlation column in relative altitudes enables the reasonable allocation of any other test result or parameter value to each defined layer along the depth or vertical sequence of formations covered by investigations.

In Figure 3. the distribution of plasticity index with depth in boreholes along the dam axis is presented together with on site estimated soil layers and individual classification results. The level with highest value of plasticity index (IP) was taken as the starting point (RNK) for graphical procedure. In Fig 4. the unique column in relative altitude (or relative depth) is presented where the best fit (the least scatter) of IP distribution from several boreholes was obtained. The bottom layer with occasional silt and sand lenses has the largest scatter. Also presented is the distribution of liquid limit (w_L), which helped in selection of typical clay layers along the depth (the criteria 35% and 50% were used). The results for undrained strength (from unconfined compression and vane shear test) are associated in relative depths and the selected characteristic values for typical layers are presented. The values for angle of internal friction at constant volume (φ'_{cv}) are obtained using correlation with plasticity index (Manual, 1990).

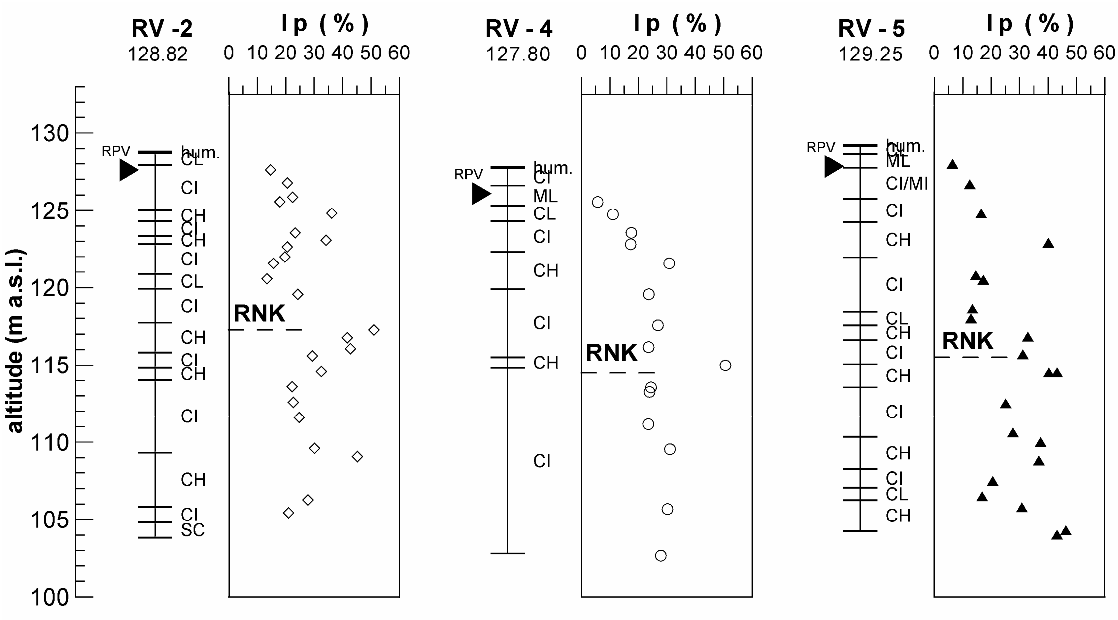

Figure 3 Distribution of plasticity index with depth

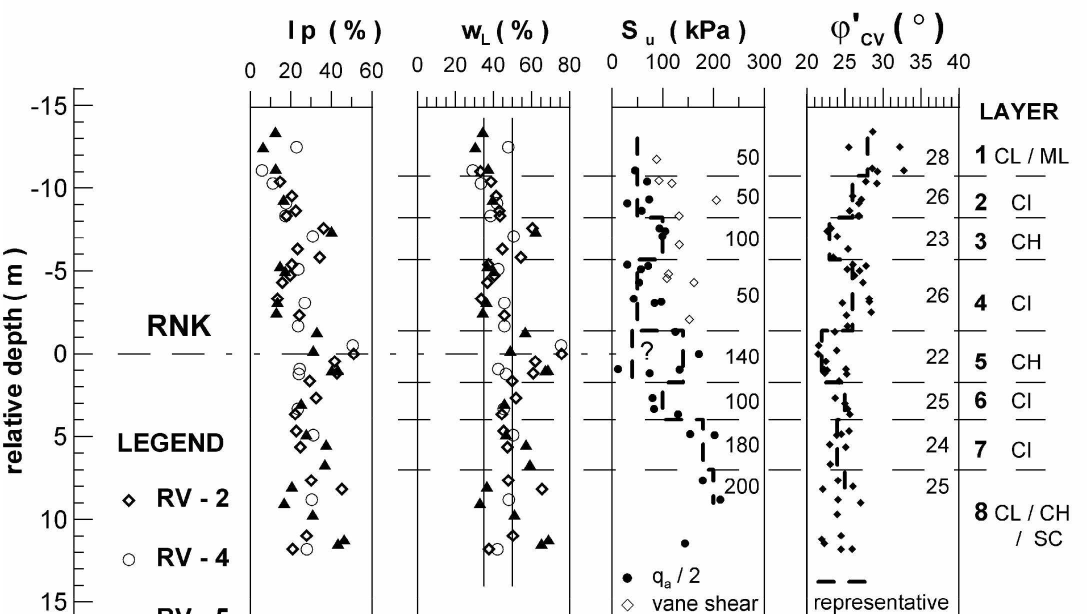

Figure 4 The geotechnical correlation column with typical clay layers

The distribution of overconsolidation pressures presented in Fig.5 shows in upper layers (2 and 3) the trend characteristic for desiccated crust, where “overconsolidation” was obtained by capillary forces and chemical postsedimentational processes in atmospheric conditions and conditions of partial saturation, in region of water table oscillations. In lower, older, layers (5-7) the overconsolidation pressure difference seems to be constant indicating typical overconsolidation by additional pressure (probably due to low underground water levels in past). Also specific is the visible break of mentioned trends in layer 4. For more detailed interpretation more data would be needed. However, the presented data, especially from upper part of profile, indicate that the past long-term underground water level in recent sediments was significantly lower than actually measured values. Also shown in Fig 5 is the distribution of initial void ratio (obtained from oedometer tests and from natural water content in case of full saturation) and the distribution of compressibility coefficient (C_c), also obtained from oedometer test and by correlation with liquid limit.

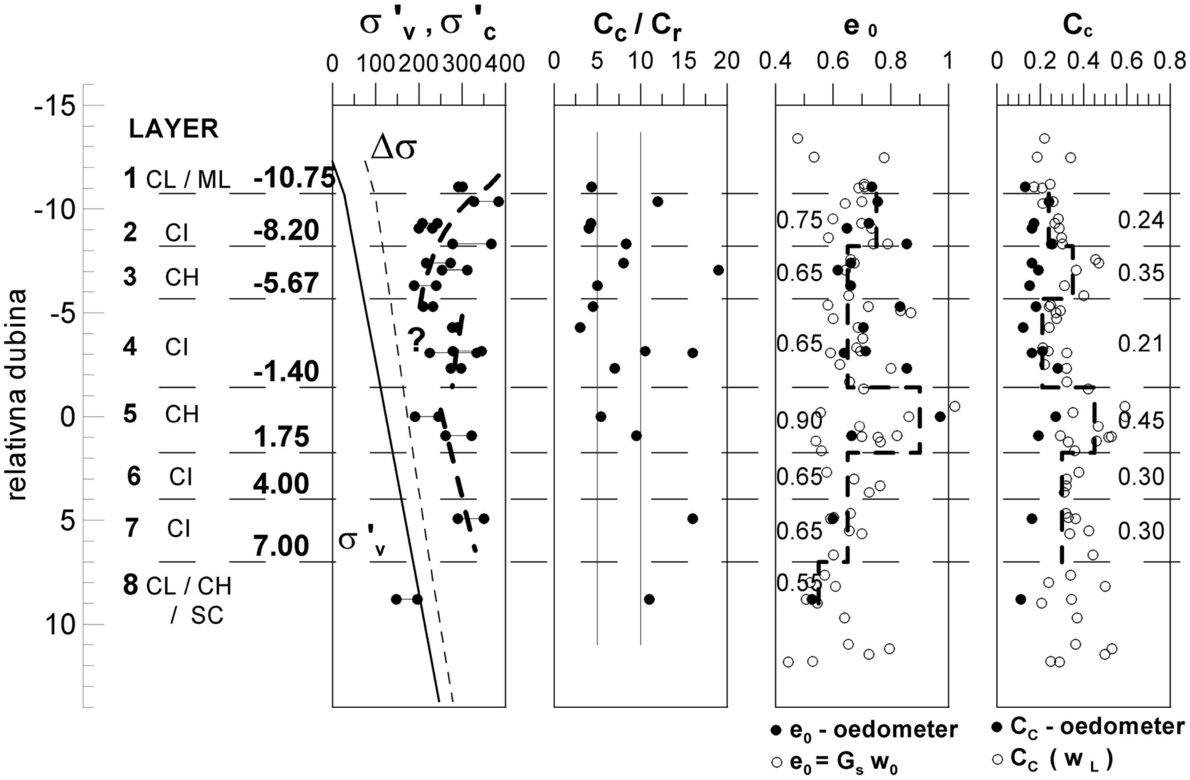

Figure 5 Oedometer test results in geotechnical correlation column

The geotechnical column from above figures in relative depths was reverted to absolute altitudes for specific borehole and idealised geometrical model of layers in profile along the dam axis. The inclination of interpreted layers is different from the terrain surface line in the middle and eastern part of valley. This indicates probable former terrain surfaces during time when sedimentation of loessy clay layers occurred, and is consistent with, geologically recent, process of filling of valley bottom with fluvial debris.

3.3 The settlement analyses

The additional loading of embankment in highest portion is estimated to $\Delta\sigma \approx 80$ kPa at the foundation level and is diminishing with depth. The distribution of overconsolidation pressures in Fig.5 shows that practically through the whole depth of profile the sum of actual effective vertical stress and additional stress is lower than the corresponding overconsolidation pressure. Therefore the settlement analysis was performed using the recompression coefficient (C_r) for all layers.

The interpreted values of C_r from several oedometer test showed a large scatter of results with average value of C_c / C_r = 8.7. According to the literature the expected ratios C_c / C_r are 10-20 (Carter and Bentley, 1991) or C_c / C_r = 5 (the Cam clay model, Manual, 1990). So, for the analyses the range of C_c / C_r = 5 - 10 was selected as the conservative representative of maximal and minimal expected embankment settlements. The upper replaced soil layer was modelled with linear modulus in range M_v = 12-17 MPa.

The range of maximal total long-term settlements using above parameters was from 8 to 14 cm in the middle portion of dam.

Also, the initial settlements were estimated using linear secant modules correlated to undrained strength. The calculated maximal initial settlement values are less than 4 cm.

4. Summary

The design of small earth dam Vir, which serves as a water retention structure and road embankment, included various geotechnical aspects in selection of earth material zones in typical section. The final distribution of earth materials in cross section is the robust compromise of functional, construction and economic aspects.

The Reference Correlation Level method was used for creation of geotechnical model of dam foundation soil, which is formed of thick cohesive layers with varying properties. This method in a rational manner modelled the sedimentation sequence enabling the appropriate allocation of various data and test results from distant locations in a consistent way.

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Terrain Measurements and Calibration of Hydro Dynamical Model of Water Work Drahovce – Madunice

Radomil Kveton, Peter Dusicka

Abstract: The simplest part of Vah hydroelectric system from the point of view of mathematical modelling is the water work Drahovce – Madunice. Modelling methods verified on other channel hydro power plants are easily applicable on this hydroelectric system. Last year was made by our department terrain measurements and research on this water work. The results of measurements were used for the better calibration of hydro dynamical model of this water work. The presented paper describes this process.

1. Introduction

The water work Drahovce - Madunice is the simplest part of Vah hydroelectric system from the point of view of mathematical modelling (Fig.1). Modelling methods verified on other channel hydro power plants are easily applicable on this hydroelectric system. These methods are based on decomposition of modelling system. The system is decomposed to separate sections with own defined boundary conditions. At the same time we can verify complex methods to obtain more realistic description of interaction between natural Vah riverbed (together with water reservoirs) and artificial channels of water power plants. The results from the modelling of the hydro power plant (HPP) Madunice will be applied to the other parts of Vah hydroelectric system. The goal of this application is to get a complex hydro dynamical model of the catchments of the river Vah.

An important part of the modelling is also obtaining respectively verifying of hydraulic characteristics of channels - the roughness coefficient of wetted perimeter.

The presented paper describes terrain measurements with intention of obtaining these hydraulic characteristics at water work Drahovce – Madunice.

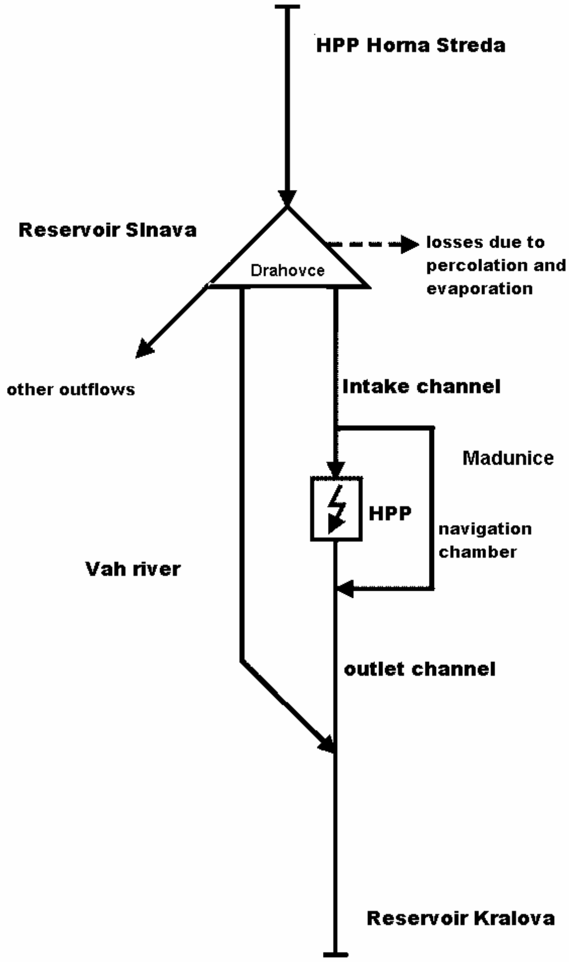

Fig. 1 Scheme of water work Drahovce – Madunice

2. Description of conditions at water work VD Drahovce - Madunice

Water work Drahovce - Madunice is a water work of the 1.category. It is situated on the river Váh between the river km 101,90 to river km 119,20. The weir Drahovce is situated in river km 113,40. Water work Drahovce – Madunice is a multipurpose work. The most important purposes are the hydropower utilization and flood protection of local area. Water work Drahovce – Madunice consists of these structures:

- reservoir Sĺňava,
- weir Drahovce,
- intake channel,
- hydropower plant,
- navigational lock,
- outlet channel.

3. Hydraulic bonds within the involved reach of the Váh cascade

The stage of the river Váh, between the water work Drahovce and water work Kráľová, is energetically utilized only in its upstream stage under the water work Drahovce by hydropower plant Madunice. Water flows from the reservoir Drahovce in intake channel to hydropower plant Madunice. A view on the intake structure of HPP Madunice intake channel and weir Drahovce is in the figure 2.

Fig. 2 Intake structure of HPP Madunice intake channel and weir Drahovce

Fig. 3 The hydropower plant Madunice

The intake channel has trapezoid cross section with bottom width 15,25 – 60 m, slope of upstream face 1 : 1,75. It is sealed with concrete face sealing. The length of the channel is 6,542 km.

HPP Madunice is created as a peak hydropower plant with maximal turbine capacity 3 x 100 = 300 m3s-1. Maximal head is 18,36 m, minimal head 11,71 m and installed power output 43,2 MW. Length of service time in a common year is around 3600 hours. A view on the HPP Madunice is in the figure 3.

Water flows from HPP Madunice in an outlet channel. This channel has trapezoid cross section with bottom width 57,00 m, slope of upstream face 1 : 3. It is fortified by rock base and spreaded rocks. The orifice of outlet channel to Váh is by chute with slope 1 : 10. Minimal hydrostatic water surface in the outlet channel is fixed by the stemming at level 139,40 m a.s.l.

A view on the chute at the end of the HPP Madunice outlet channel is in figures 4 and 5.

Fig. 4 The chute at the end of the HPP Madunice outlet channel (upstream view)

Fig. 5 The chute at the end of the HPP Madunice outlet channel (downstr.view)

It is a derivation canal hydropower plant from the hydraulic engineering point of view, where length is the dominant dimension of the canal. Upstream boundary of the derivation canal is formed by the Drahovce reservoir and downstream boundary by the chute at the end of the outlet channel.

In a physical point of view are the intake to the channel by the reservoir Drahovce, hydropower plant and the chute at the end of the outlet channel, singular points. In these points it is possible to define hydraulic parameters of the flow and its behaviour in time. Thus by the analysis of hydraulic regime and hydraulic bonds, whole scheme need not be solved at once. The scheme is divided in separate stages:

- Drahovce – HPP Madunice (intake channel of HPP Madunice)
- HPP Madunice – chute (outlet channel of HPP Madunice).

After the separation, can be singular points considered as boundary profiles and known time behaviour of hydraulic parameters in these profiles can be considered as boundary conditions for flow analysis.

4. Methodology of verification of derivation canals roughness coefficient

Verification of roughness coefficient is possible only by direct measurement during operation in these ways:

a. steady state measurement – steady non-uniform flow in channel, attained by continuous start on required flow rate through hydropower plant – minimizing of wave transition effects in channel,
b. unsteady state measurement – unsteady non-uniform flow (used only in case, when attaining of steady state is not possible, results are less precise).

4.1 Measurement of roughness coefficient by steady state

Measurement by steady state consists of next steps:

- attaining of steady state without any flow before the beginning of measurements – steady water levels in derivation canals and compensation reservoirs are necessary not only for the probes calibration but also as initial conditions for mathematical modelling
- attaining of steady flow state.

Frequency of data acquiring (water level measurement), once per minute, is usually sufficient for major part of hydrodynamic processes.

Conditions for attaining steady flow state vary in order to type of derivation canal:

a. stage Drahovce – Madunice (intake channel of HPP) – inflow to reservoir Drahovce must be the same as flow trough HPP Madunice,

b. outlet channel of HPP Madunice – its orifice is in natural riverbed – measured profiles must be situated in sufficient distance from impounding structures in channel (dikes).

For roughness coefficient calculation of measured stage by steady non-uniform flow has been the segment solution method and its appropriate formulas for hydraulic characteristics:

$$\Delta z = Q^2.\left[\xi.\left(\frac{1}{S_d^2} - \frac{1}{S_h^2}\right) + \frac{l}{K_p^2}\right]$$

$$K_p^2 = S_p^2.C_p^2.R_p \tag{1}$$

$$S_p = (S_d + S_h)/2 \qquad O_p = (O_d + O_h)/2 \qquad R_p = S_p / O_p$$

$$C_p = \frac{1}{n}..R_p^{1/6}$$

where Δz - water level elevation change on the stage z_h - z_d
Q - steady flow rate
ξ - coefficient for widening of channel 0,028
for constriction of channel 0,056
S_d, S_h - flow area of downstream and upstream profile
l - length of stage
K_p - average flow-rate module
S_p - average flow area
C_p - velocity coefficient by Manning
R_p - average hydraulic radius
O_p - average wetted perimeter
O_d, O_h - wetted perimeter of downstream and upstream profile
n - roughness coefficient

Final formula for roughness coefficient computation, obtained by adjusting upper formulas (1):

$$n = \sqrt{\frac{\Delta z - Q^2.\xi.\left(1/S_d^2 - 1/S_h^2\right)}{l}}.\frac{S_p}{Q}.R_p^{2/3} \tag{2}$$

Analysis of this formula (2) shows, that the final computation error is relied on accuracy of difference measurement between upstream profile (z_h) and downstream profile water level elevation (z_d). In regard to limited precision of measurement (centimetres) and other surround influences such as water surface waving (e.g. caused by wind), it is needed to attain maximal difference of measured water levels, what is succeeded in formula (1) by maximal flow rate in channel and maximal length of measured stage.

4.2 Measurement of roughness coefficient by unsteady state

Measurement results by unsteady state can be used only for indirect assessment of roughness coefficient from several different scenarios of measurements.
For data processing is used a hydrodynamic model (HDM) for modelling unsteady non-uniform flow according to scheme:

- calibration of the HDM for maximal flow rate scenario,
- verification of the HDM for other scenarios.

For solution of connection of inlet channel to the compensation reservoir was used a mathematical model of reservoir based on volume balance

$$V_{t+\Delta t} = V_t + \sum Q_{pri}.\Delta t - \sum Q_{odt}.\Delta t \tag{3}$$

where V - water volume in reservoir
t, Δt - time and time step
Q_{pri} - inflow to reservoir
Q_{odt} - outflow from reservoir

and known reservoir volume curve $V = f(h)$, which enables backward computation of water level in reservoir from volume of reservoir.

Used HDM is based on numerical solution of Saint-Venant partial differential equation system as follows:

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} - q_\ell = 0 \tag{4}$$

$$\frac{\partial(\beta Q V)}{\partial x} + \frac{\partial Q}{\partial t} + gA\frac{\partial h}{\partial x} = gA(i_0 - i_e) + q_\ell v_\ell$$

where Q - flow rate [m^3s^{-1}]
A - flow area [m^2]
q_l - density of lateral side inflow or outflow [m^2s^{-1}]
x - profile distance from the beginning (x=0) in flow direction [m]
t - time [s]
V - average section velocity [ms^{-1}]
h - water level [m]
g - gravitation acceleration [ms^{-2}]
β - correction factor reflecting the influence of non-uniform velocity distribution
i_0 - bottom gradient
i_e - power line gradient
v_l - velocity component of side inflow or outflow in direction of axis x [ms^{-1}]

For conversion of foregoing equation system to numerical solution by the finite differences method has been used Preissmann implicit scheme with weight coefficient 0,67.

5. Terrain measurements (measurements “in situ“)

Measurements in real conditions (measurements “in situ“) were realized for the purpose of calibration and verification of the hydrodynamic model of inlet and outlet channel of HPP Madunice.

Measurements took place in September 13~16 2004, while on September 13 had been measurement devices installed and on September 14, 15 and 16 was the measurement in progress. By the measurements had been the water level regime examined depending on the operation of HPP Madunice in chosen profiles of inlet and outlet channel.

For water levels measurements were used pressure probes from company MERET Bratislava. Probes had worked in real time. Their time step was set on one data record per one minute. Installation of probe in protection pipe on the channel bank is shown in figure 6.

Used chainage of the probes placement (measured profiles) was used according to operation manual of water work Drahovce – Madunice – in the downstream direction, where km 0,000 is situated in the intake structure axis of the inlet channel on the weir Drahovce. Five measured profiles were situated in the inlet channel.

Two probes were installed on the upstream side, 15 m and 100 m above the intake structure of the inlet channel (above the coarse rack). Probe location, directly above and under the coarse rack of the intake structure, had been chosen with purpose to determine local hydraulic loss. This loss is being considerable mostly by clogging of the coarse rack (fig. 7) and it causes decreasing of water level at the beginning of the intake channel. This results in increase of hydraulic losses in the intake channel and thus lowering the head used by HPP Madunice. Final result is decrease of power output and electric energy generation in HPP Madunice.

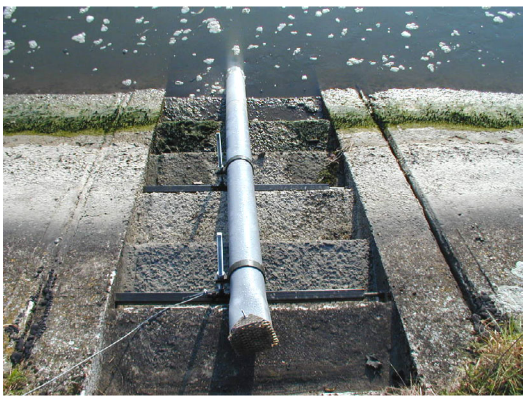

Fig. 6 Probe protection pipe on the bank of the HHP Madunice inlet channel

Fig. 7 Intake structure of the inlet channel – clogging of coarse rack

In the outlet channel including natural riverbed of Váh were 3 measured profiles situated. Besides these probes, 2 probes were installed in natural riverbed of Váh and 1 below the chute at the end of the outlet channel. These probes monitored the water level beyond the chute to enable analysis of the chutes influence on chute overflow discharge and the influence of the chutes impounding by water from natural riverbed of Váh (below the confluent of the outlet channel and natural riverbed).

The measurement proceeded according to in advance prepared and approved program. The measurements were complicated by the fact that one aggregate had been shut down due to repairs. The capacity of the HPP was 2 x 100 m^3s^{-1} = 200 m^3s^{-1}. The operation of the HPP Madunice had been according to agreement with HPP dispatch centre of Slovenské elektrárne a.s., Vodné elektrárne o.z. Trenčín, such as to ensure during the operation in every day of the measurement the achievement of steady state for minimum of 4 hours (steady flow rate through the HPP Madunice). This agreement was for a scale of discharges with the operational start from standstill in 20 minutes: 180 m^3s^{-1} (September 14 2004), 130 m^3s^{-1} (September 15 2004) and 80 m^3s^{-1} (September 16 2004). Before the operation start (7:00) was since midnight zero discharge to set the hydrostatic water level. Real flow rates achieved during measurements were 190 m^3s^{-1} (September 14 2004), 132 m^3s^{-1} (September 15 2004) and 82 m^3s^{-1} (September 16 2004).

The power operator recorded in real time (in 1 minute intervals) following data:

1. water level at reservoir Drahovce,
2. Upstream water level at HPP Madunice,
3. downstream water level at HPP Madunice,
4. flow rate in HPP Madunice.

These data served as underlay for following measurement evaluation and for calibration and verification of the hydrodynamic model of the inlet and outlet channel of HPP Madunice.

6. Achievements

After the data processing of terrain measurement records, has been the method of following processing divided in these steps:

- calculation of the roughness coefficient of the outlet channel – steady flow rate state had been attained – segment solution method has been used for calculation (4.1),
- calculation of the roughness coefficient of the inlet channel – steady flow rate state had not been achieved for operational reasons (repairs on upstream group of HPPs and economically disadvantageous release of idle discharges by other HPP objects) – simulation on the hydrodynamic model had been used for calculation (4.2).

Results of roughness coefficient computation for the outlet channel are shown in following table.

day	flow rate $m^3.s^{-1}$	downstream profile		upstream profile		roughness coefficient n
		profile	water level m a.s.l.	profile	water level m a.s.l.	
14.9.2004	190	10	140,45	8	140,73	0,0224
15.9.2004	132	9	140,27	8	140,37	0,0220
16.9.2004	82	9	139,27	8	140,02	0,0206

Then, for outlet channel was for next processing used average value of the roughness coefficient n = 0,022. Value of the roughness coefficient for the inlet channel has been after consideration of technical state of the inlet channel set as increased according to design value (0,015). It has been experimentally designed on value n = 0,020 and consequently verified by modelling on the hydrodynamic model.

7. Conclusion

The measurements at water work VD Drahovce – Madunice enabled refinishing of the methodology of roughness coefficient in derivation canals assessment. The correct assessment of the roughness coefficient in derivation canal is necessary for quality increase of the hydrodynamic models output.

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Quay Structure on River Sava in Croatia

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Josip Galić

Abstract: The 120 meter reinforced concrete quay on river Sava at Slavonski Brod, Croatia, has been finished, after eighteen months of construction work. Foundations of the structure, consisted of reinforced concrete geotechnical diaphragms and piles. The quay structure was simple and robust. It has been constructed of in-situ made base girders, longitudinal and transverse walls, and of columns. Reinforced concrete elements at upper level such as girders and plates were prefabricated, while upper slab was cast in place. Back-wall has anchorage tendons at the top for decreasing of top horizontal movement of structure. The anchorage tendons, two of them at every 5,0 m, were anchored to the another longitudinal wall that was placed in intact soil, 15,0 m away from the back wall. The space behind back-wall was filled with soil and compacted in layers.
Keywords: quay structure, reinforced concrete, prefabricated elements, construction, walls

1. Introduction

Quay structure on the bank of river Sava, near town Slavonski Brod, about 200 km east from Zagreb, was planned to be constructed in several phases. The first phase of reinforced concrete quay structure consists of two parts of 60 m length each, separated by movement joint. Inland waterway of river Sava is of IV international class, as found in Review of the Classification of European Inland Waterways, (1996), [1]. Navigation clearance of the waterway is 2,9 x 70,0 m with the minimum radius of 400 m. The waterway can take pushing convoy of one lighter and boxer whose dimensions could be as much as 85,0 x 9,5 x 2,5 m. Oscillation, i.e. difference between the extreme water levels is 10,0 m and oscillation of navigable water levels is 7,0 m. The development plan predicts in perspective that Slavonski

Brod will become an traffic and trade center. Inland waterway goods traffic on landing stage would be realized by reloading gravel, sand, grain and raw oil. The quay structure was constructed of concrete grade 40 (MPa) and reinforcement of grade S-400 (MPa) rebar.

2. Quay structure loadings

- Sustained loading: structure self-weight, hydrostatic pressure for regular high, mean and low water level (buoyancy), soil pressure for regular high, mean and low water level, uniform live load: 2 rows of containers, with loading of 35 kN/m2, crane not in use; crane in use; crane driving, road loading on quay, snow, ice.
- Variable loading: additional soil pressure from: uniform live load on embankment; mobile crane on embankment; road load on embankment, temperature, braking and accelerating of: mobile crane; road vehicle, operating wind (v=20m/s) on crane in use, perpendicular to and along the quay, ship impact by landing for different water levels: perpendicular and along to quay length; rope pull force, for different water levels: perpendicular and along to quay length; vertical force due to friction of ship toward quay, for different water levels; hydrostatic pressure of residual water behind wall; wave pressure for regular high water level; wave pressure.
- Accidental loading: wind storm (40 to 50 m/s), on crane; ship impact by landing; hydrostatic pressure for accidentally high or low water level; seismic force on self weight of structure at mean water level; seismic loading on soil behind wall by different water levels; seismic forces on crane (horizontal perpendicular and alongside).

3. Structure

The quay structure has been divided into two parts of 11,6x60,0 m each. From working plateau, geotechnical diaphragm of 60-cm thickness and 15,0 m depth in soil is constructed on the riverside. The so called half-diaphragm is constructed 5,25 m apart from the center of diaphragm in transverse direction. Its depth in soil is 13,0 m, thickness is 60-cm and length 2,5 m, and its centre to centre distance is 5,0 m apart alongside, which means that every 2,5 m between two half-diaphragms is soil. Drilled piles of 1,0 m diameter are constructed on backside of quay, 5,25 m apart from the half-diaphragm in transverse direction, with depth of 13,0 m inside soil and 5,0 m apart in longitudinal direction. Figure 1 shows cross-section of the reinforced concrete structure and slope of riverbank before construction. Figure 2 shows construction of geotechnical diaphragms and piles.

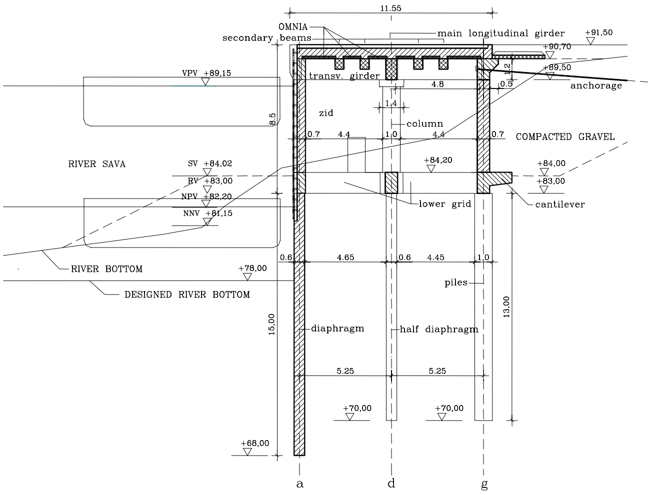

Figure 1 Cross-section of the quay structure


Figure 2 Construction of geotechnical diaphragms and piles

The back-wall cantilever and anchorage tendons on the right side of the structure (Figure 1) could be seen as well.

Upper-structure (from the level +83,00 up) consists of lower and upper longitudinal and transverse girders grid. On the top of the lower grid, longitudinal walls were constructed (riverside-wall and backside-wall), as well as transverse walls and columns. Upper grid consists of longitudinal and transversal girders as prefabricated reinforced concrete elements. Upon them prefabricated reinforced plates of span L = 1,1 m and 1,8 m, and thickness of h = 8 cm, were placed. Girders and prefabricated plates were then all connected by reinforcement and cast in place concrete slab.

Back-wall has the longitudinal cantilever at its bottom for the reason of decreasing the overturning moment caused by soil pressure, and anchorage tendons at the top for decreasing of top horizontal movement of structure. The anchorage tendons, two of them at every 5,0 m, were anchored to the another longitudinal wall that was placed in intact soil, 15,0 m away from the back wall. The space behind back-wall was filled afterwards with soil and compacted in layers. In transverse direction, between front and back longitudinal walls, the 30-cm walls, 10,0 m apart were constructed above transverse girders. In the centre of the field between transverse and longitudinal walls (at the connection of transverse and longitudinal girders) the RC columns of cross section 1,0x1,0 m were constructed. Those columns, 10,0 m apart of each other, were provided with square (1,4 x 1,4 m) heads at their top level of +89,5 m, in order to allow placing of prefabricated girders of upper grid on top of them.

Concrete cover of all elements was at least 4 cm for those that were not in contact with soil. For those in direct, unprotected contact with soil, cover was 7 cm.

Of all loading that act on top surface of structure at level +91,50 m, the mobile crane loading is the most important, because of its weight and size. Besides, the mobile crane can move in all possible unfavorable places for structural elements. This type of crane has four axles on wheels and extensible legs. Distances from center to center of legs are 11,50 m in one and 10,00 m in other direction. The size of each leg is 1,20 x 1,80 m. Its total weight in work is 2500 kN. The maximum load on one leg could be as much as 1462 kN which means 677 kN/m2 on reinforced concrete slab below the leg. This kind of load is way above any other load and therefore is relevant for design. Calculation was done for two parallel-working cranes.

Figure 3 and 4 show reinforcement for front wall in September 2003.

Figure 3 Front wall reinforcement (beginning of September 2003)

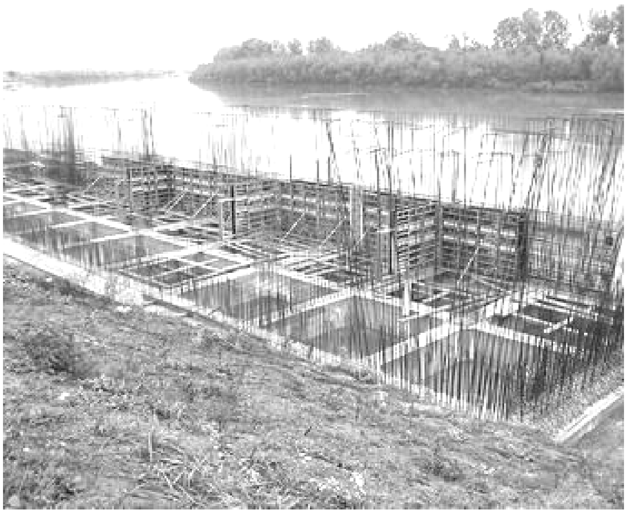

Figure 4 Front wall reinforcement (end of September 2003)

In order to prevent extensive cracking in reinforced concrete elements, the thinner steel ribbed bars were used. The diameter of steel bar was no greater than ϕ22 mm in lower zone and no more than ϕ25 mm in upper zone, and the tensile stress in steel bars during service were calculated to be no greater than 300 N/mm2. All girders columns and slabs were reinforced with steel bars S-400. One of the reinforcement-plans could be seen in Figure 5.

4. Analytical model

Due to large horizontal forces of soil pressure on back wall, it was necessary to provide anchorage tendons at upper part and cantilever at bottom part of the back-wall of the structure.

The structure analysis has been made on the 3-D model. This analytical model comprised foundation structure and upper structure. The contacts of both structures have been modelled by springs. The mechanical properties of springs were obtained from soil mechanic expert, according to test results, as well as from iterative calculation of soil force/displacement interaction. Diaphragm has been modelled by shell-elements, while half-diaphragm and piles have been modelled by beam-elements. At first, structure was modelled without the back-wall cantilever. Though, horizontal displacement toward river, due to soil pressure, was too big, causing too high force in anchorage tendons, that it was necesary to model back-wall cantilever. The ship impact on the front wall was modelled by movable loads. This impact force was distributed on top edge of each field at 5,0 m length and moved down stepwise every1,0 meter. It was found that the ship impact has its unfavourable influence when applied to the top of end field of the structure, while it caused inpact and torsion of the structure as well.

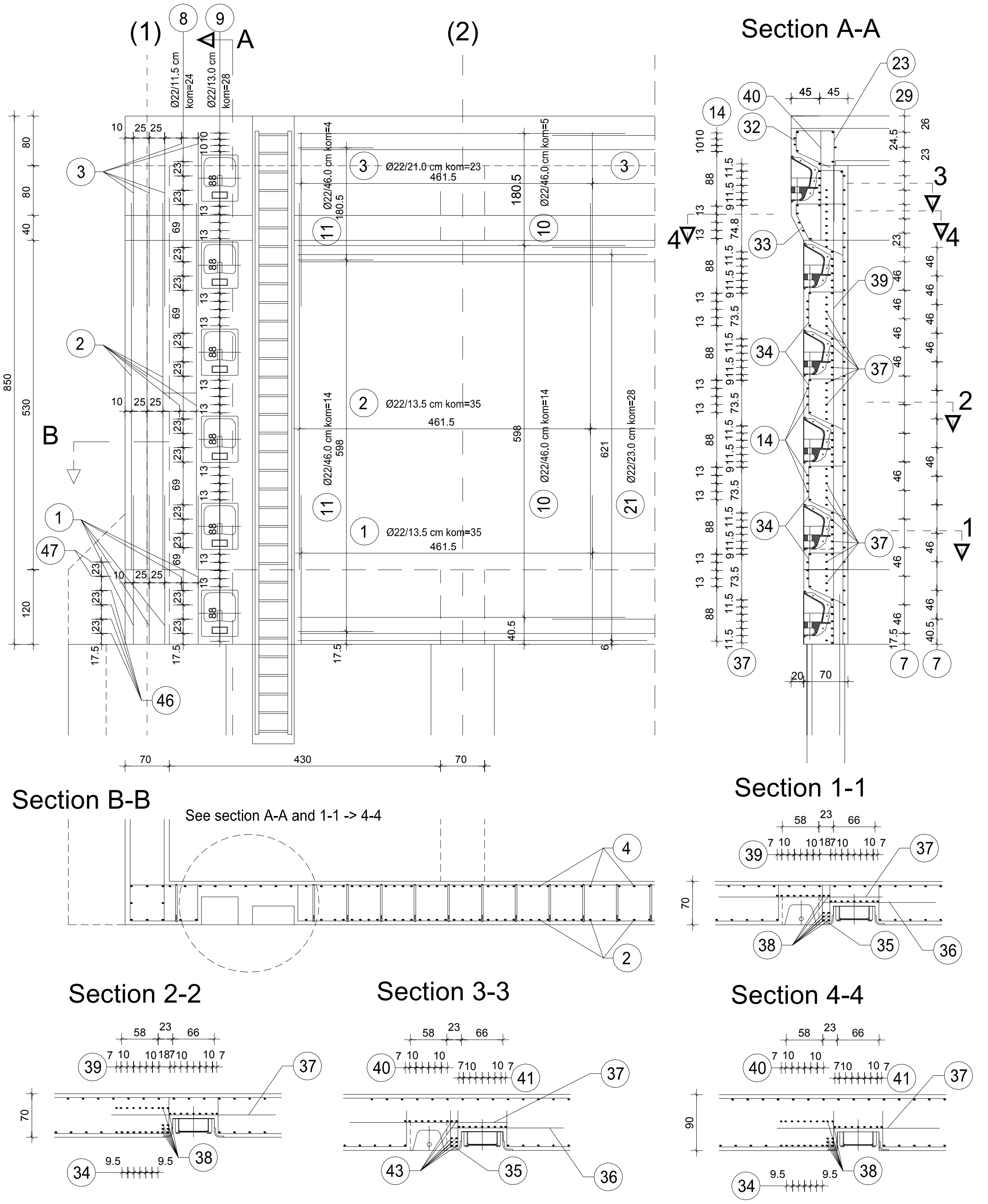

Figure 5 View and cross-sections of front wall with reinforcement

Figure 6 shows soil pressure loading on back-wall and cantilever as well as anchorage tie force. The mobile crane loading was modelled as a load of each of four legs by a group of three ertical forces in one line. Those forces were moved along the structure on different positions in order to induce maximum internal forces in structure. There were altogether 271 load cases.

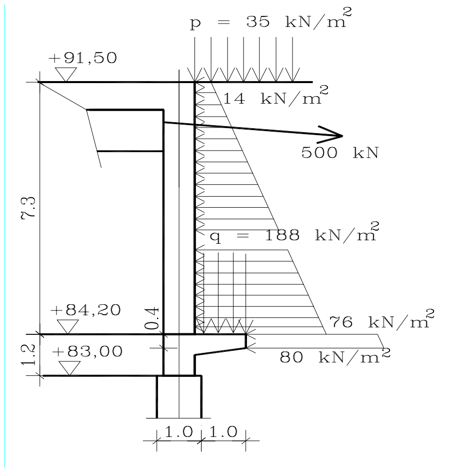

Figure 6 Soil pressure on back-wall and cantilever

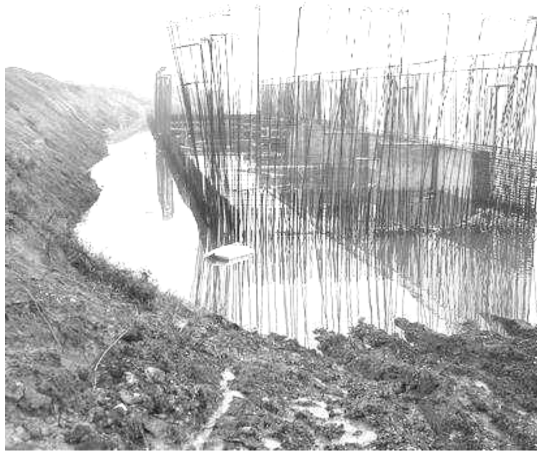

Figure 7 Flood, after front wall was erected (October 2003)

5. Construction

Construction of quay structure started by the end of the year 2001. Then it stopped for more than a year, to be continued in spring of the year 2003. At that time the construction of geotechnical diaphragms and piles was done. Erection of upper structure began in September 2003. Figure 7 shows state of structure flooded by high water in October 2003. Figure 8 shows the state of construction one month before flooding.

Figure 8 Construction of cross walls (September 2003)

Figure 9 Construction (November 2003)

Figure 9 and 10 show construction of the quay structure walls in November 2003.

Figure 10 Construction of west part of front wall at its half high (November 2003)

Figure 11 Reinforcement of walls (May 2004)

During winter time construction work was stopped, and continued in spring 2004. Wall framework and reinforcement of walls, in May 2004, is shown in figure 11.

Connection of prefabricated beams of upper grid with column is shown in figure 12.

Figure 12 Girders of upper grid (September 2004)

Figure 13 Reinforcement of upper slab (December 2004)

Thin prefabricated (6 cm) plates were then placed upon beams. Final reinforcement of slab was then placed on prefabricated slabs and beams, and connected with reinforcement from walls and girders. Figure 13 shows reinforcement of upper slab before concreting in December 2004.

6. Conclusions

Design and construction of ship landing stage is complex engineering task that needs good preparation and cooperation among several civil engineering experts. This type of structure is submitted to a great number of different influences, but 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. Construction of the quay structure is finished by the end of the year 2004, and opening for function will take place in summer 2005. All quay structure, with diaphragms, piles and upper structure, contains 6396 m^3 of concrete and 650 t of steel reinforcement.

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A Slipway Structure for River Ship Repair

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Josip Galić, Leo Matešić

Abstract: A slipway structure for ship repair discussed in this paper has been designed and constructed on the River Sava near the town of Sisak in Croatia. The structure consists of reinforced concrete bored piles and girders. In the river, girders are supported by a trapezoidal concrete block of 16x2 m placed in the river bed. From this block up, girders of 2x2 m in cross-section are inclined from the river to a certain bank height after which they are horizontal. For the predicted loads foundation soil was too weak, and for this reason each girder rests on 9 piles from 3 to 12 m in length and 1.0 m in diameter. The joint between the girder and piles is rigid, and thus they make a frame element. There are 11 girders, whose inclined and horizontal parts have a length of 41 m and 26 m respectively. The whole structure covers an area of 82x86 m. Ships are to be supported by trolleys travelling on an inclined rail system. The structure extends underwater for a sufficient distance to enable the ships to float on and off. The movement of trolleys together with a ship is controlled by hauling winches that are situated at the upper end of the slipway.
Keywords: slipway, ship repair, foundation, piles, concrete girders, reinforcement

1. Introduction

There are over 300 river ships in the Croatian merchant fleet of inland waterway navigation. The average age of these ships is more than 25 years. By law, they are supervised by the Croatian Register of Shipping (the Register). According to the Rules for Technical Supervision of Inland Waterway Navigation Ships laid down by the Register, a river ship must be pulled up on shore once every five years. In addition to compulsory ship pull-ups for inspection and checks, ships are also pulled up for repair of damage, suffered in service, which cannot be repaid in the water. It should be noted that the slipway discussed in this paper is the first to be designed and constructed in Croatia.

At present, only a small number of ships in the Croatian merchant fleet of inland navigation supervised by the Register are regularly repaired in the Slovenske Lodenice Shipyard in Komarno (Slovakia). Of all the other ships in the fleet, since 1990, only a few of them were pulled up onto dry land in Croatia, either on improvised slipways or by using the changes in the water level.

For the reasons mentioned above, there is a need for the construction of two or three slipway structures on the Croatian inland waterways that will be economically justifiable. There are three suitable locations for their construction: 1) two on the River Sava in the area of the towns of Sisak and Slavonski Brod, and 2) one on the River Drava or on the Danube in the area of the city of Osijek and the town of Vukovar respectively.

A study showed that the slipway structure on the River Sava at Sisak can attract at least 50 vessels. This number of ships can justify the construction of the slipway and provide employment for 70 workers.

The Sisak slipway was constructed as a 13^0 inclined slipways with the horizontal top part (Figure 1). It can hold inland navigation ships of up to 400 tonnes deadweight and 80 metres in length. On this structure ships will be pulled up from the river and lowered into the water sideways using double trolleys on rails, the one for a sloping part of the structure and the other for a horizontal part. The trolleys are pulled up and down by means of special motor driven devices, hauling winches, and ropes.

The inclined part of the slipway structure is expended at its top by the horizontal part of the structure extending towards inland. The horizontal part is also provided with supports and pulling system for sideways ship pulling. The slipway can accommodate three ships on the inclined part and two on the horizontal part at the same time. The structure design enables the pulling of ships from the river even at a low river level and holds ships high and dry even when the river level is high.

The shipyard site has a shape of a right-angled triangle with the hypotenuse on the river bank. The part of the bank occupied by the shipyard is about 320 m long.

The highest water level recorded at the Galdovo measuring station (located near the slipway structure) was 99.7 m above see level and the low water was at the level of 90.5 m with 95% of probability. An analysis of water levels measured over the period of last 10 years showed an average at 89.8 m. The maximum water level was at 99.0 m while the level of the river bottom at the site of the slipway structure is at 88.0 m and that in the waterway is at 85.5 m a.s.l. The elevation of the yard is approximately 100.0 m.

2. Foundation of the structure

Geotechnical investigation showed that the foundation soil consisted of four horizontal layers: clay, silty sand, gravel and silty clay. The depths of the layers and their descriptions are given in Table 1:

Table 1 Data on foundation soil

Layer	from [m]	to [m]	Type of material	Description
1	0.0	2.5	Clay	high plasticity, firm consistency
2	2.5	9.5	silty sand	loose
3	9.5	12.5	Gravel	dense, poorly graded
4	12.5	>15	silty clay	high plasticity, firm consistency

For the predicted loads the foundation soil was too weak and for this reason each girder rests on a set of nine piles. An analysis of foundation indicated that bored piles perform better when supported by the third layer (i.e. gravel) than when supported in deeper fourth layer (i.e. silty clay). Joints between the girders and piles are rigid, and thus they make a frame element. Figure 1 shows concreting of piles.

Figure 1 Concreting of piles

3. Description of the slipway structure

The slipway structure has shape of a rectangle having the length of 72 m and the width of 70 m. It consists of a horizontal part of 30 m in length at the elevation of 99.3 m and a sloping part inclined at 13^0 of about 53 m width. The lowest height of the latter part is 87.0 m above see level. Basic structural parts of the slipway are eleven reinforced concrete girders spaced 9.0 and 5.0 metres apart. The rails for trolleys/bogies are installed on the girders. The cross-section and the plan view of the slipway, together with pulled up ships are shown in Figure 2.

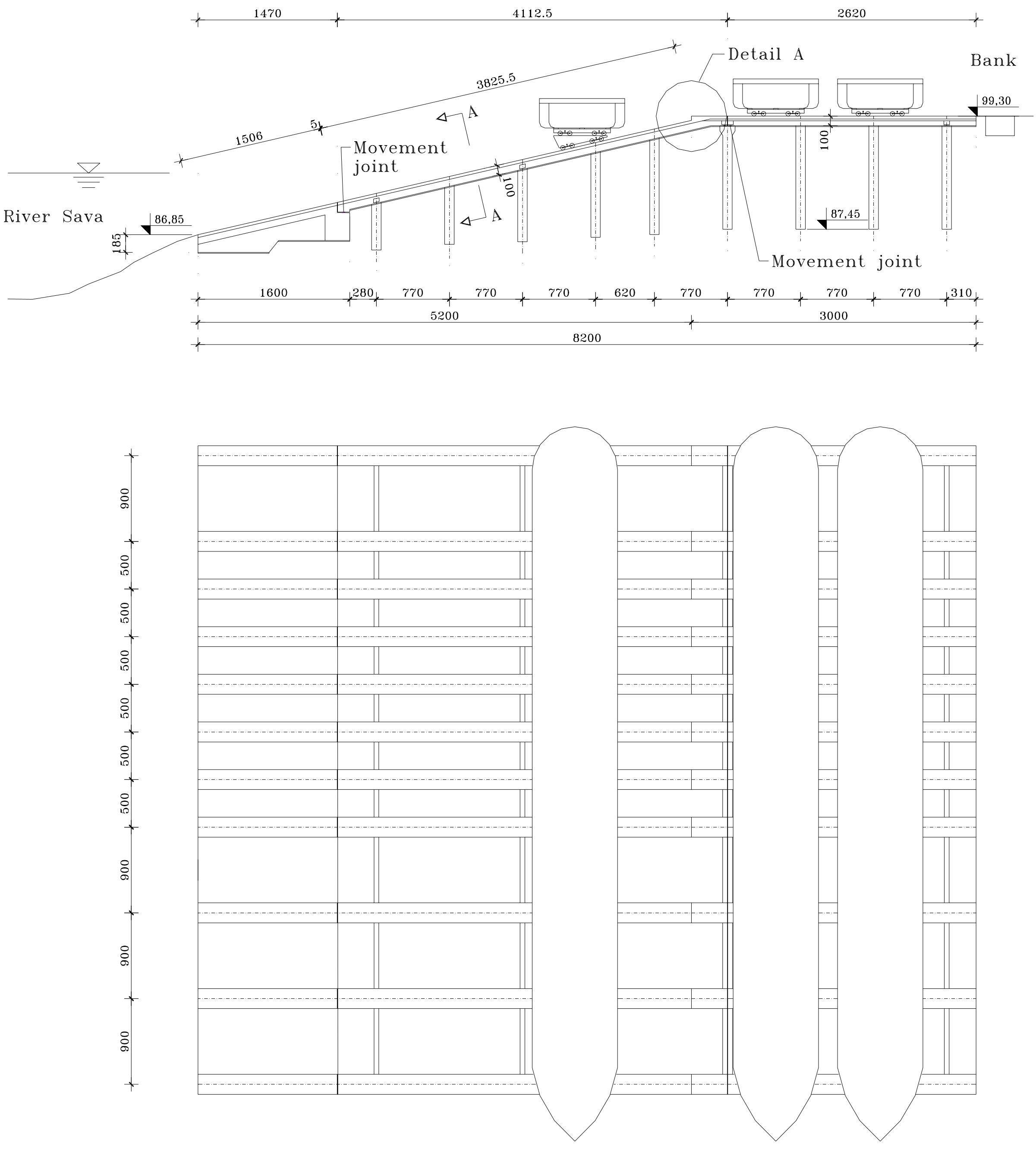

Figure 2 The cross-section and the plan view of the slipway structure

The foundation strips, that are underwater, have a width of 1.0 m. They are located at the elevations of 85.0 m and 86.25 m. The strips are constructed of reinforced concrete of concrete strength of 30 MPa, a concrete cover of 7 cm, and reinforcing steel of yield strength of 400 MPa.

The other parts of the slipway structures consist of eleven girders supported by bored concrete piles. Piles have 100 cm in diameter and are made of reinforced concrete of 30 MPa concrete strength and reinforcing steel of 400 MPa yield strength. Eight piles are spaced 7.7 metres apart and the ninth is 6.2 metres apart. They are bored down to the depths from 4.85 to 10.85 m. The total number of piles installed is 99. The inclined part of the structure under construction is shown in Figure 3.

Figure 3 Inclined part of the slipway structure under construction

The horizontal girders are shown in Figure 4. The reinforced concrete girders of the slipway structure have a T-shape cross-section. The width of the girders at the bottom is 170 cm and at the top 210 cm so that rails for the trolleys can be installed.

Figure 4 One horizontal girder completed and the other under construction

The shapes of the girder cross-sections are shown in Figures 5 and 6.

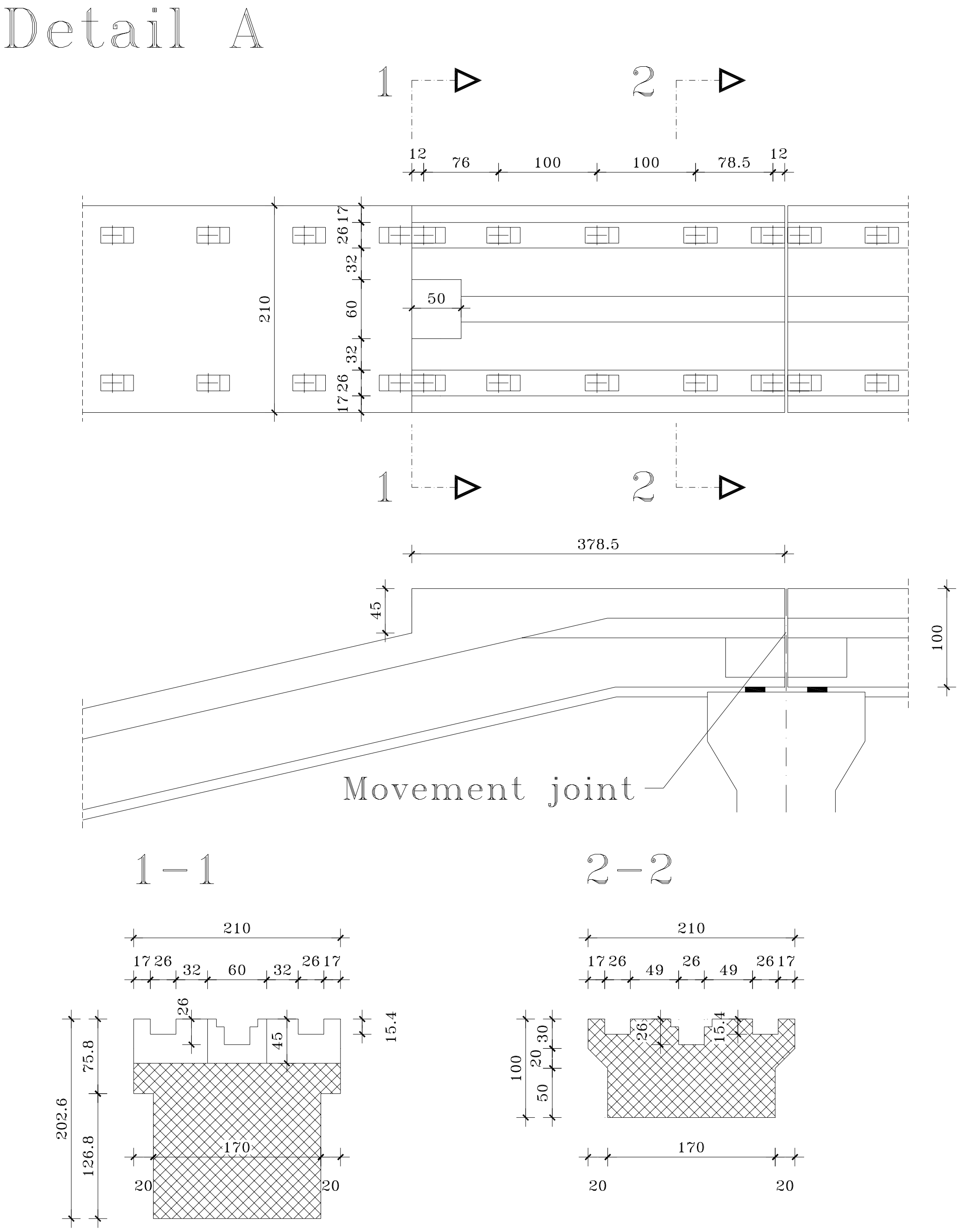

Figure 5 Detail A from Figure 2 and cross-sections of girders

The reinforced concrete girders were constructed of concrete having compressive strength of 30 MPa and of steel reinforcement with yield strength of 500 MPa. At expansion joints, elastomer bearings of 100x250x52 mm and 625 kN bearing capacity were placed. These elastomer bearings allow horizontal movements of the structure caused by different reasons such as temperature effects, loading forces, settlement etc. Between the girders, transverse beams of 40 by 40 cm cross-section, perpendicular to the girders, were constructed. These transverse reinforced concrete beams with compression strength of 30 MPa and with steel reinforcement having yield strength of 400 MPa are placed in five lines (see Fig. 2).

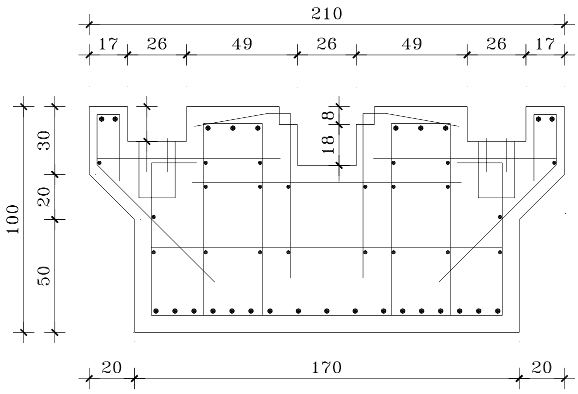

Figure 6 Cross-section 2-2 with reinforcement of girders shown in Figure 5

A working platform is at the height of 98.8 m above see level. The foundations for hauling winches are installed 4.20 m from the end of the horizontal part of the structure. There are ten winch foundations of 3.0x3.4x2.0 m. In the space between them and the slipway girders, concrete channels of 26 cm in depth and 120 cm in width were placed for hauling winch ropes to be used for ship pulling up from the water.

After compaction of the subgrade to the modulus of 50 MPa, reinforced concrete slabs between all girders were constructed.

A 2.4 by 5.0 by 2.0 m foundation block was constructed for a drive-motor for all ten hauling winches. The constructed slipway structure with trolleys is shown in Figure 7.

Figure 7 The constructed slipway structure with trolleys

4. Conclusions

The slipway structure designed for repair of inland water navigation ships was an interesting challenge both for designers and for constructors. Such a structure was needed for a long time, and this is the first slipway to be built in Croatia. It is planned to construct two additional similar structures on Croatian rivers in the near future. The entire slipway structure together with foundations, piles, girders and slabs contains 3606 m^3 of concrete and 2508 tonnes of steel reinforcement.

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Rolling Concrete Dams – Characteristics of Technology and Mechanization

Valentina Zileska-Pancovska
Zvonimir Vukelic

Abstract: Rolling concrete dam is type of dam that is built very often, because of its simple, economical and short term construction. Rolling concrete dams are not built in the Republic of Macedonia yet. Also, the theoretical and practical investigations in the fields relevant for rolling concrete usage for dam construction in the Republic of Macedonia are modest. Due to that, authors of the paper made some investigations about rolling concrete dams. The investigation results are presented in the paper. The results are connected with the technology of construction and with constructive machines at the rolling concrete dams.
Keywords: Dam, rolling concrete, technology of construction, constructive machines

1. Introduction

Rolling concrete dams, as their name says, are made up of rolling concrete (RC). It is a material which due to the small quantity of bindig material and water is more like ,”wet earth,” (4) than like massive concrete. It contains the same basic components as with the conventional concrete: aggregate, binding medium, water and additives (in case of need). Mutual proportions of those components differ not only in comparison with the conventional concrete, but they differ with particular rolling concrete mixtures, too.

Quality, quickness and thriftiness of rolling concrete dams construction directly depend on the quality study and on the regular selection of technology and construction mechanization. That is the reason for their significance and the attention to be paid.

2. Selection of technology and construction mechanization with rolling concrete dams

2.1 Loading rolling concrete into transport means and internal transport

The wet concrete mixture from the concrete preparation machines is loaded into transport means. The selection of the transport means related to the LC internal transportation depends on several factors among which the most significant are as follows:

- disposable mechanization and equipment;
- topographic condition of the site and the possibility for access roads construction;
- quantity of material to be built in;
- the size of the maximum bead of aggregate etc.

Due to the small quantity of water (up to 40% less than in comparison with the classical concrete (4), the rolling concrete is very sensible to loss of moisture as a result of evaporation. For that reason it has to be transported and built in very quickly and at most 30 minutes after the dosing of the mixture components, that is up to 15 minutes after the laying (4). Due to that, it is necessary to provide for a quick vehicle manipulation, without stoppage. It is possible through a regular broad and close selection of mechanization and by an appropriate site arrangement of the machines.

Construction machines which can be used in RC internal transportation are the machines used in the earth mass transportation as well as the machines for conventional concrete transportation. RC internal transportation construction machines used for the most part are as follows:

- Vehicles out of public roads: dumpers, dischargers through the bottom and rear dischargers-tippers
- Transportation belts;
- Cranes;
- Scrapers;
- Concrete pumps and
- Combinations of machines.

2.2 Rolling concrete laying

RC mixture laying is in layers, coast to coast – without joints. **Scrapers, dosers and graders** have been used for rolling concrete laying. Laying has been mostly made by dosers which quickly and equally lay the RC, that is the machines provide for regular levelling of the wet RC concrete mass. The laying is in layers and the thickness is mostly d=15-35 (40) cm. Ofenly (according to the Japanese technology) there are several layers together of maximum thickness d=50-100 cm (2).

2.3 Building in rolling concrete

There are mainly *two technologies* in RC laying, as follows:
a) Technology of building in RC layers of small thickness;
b) Japanese technology in building in RC.

a) Technology of building in rolling concrete in layers of small thickness.

According to this technology, RC laying is in layers of small thickness (about 30 cm). Layers have been made continuously, coats to coast, and each layer is separately packed up to the full compactness. Layers of this thickness are made with the three types of mixtures (2): poor dry mixture, fatty wet mixture and medium- wet mixture.

b) Japanese technology of building in rolling concrete

According to the Japanese technology, the thickness of the layers to be layed is from 50 to 100 cm and the performance of each layer consists of laying 3 to 5 equal layers d= 15-:35 cm which are packed together. On the packed layers there are joints from the upstream to the downstream side of the dam and compulsory there is a layer of conventional concrete d= 2,5 - 3 m (1). Thos layer is made by means of panels and it is considered that its performance will enable a good RC waterproofing, that is the filtered water will be reduces, particulary on the joints between the layers and there will be a better protection of RC from freezing (in the regions of height above sea level and severe climate). For the purpose of better binding between the layers, floors have been mutually connected by a layer of mortar, d= 2 cm (1), which is to be layed on the already packed layer immediately prior the start of building in the next layer.

According to the Japanese as well as according to the other technology of building in RC, the building in RC should be in optimum wetness mixture. Optimum wetness is the mixture wetness with which the accepted type of roller and the accepted thickness of the layer there is maximum compactness of the mixture (5).

Building in RC is made by means of vibrating rollers. The choice, that is the acceptance of the type of roller is in a close mutual dependancy on several factors of which as more significant we can point out: characteristics of the RC mixture, first of all, the maximum size of the aggregate bead - Dmax, the thickness of the floor, degree of the prior consolidation of the wet RC during the laying, disposable mechanization, disposable working front, the volume of work to be performed etc. In packing RC layers of bigger thickness, always, the first packing or the first two, are without vibrations. It makes possible for the top of the RC layer to be consolidated. The usual speed of movement of the vibro-rollers is 0,5-1,0 km/h. Rollers with pneumatic wheels (compactors) are usually used in completion of packing.

The RC packing should be made at last 10-15 min after the laying, that is 30-40 min from the beginning of the mixture mixing (3) (4). It means that the building in RC should be performed continuously, that is the time period between the building in of the last two RC layers should not be longer than 8 hours (1). For that reason, in building in RC there should be two or three shifts. If the time period of 8 hours in building in the last two RC adjoining layers can not be reached, in that case those layers should be worked out with a layer of mortar or concrete.

The most usual practical norm reached in building in RC is Up=100-150 m^3/h (4).

As previously mentioned, the building in RC is in layers of various thickness, depending on the applied technology. Those layers mostly are not made as horizontal, that is they have a slight gradient 1:15 or slighter, for example, 1:20 towards the downstream or towards the upstream side (2). In that way you enable drainage of the surface of each layer during the construction and you improve the resistence against sliding in the joints between layers of the performed body of the dam.

During the building in RC a special attention should paid to: the joints of RC with reinforced or conventional concrete, to joints with the prefabricated concrete panels, as well as to the joints of various types of mixtures of RC in the dam body.

Also a particular attention in building in RC should be paid to the contact of RC with a rock in order to minimize the cracks in the RC. Mostly contacts have been worked out with mortar or with concrete (5). Also you should take into account the air temperature and the temperature of the concrete mixture (whose optimum value is about 18°C (3)).

As to the RC building in the dam body, we must mention that it can be performed in two phases, so that the built in RC in the Ist phase serves as a protection of the construction pit during the building in RC in the second phase.

2.4 Making dilatation joints

The following operation follows in building in RC in a layer of a certain thickness- making dilatation joints. Their performance is in order to reduce the temperature stress and strain in the dam body.

They are performed in the following way: firstly, the RC layer is to be cutted by means of *vibrating cutter* (a special machine for cutting joints) from the upstream to the downstream side of the dam, and then plates should be inserted into the joints. Plates can of various material and of various sizes. With the largest number of constructed dams, those plates are made up of geosynthetic materials or galvanized steel plates, and their dimensions with various dams have been different (for example, 20 cm (2) or 90x60 cm (3)). Those plates should be placed vertically on appropriate places in each layer of RC. The distance between the is different with the already constructed dams. At the beginning, with the dams constructed according to the Japanese technology, the distance between them was 10 to 15 m (5). The joints were reduced with the later constructed dams. The distance between those transversal joints with the larger number of constructed dams is approximately 70 m, because it is proven by an analysis that through the reduction of that distance there is no a more significant impact on the reduction of the thermal stress (2).

Dilatation joints are spreaded from the upstream to the downstream side of the dam, and, they, in fact, are weakened vertical plains through the whole section of the dam. They should be well closed on the upstream side of the dam in order to prevent possible filtration of the water. Mostly their closing is by means of a system containing membranes and steel stainless plates. (2).

2.5 Cleaning and wetting the surface layer

In order to provide a good binding and at the same time a good waterproofing between the horizontal layers, it is necessary to pay a special attention to the joints between the two consecutive layers of RC, that is, if the layer of RC is not covered by the next layer prior to its binding, that layer is a cold, that is unplanned working joint. Those joints, that is horizontal construction joints, can be weak points of the dam. For that purpose it is necessary to treated regularly. Their treatment consists of their cleaning, wetting, and in case of need laying a bed layer.

Road cleaning machines were used for cleaning those joint with the largest number of constructed dams, but later there were efforts to develop and construct *special vehicles for cleaning concrete surfaces of RC.*

Wetting of concrete areas is performed by means of cisterns and water jets. After the cleaning of the horizontal construction joints, it is necessary to make possible for the free water to be removed prior the start of the mortar placing.

2.6 Placing foundation for laying of the next layer

In order to provide for a good waterproofing between the RC layers as well as to ensure the necessart hardness of tightening and pulling down, it is necessary to pay a particular attention to the time of exposure of each floor to external impacts (particularly at very high and very low temperatures which depends on the joint maturity). Depending on the length of that time period a decision is to be made whether it is necessary to place a foundation layer between two neigbouring layers of RC.

If that time is longer than 8 h, and very often less than 8h, the provision of a good binding between the layers is possible only if there is a foundation layer. The foundation layer is mostly made up of cement mortar, but also of concrete (of mixture with bigger aggregate and bigger quantity of binding medium - cement). The thickness of the mortar layer is 1,5-4 cm and mostly 2 cm (3).

That foundation layer between the layers of RC serves a s a type of “adhesive” related to the binding of the RC layers.

Placing of the foundation layer is mostly made by means of *pneumatic loader* and manually. Its placing requires an appropriate control, takes much time, is expensive and contrary to the objective for quick and economical construction of RCD.

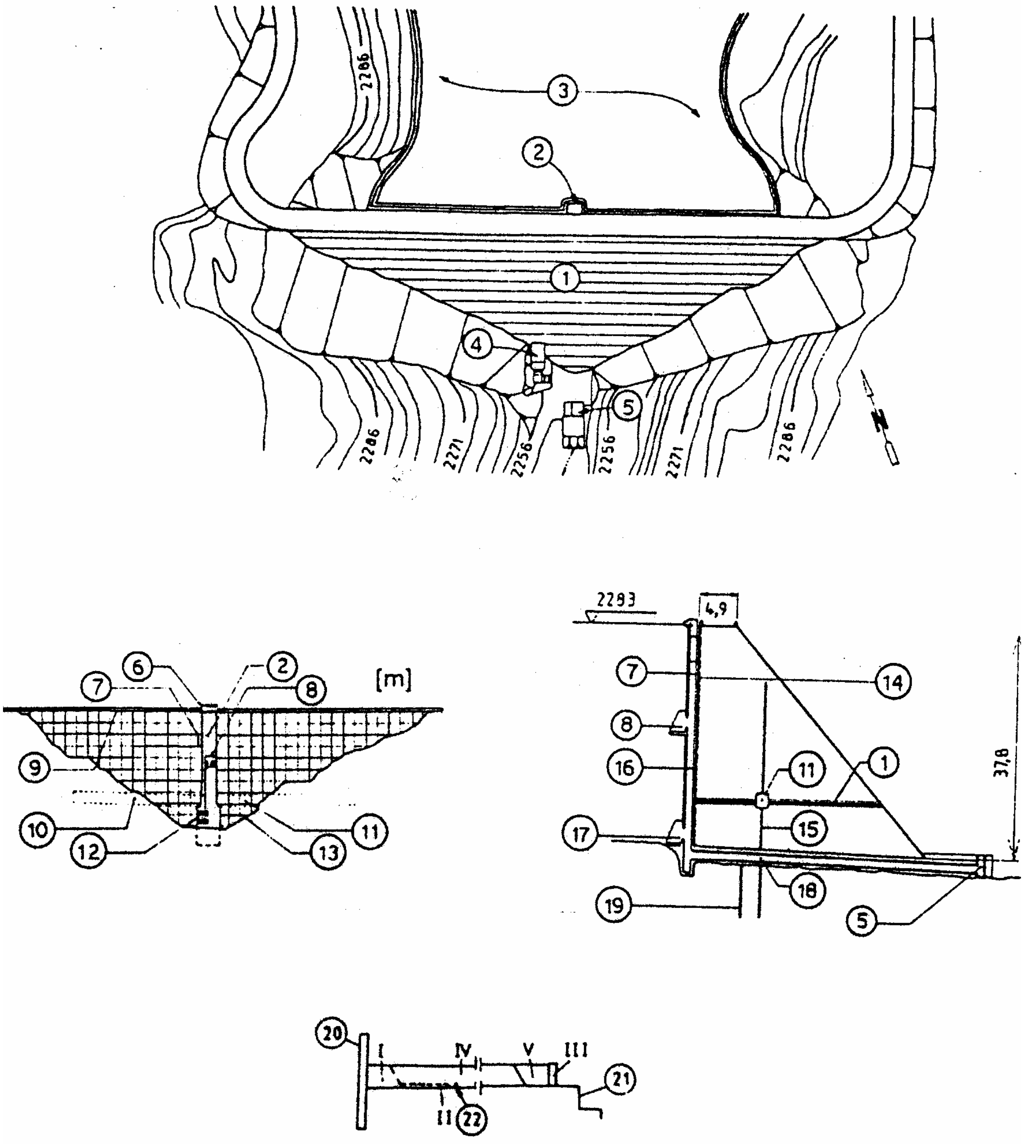

Figure 1 Dam Middle Fork (USA)
1- body of rolling concrete dam, 2- overflow and outflow structure- entrance structure,
3- normal level, 4- access to the drainage gallery, 5- exit construction, 6- higher and 7- over overflow, 8- outflow, 9- concrete fence, 10- drainage tunnel, 11- drainage gallery, 12- outflow during the construction, 13- dam foundation, 14- stoppage of concreteing because of drilling, 15- drainage holes, 16- side contrary to the water, 17- drainage entrance of the construction waters, 18- overflow and outflow pipes, 19- injection curtain, 20- panels of the side contrary to the water, 21- ruggedly formed downstream gradient, 22- drainage pipe, I-IV construction phases.

2.7 Controls during the dam body construction

In order to provide for a high quality construction of RCD, it is necessary to perform several controls during the technological process. Because RCD have been a new type of dams, besides the application of the old methods and techniques, it is necessary to develop and apply new methods, techniques and equipment by means of which the results during the dam construction can be achieved quickly and simply. Among the more significant controls, the following controls should be pointed out:

- Consistence of the RC and of the optimum wetness of the RC mixture;
- RC mixture components;
- Thickness of the placed and built in layer of RC as well as of the density of the built in layer of RC;
- The reached quality of the built in RC, particularly of its hardness to pressure;
- Foundation layer between the floors of RC (quality, thickness etc.);
- Cracks;
- Filtered water;
- External temperature and the mixture temperature;
- Foundation etc.

Conclusion

As a result of the various mechanization which can be used in the construction of rolling concrete dams, as well as due to the various technologies of construction, there are their numerous possible combinations. For that purpose, in order to choose the optimal construction technology and mechanization, it is necessary to make appropriate technological-economic analyses.

It means that through studious research in the construction technology and mechanization you can make a rational selection. In that way you can make possible: gradual including the construction machines in the work, elimantion of the part of causes for numerous stoppages in their operation and their equal utilization. It can lead towards a large paralelization of works and reduction of the unite product price. At the same time you can improve both the quality and the speed of the rolling concrete dams construction and their more quickly putting into operation, that is it cam be said that the regular selection of technology and mechanization of construction is the basic precondition for an effective construction of complex engineering structures - rolling concrete dams.

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Theme IV

Sanitary Engineering and Sustainable Water Use

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Implementation of Passive Air Pump in Wastewater Treatment

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Abstract: In the processes of biological treatment of wastewater, oxygen concentration plays a significant role. In conventional mechanical technology, oxygen is being added by use of the mechanical equipment for air pumping. In the constructed wetlands, oxygen is being infiltrated into water through water surface, as well as through plant roots. In this paper the results of the aeration by use of passive air pump (PAP) consisting of a vessel filled with coarse gravel granulation, in which water flows in continuously and is being discharged discontinuously by use of a siphon have been presented. The passive air pump is to be installed between pre treatment (settling tank) and wetlands in which wastewater is being treated biologically. In this way disposable potential energy can be used for enriching water with oxygen needed for biological processes. In the performed research, aeration quantity in dependence on granulation size, siphon level, retention of water in PAP and discharge has been measured. The research has shown that it is possible to benefit from terrain decline for water aeration by using the passive air pump. In that way 2.5 - 3.5 mg/l of oxygen can be added which can improve processes of wastewater treatment.
Keywords: biological treatment of wastewater, passive air pump,

1. Introduction

In order to test the efficiency of the constructed wetland for wastewater treatment and to estimate the feasibility of passive air pump application in enriching wastewater with oxygen two models were made. The first one - the laboratory model was focused on the possibility of injecting oxygen into water by use of PAP, and that model will be described in the paper. The other model was made as a pilot model next to the city collector, in the vicinity of the location of the conventional system for wastewater treatment, and consists of three equal parallel lines through which the same quantity of wastewater flows. In that way the efficiency of PAP, various plants and other parameters influencing the efficiency of wastewater treatment by use of constructed wetlands could be estimated. The testing of the pilot model on the spot is still going on, so the test results will be published subsequently.

The constructed wetland costs are much less (50% to 90%) than those of the conventional systems and their operating costs are very low. They require little or no energy to operate.

In the constructed wetlands, organic matter is removed through biodegradation. Biological treatment is in most cases aerobic with oxygen supplied by both diffusion from the atmosphere at the surface of the beds and by leaking of oxygen from the roots or bulrushes reeds and other emergent aquatic plants. Aerobic treatment processes are faster than anaerobic processes, but oxygen is limited within the wetland. The aim of this research is to test the possibility of increasing the quantity of oxygen injected by use of PAP.

2. Description of the PAP Laboratory Model

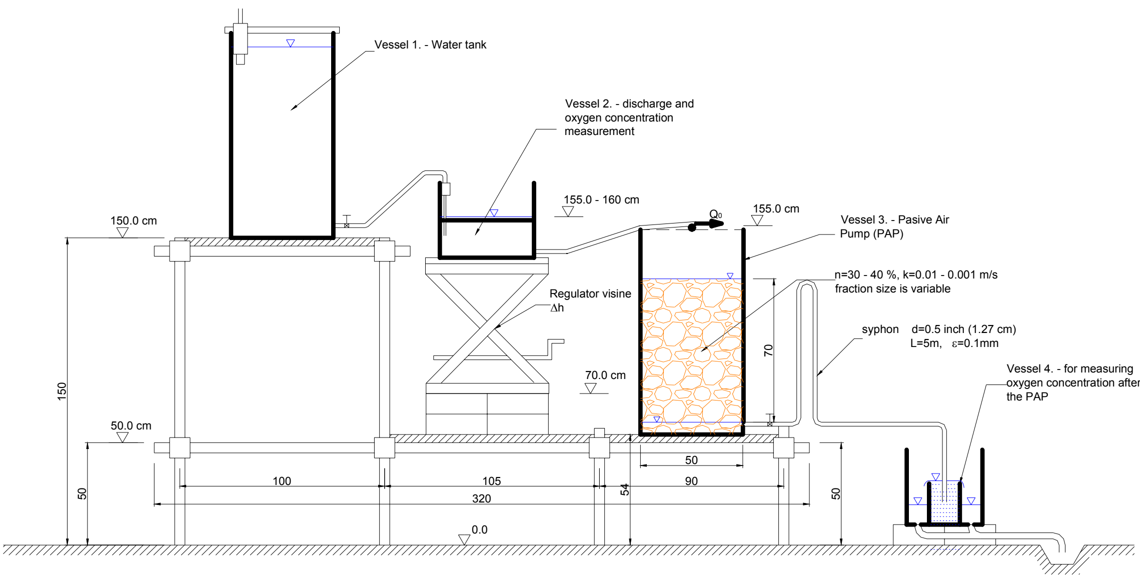

Fig.1 Sketch of the PAP laboratory model

In the laboratory model the possibility of oxygen infiltration into the wastewater by means of the passive air pump (PAP) was tested, in order to improve the efficiency of the constructed wetland for wastewater treatment. The functioning of the constructed wetland is based on the capability of bacteria in substratum to decompose organic matters in water that flows through it. As oxygen is needed for such decomposition one of the options would be the above described passive air pump. The principle of work of the passive air pump is based on continued injecting of wastewater into nonsaturated porous setting and its discontinued draining by the use of a siphon. The increased oxygen concentration in outflowing water is obtained by the processes of diffusion and mixing when the water flows through the nonsaturated porous setting.

The measurements carried out for different granulations of air basin filling material, different filling and emptying cycles, and different levels of siphon and flow through the device (apparatus). The change of one parameter causes a change of oxygen concentration at the exit of the apparatus. The aim of these laboratory tests is to find such a parameter combination that enables maximum infiltration of oxygen into water. All measurements were carried out with clean water in order to avoid influence of uneven load of water injected into the model.

The laboratory model was made to the scale 1:1, that made possible direct application of the obtained results on the on site conditions. The laboratory test device consists of four vessels connected with plastic tubes, and of auxiliary equipment by which desired conditions have been achieved (schematic description presented in the Fig1).Model components are the following:

- Vessel no. 1 - with the water injected in the model
- Vessel no. 2 - has got two major goals: to achieve the desired flow Q_0 and to measure oxygen concentration in water just before entering PAP.
- Vessel no. 3 - Passive Air Pump (PAP). It consists of a vessel with circular cross-section, made of acrylic glass filled with gravel of a specific granulation. The water is being injected into PAP by dripping from another vessel (vessel for flow regulation and measuring of oxygen concentration before entering PAP) through a plastic pipe that ends at the very top. When the PAP is filled up to a certain height, the siphon is being activated, and the PAP begins to discharge. At the moment when the water level drops to the exit elevation, the siphon flushes the air and deactivates. After that the PAP begins to fill again, i.e. the new cycle gets started.
- Vessel no. 4 - for measuring oxygen concentration after the PAP. The goal of this part is the model is to collect water after it passes through the PAP in order to measure oxygen concentration.

When making a laboratory model of PAP, an acrylic glass vessel with a circular cross-section diameter ϕ=48 cm and height h=100 cm was used. From the height h_s=77 cm onwards it was filled with gravel a defined granulation. In all experiments water was injected by dripping from the top of the vessel. For siphon a plastic tube of length L=200 cm and diameter ϕ=8 mm was used.

The testing of the efficiency of the PAP innovative technology was carried out for three different granulations. The grain granulation of 4-8 mm was named G1, grain size 8-16 mm was named G2 and from 16 to 32 mm G3. Geotechnical porosity of all samples is ca. 40%.

Possibility of the increased oxygen infiltration at the periodical PAP exhausting was tested for 4 siphon levels. The option in which water was just filtered through the PAP (ΔH_{nat}=0 cm) was tested, and also with the level oscillations of ΔH_{nat}=30 cm, ΔH_{nat}=50 cm and ΔH_{nat}=70 cm.

The tests were made for four different flows (Q_1=800 l/day, Q_2=1200 l/day, Q_3=1500 l/day and Q_4=2000 l/day). By changing the flow, cycle duration and PAP load also change. The experiments were carried out with plain water, i.e. water from the water supply system.

3. Test Results

From the obtained data values of the increased oxygen concentration in water were calculated ΔC (mg/l O_2). Values of the changed concentration were expressed in dependence of the changeable parameters: flow, siphon level and granulometric composition.

In the table 1 the results of the measured oxygen concentration changes are shown (Δc_{sr} – mean value of the concentration increase, ΔC_{end}- increase at the end of siphon operation) represented in dependence on the flow (Q) for every single granulation (G) and siphon level (ΔH). From the table it is immediately obvious that the increase of oxygen concentration at the end of the cycle (ΔC_{end}) is somewhat higher than the mean value (Δc_{sr}).

Granulation	G1 (4 - 8 mm)				G (8 - 16 mm)				G3 (16 - 32 mm)			
DH (cm)	Q (l/day)	T (min)	DC_{sr} (mg/l)	DC_{end} (mg/l)	Q (l/day)	T (min)	DC_{sr} (mg/l)	DC_{end} (mg/l)	Q (l/day)	T (min)	DC_{sr} (mg/l)	DC_{end} (mg/l)
0	790	0,00	2,98	3,05	790	0,00	3,31	3,40	790	0,00	3,51	3,46
	1220	0,00	3,08	3,06	1220	0,00	3,30	3,36	1220	0,00	3,49	3,47
	1520	0,00	3,06	3,05	1520	0,00	3,32	3,31	1520	0,00	3,39	3,42
	1940	0,00	3,04	3,08	1920	0,00	3,26	3,26	1940	0,00	3,35	3,37
30	790	52,08	2,97	2,96	790	48,63	3,09	3,11	790	49,63	3,26	3,28
	1200	39,54	3,05	3,09	1220	37,13	3,13	3,16	1200	38,79	3,30	3,30
	1520	34,96	2,77	2,84	1520	34,63	2,98	3,00	1550	35,63	3,35	3,36
	1940	51,63	2,81	2,88	1920	34,46	2,91	3,02	1900	36,17	3,29	3,35
50	790	74,78	2,87	2,92	790	78,96	3,02	3,13	790	78,33	3,22	3,29
	1220	57,25	2,95	2,98	1220	59,50	2,93	3,05	1220	59,67	3,18	3,32
	1530	51,63	2,77	2,84	1520	53,38	2,98	3,06	1510	51,96	3,08	3,17
	1940	50,74	2,79	2,91	1920	51,67	2,84	2,96	1940	52,13	2,84	3,12
70	790	104,79	2,53	2,67	790	105,00	2,91	3,00	790	98,95	3,12	3,30
	1220	76,00	2,66	2,81	1220	78,38	2,91	3,10	1240	77,11	2,92	3,18
	1510	70,38	2,72	2,92	1520	70,25	2,70	2,96	1550	69,39	2,76	3,04
	1900	66,33	2,63	2,88	1920	66,71	2,56	2,82	1900	66,83	2,76	2,90

Table 1 Results of the measurements of oxygen concentration in water after passing through PAP represented for each siphon level and granulation depending on the flow

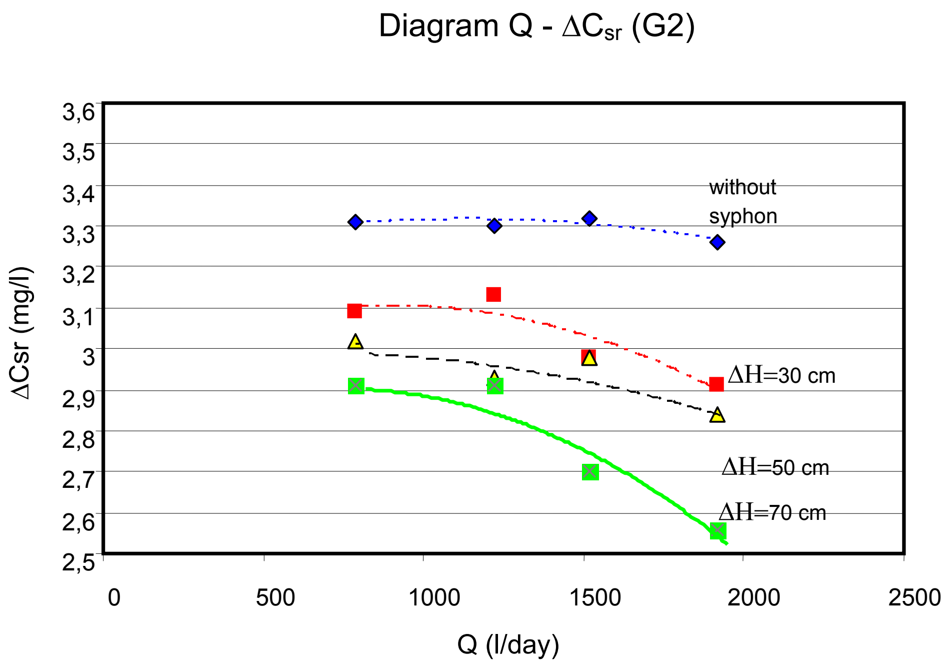

Illustration 2 Comparison of the mean value of concentration change for particular siphon levels depending on the flow at fraction G2

From the diagram in the illustration 2 it is obvious that by decreasing the flow through PAP its efficiency is increased, that is oxygen infiltration into water becomes enlarged. At the flow *Q=1220 l/day* its efficiency reaches the amount that does not change with further decreases of flow, i.e. it keeps approximately the same value.

From the pattern of the curves representing particular levels of siphon it is visible that the most efficient PAP is that without a siphon, that is when the water filtrates freely through the gravel.

In the illustration 3 a comparison of mean value of oxygen concentration increase depending on the level of siphon is shown.

From the above represented diagram it could be concluded that with the increase of siphon level efficiency of the passive air pump is decreased (PAP). The difference of the oxygen infiltration for the PAP without siphon and with siphon *H=70cm* high varies between 0.40 and 0.60 mg/l O_2 that amounts to approximately 20% of total oxygen infiltration by means of PAP, therefore it could be concluded with sufficient certainty that the best results are obtained when using PAP without siphon.

In the illustration 4 diagrams of oxygen infiltration dependence on cycle duration (charging + discharging) of PAP are shown. From the diagram it is marked that with the prolongation of cycle efficiency of the passive air pump (PAP) is diminished. The conclusion would be that it is more favourable for water to stay in PAP as briefly as possible, i.e. best results were obtained at plain filtration of water through the gravel.

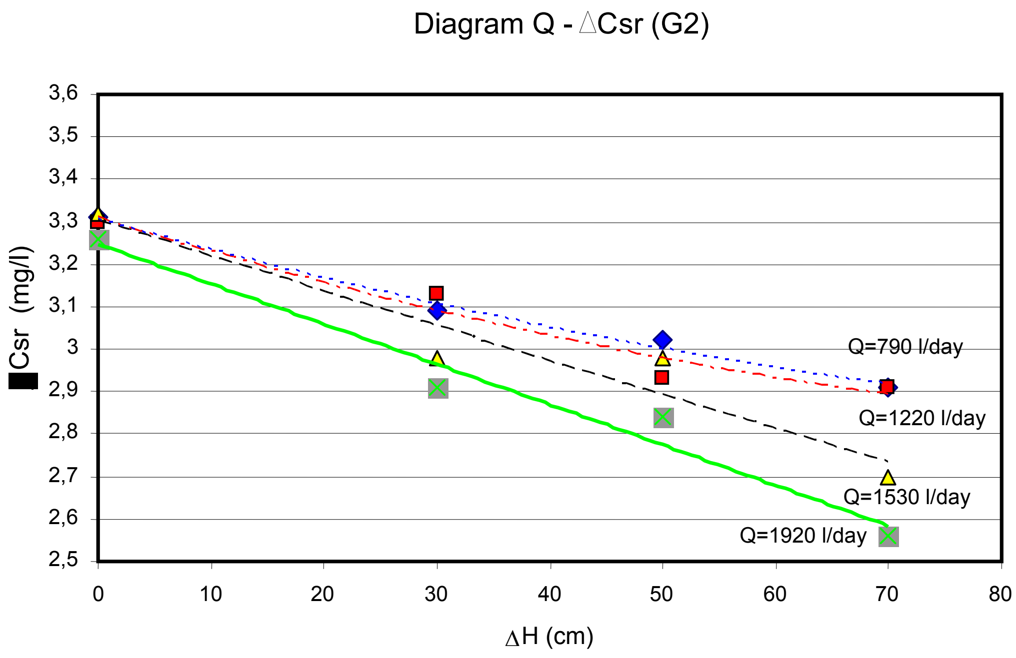

Illustration 3 Comparison of the mean value of concentration change for particular flows depending on the siphon levels at fraction G2

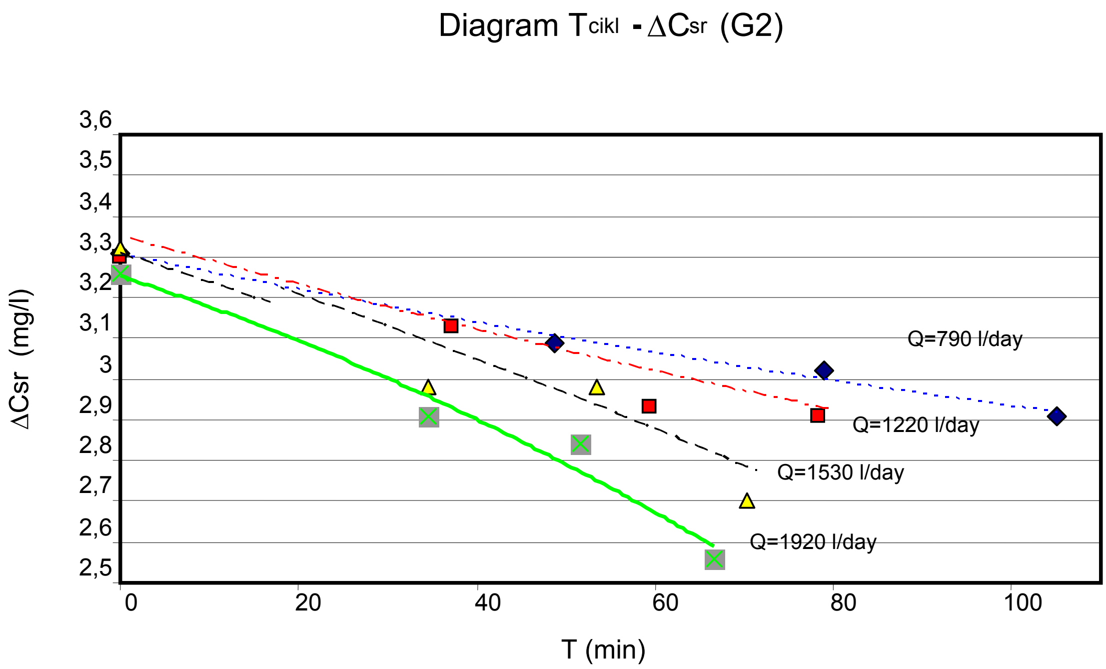

Illustration 4 Comparison of the mean value of concentration change for particular flows depending on cycle duration at fraction G2

If considering the flow, at short cycle duration (T_{cikl}<40 min) there is no major difference in PAP efficiency depending on the flow. From the diagram it is obvious that flow curves are concentrated in the narrow zone and that oxygen concentration varies ca. 0.15 mg/l. The more significant differences of results depending on the flow are noticeable at the cycles longer than $T_{cikl}=40$ *min*. Based on the mentioned it could be concluded that at shorter cycles efficiency of the PAP is not reduced significantly if the flow is increased, while in the experiments with longer cycles a considerable dependence of PAP efficiency on the flow could be noticed.

In the illustration 5 there is a graphic representation of PAP efficiency dependence with a certain granulometric composition on the siphon level and on the flow. Each curve represents PAP with gravel filling of a particular granulometric composition.

Analysis of the obtained results shows that after passing through the PAP, at every combination of flow and siphon level, granulation G3 (16 – 32 mm) provides the best results, while the worst are provided by the finest granulation G1.

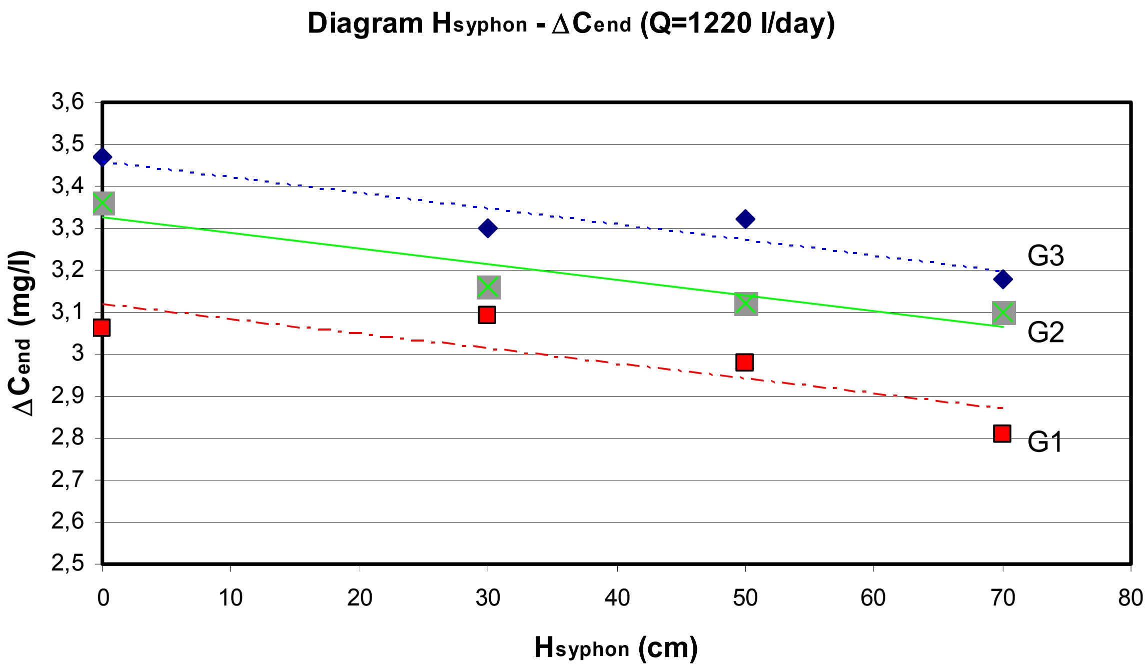

Illustration 5 PAP Efficiency for Different Granulations Depending on the Flow

4. Summary

In the article the results of feasibility tests of oxygen injection by means of passive air pump (PAP) have been described. The tests were carried out on the laboratory model with the basic goal to improve constructed wetland function.

On the laboratory model the injection of oxygen depending on substrate granulometric composition, flow and siphon level was measured.

Measurements on the laboratory model showed that best results were obtained for PAP filled with substrate of G3 (16 – 32 mm) fraction, and worst with substrate of finest G1 (4 – 8 mm) fraction. The difference between oxygen injection at G1 fraction and G3 fraction is on average ca. 0.5 mg O_2/l of water for equal flows and siphon levels.

For all substrate fractions and flows it is obvious that the best results of oxygen injection were achieved for PAP without a siphon, i.e. when the water was filtrated freely. However, it must be stressed that the laboratory tests of PAP were made in separated part of the apparatus, that is the interaction of PAP and constructed wetland basin was not observed. Consequently the complete results will be obtained on the pilot constructed wetland.

In case of flow variations a certain dependence of the injected oxygen quantity on the flow for all substrate fractions in PAP could be noticed. It could be concluded in general that by diminishing the flow up to the 1200 l/day quantity of the injected oxygen is increased, while for lesser flows no significant changes were noticed. On the whole it is evident that the efficiency of PAP is higher with lesser flows.

From the above mentioned conclusions it is obvious that for the tested PAP best results were reached with substrate of 16 – 32 mm fraction, without a siphon, at a flow of approximately 1200 l/day and lesser.

A series of experiments and measurements showed that the efficiency of passive air pump (PAP-a) is very high for oxygen injecting into wastewater. From the given results it is obvious that when the water passes through PAP the oxygen concentration is increased by ca. 3 mg/l. Conventional mechanical devices add ca. 8 mg/l, but call for substantial expenses for purchasing, installing, maintenance and operation

At present a pilot constructed wetland is under construction, on which feasibility tests of various passive air pump and siphon applications would be carried out.

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Chlorine Dioxide Disinfection by Products in the Nová Bystrica-Čadca-Žilina Long Distance Water Supply System

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Monika Karácsonyová, Ľuboslav Gajdoš

Abstract: In a process of water disinfection it is necessary to distinguish between primary disinfection focused on removal or inactivation of microbiological contaminants from raw water and secondary disinfection focused on maintenance of residual concentration of disinfector in distribution system. Current practice related to disinfection follows two approaches. The paper presents results from stage task solution “Research of physical-chemical changes in water quality during its distribution” at the Nová Bystrica-Čadca-Žilina Long Distance Water Supply System (LDWSS) focused on presence of disinfection by-products by using chlorine dioxide.
Keywords: Disinfection, chlordioxide, by product, water treatment

1. Introduction

In a process of water disinfection it is necessary to distinguish between primary disinfection focused on removal or inactivation of microbiological contaminants from raw water and secondary disinfection focused on maintenance of residual concentration of disinfector in distribution system. Current practice related to disinfection follows two approaches:

- Maintenance of residual concentration of disinfector and effort to find methods for achievement of acceptable concentrations of disinfection by-products.
- Search for new methods of sound management of distribution system without maintaining the residual concentration of disinfector.

In some countries it is required or recommended to maintain residual concentration of disinfector in order to meet limits of microbiological parameters, minimize bio-film formation, prevent from secondary contamination in distribution system and indication, whether contamination occurs as results of disinfector concentration reduction. Other countries follow the opinion that residual concentration of disinfector is not required for good water quality, microbiologically safe groundwater or for some surface water resources treated by multilevel technologies. Another reason is a minimization of disinfection by-product formation and reduction of risk connected with their formation, respectivelly. The residual concentration is not required, because water is treated in order to minimize amount of organic substances entering distribution system. There is an opinion followed that maintenance of residual concentrations of disinfectors in distribution system does not prevent from any significant water contamination and significant decrease or elimination of residual concentration of disinfector is not considered as reliable indication of contamination presence. Changes in pH value and water conductivity might be such indicators. Other authors give reason for that there is no reliable indicator of contamination and the most reliable measure preventing contamination is thorough proposal, operation, maintenance and control of distribution system.

The paper presents results from stage task solution "Research of physical-chemical changes in water quality during its distribution" at the Nová Bystrica-Čadca-Žilina Long Distance Water Supply System focused on presence of disinfection by-products by using chlorine dioxide.

2. Brief characteristics of the Nová Bystrica-Čadca-Žilina Long Distance Water Supply System

The Nová Bystrica-Čadca-Žilina Long Distance Water Supply System supplies northern part of the Žilina Region with water from the Nová Bystrica water supply reservoir. The Nová Bystrica water treatment plant has been constructed in several stages. The first stage was put into the pilot operation in 1983 and to permanent operation in 1987. The designed capacity of water treatment plant is 200 $l.s^{-1}$ at the first stage, with designed capacity enhancement to 700 $l.s^{-1}$ at the second stage and 1060 $l.s^{-1}$ at the third stage of construction. Current operation capacity is 248 $l.s^{-1}$. Treatment technology has been designed as a two-stage technology with technological levels: dosage of chlorinated ferric sulfate, rapid mixing, slow mixing - mechanical and hydraulic, sedimentation, calcium hydrate dosage, rapid filtration and disinfection. Since problems with preparation of primary coagulant have occurred early during the pilot operation, provider has used alternative dosage of aluminium sulfate. Initial water disinfection was designed with using chlorine–ammonization methods. However chlorine dioxide has been used for disinfection since 1997.

The Long Distance water supply system supplies with water 31 municipalities and cities of the Žilina Region with the total number of 164 000 inhabitants. The Long Distance water supply system is made of welded steel pipes with bituminous lining. The stretch Nová Bystrica-Krásno u/Kysuca-Žilina (49km) is made of pipes with diameter of 800 mm and the stretch Krásno upon Kysuca-Čadca (7 km) is made of pipes with diameter of 600 mm. The total length of Long Distance water supply is 607,4 km including 53 water reservoirs and 30 pumping stations. Original disinfection by chlorine-ammonization was sufficient for bacteriological and biological safety of drinking water distributed by the Long Distance water supply, but it gave rise to quality deterioration considering nitrite parameter at the end of Long Distance water supply at Žilina. In addition to this problem another one related to iron presence has occurred in connection with reduced water abstraction from the network in the early 90's. After introducing the chlorine dioxide disinfection at the Nová Bystrica WTP such problems with nitrites have been eliminated completely and iron experienced a rapid decrease of its concentrations.

3. Monitoring of water disinfection by-products in the Nová Bystrica-Čadca-Žilina Long Distance Water Supply System

The monitoring of the Nová Bystrica-Čadca-Žilina Long Distance Water Supply System was carried out in the following periods: 09.2000, 11.2000, 02.2001, 05.2001, 08.2001 and 11.2001. During these samplings the following water disinfection by-products have been determined:

- trihalogenmethanes: chloroform ($CHCl_3$); bromdichloromethan ($CHBrCl_2$); dibromochloromethan ($CHBr_2Cl$); bromoform ($CHBr_3$)
- dichlorophenols: 2,3-DCP; 2,4-DCP; 2,5-DCP; 2,6-DCP
- trichlorophenols: 2,4,5-TCP; 2,4,6-TCP
- perchlorophenol: PCP
- benzene and its derivates: benzene, chlorobenzene, ethylbenzene, 1,2-DCB; 1,3-DCB; 1,4- DCB; toluene, xylenes
- another substances: 2,4-D acid (2,4-dichlorophenoxyacetic acid); 1,1,2-trichloroethen (TCE); 1,2-dichloroethan; tetrachloromethan (CCl_4); 1,1,2,2-tetrachloroethen (PCE)
- humic substances

At the same time (except for samplings carried out in 09.2000 and 11.2000) the aliphatic carboxyl acids have been also determined (butanoic, pentanoic, hexanoic, heptanoic, octanoic, nonanoic, decanoic, undecanoic, dodecanoic, tetradecanoic, hexadecanoic a octadecanoic acid) was monitored. Chlorine dioxide was determined in regular monthly intervals during the period from 07.2001 to 12.2001 and chlorites in regular monthly intervals during the period from 06.2001 to 12.2001.

4. Concentrations of chlorine dioxide in the Nová Bystrica-Čadca-Žilina Long Distance Water Supply System

Chlorine dioxide concentrations at the beginning of Long Distance water supply system were ranged from 0,20 to 0,50 mg/l. The concentrations at the Krásno upon Kysuca water reservoir were determined in range from 0,02 up to 0,25 mg/l. The higher concentrations were observed within its higher entering concentrations. The most frequent concentration was on the level of 0,10 mg/l. The highest concentration at the Čadca water reservoir was determined on the value of 0,15 mg/l., but the concentrations were mostly ranged from 0,02 to 0,05 mg/l. The concentrations lower than 0,02 mg/l were observed at the Považský Chlmec water reservoir (only in one case the concentration reached value of 0,05 mg/l). Based on this course of concentrations it shows that such disinfection used in the Long Distance water supply system results in rather significant reduction of chlorine dioxide concentrations together with formation of chlorites already at 19 km long stretch from the Nová Bystrica WTP to the Krásno u/ Kysuca water reservoir. Following the above facts it can be stated that the stretches from Nová Bystrica to Krásno u/ Kysuca and Krásno u/ Kysuca to Považský Chlmec were disinfected mostly by chlorites.

5. Results from monitoring of disinfection by-products and other organic substances

During the 09.2000 sampling only chlorobenzene, ethylbenzene, 1,1,2,2–tetrachloroethen, xylenes and humic substances were determined over a detection limit. Chlorobenzene concentrations were slightly increased (Nová Bystrica WTP 1,5 μg/l, Považský Chlmec water reservoir 6,3 μg/l, while the limit-MH for drinking water is 10 μg/l) and 1,1,2,2 – tetrachloroethen was detected only in concentration levels of approximately 0,2 mg/l (limit-NMH for drinking water is 10 μg/l). Xylenes, similarly to chlorobenzene, experienced upward trend and their maximum concentration did not exceed 4% of the limit (Nová Bystrica WTP 1,4 μg/l, Považský Chlmec water reservoir 3,8 μg/l, while the limit-MH for drinking water is 100 μg/l). Humic substances in raw water reached the value of 2,0 mg/l, but water treatment reduced their concentration to 0,70 mg/l.

During the 11.2000 sampling only chloroform was determined from THM on the level of 3 μg/l (limit for THM is 40 μg/l) and 1,1,2,2 – tetrachloroethen was detected in the same concentrations as in previous sampling (0,2 μg/l). Humic substances in raw water reached the value of 3,4 mg/l, but at the outflow from water treatment plant their concentrations reached value of 0,80 mg/l.

During the 02.2001 sampling chlorobenzene was detected in concentrations of 3,8-6,6 μg/l with downward trend and its concentration in raw water reached 8,9 μg/l. At the Považský Chlmec water reservoir the 1,1,2,2 –tetrachloroethen concentration reached the value of 0,9 μg/l. The content of humic substances in raw water was 1,2 μg/l and in treated water their content decreased to 0,6 μg/l.

During the 05.2001 sampling only xylenes were detected in concentrations up to 2,1μg/l and humic substances with value of 1,0 mg/l. During the 08.2001 sampling only humic substances were determined (raw water 0,9 mg/l, treated water 0,3 mg/l).

During the 11.2001 sampling only chloroform from THM substances was determined in concentrations ranged from 1,5 to 2,7 μg/l, benzene to 0,5 μg/l, toluene in raw water 1,3 μg/l (

limit-MH for drinking water is 50 μg/l), xylene less than 1,5 μg/l and humic substances (raw water only 0,7 mg/l).

During the samplings the aliphatic carboxyl acids were also determined because of their possible presence in the distribution system for distribution of water disinfected by chlorine dioxide (targeted analysis). 02.2001 sampling has proved that all aliphatic carboxyl acids were below the detection limit. During the 05.2001 only hexadecanoic (less than 0,40 μg/l) and octadecanoic (less 30 μg/l) acids were observed in higher concentrations and the rest was determined in lower concentrations. The numbers from 05.2001 sampling are the same as for 08.2001 sampling. The only difference is that heaxadecanoic acid concentrations were lower than 50 μg/l and for octadecanoic acid less than 0,40 μg/l. The highest concentrations during the 11.2001 sampling were determined for octadecanoic acid (0,30 μg/l).

Table 1 Results from monitoring of water disinfection by-products and other organic substances in the Nová Bystrica-Čadca-Žilina Long Distance Water Supply System

Water disinfection by-product	Maximum concentration	Limit Decree no.151/2004	Percentage limit
Chlorobenzene	6,6 μg/l	10 μg/l (MH)	66 %
1,1,2,2-tetrachloroethen	1,0 μg/l	10 μg/l (NMH)	10 %
Xylenes	1,4 μg/l	100 μg/l (MH)	1,4 %
Benzene	$<$ 0,5 μg/l	1,0 μg/l (MHRR)	$<$ 50 %
Toluene	1,3 μg/l	50 μg/l (MH)	2,5 %
Chloroform	3,0 μg/l	40 μg/l (MH)	8 %
Humic substances	Raw water: 1,2-3,4 mg/l Treated water: 0,6-1,0 mg/l	- -	- -
Aliphatic caroboxyl acids:			
Hexadecanoic acid	$<$ 0,50 μg/l	-	-
Octadecanoic acid	$<$ 0,40 μg/l	-	-
Other carboxyl acids	$<$ 0,10 μg/l	-	-

The table 1 shows results from monitoring of disinfection by-products and other organic substances at the Nová Bystrica-Čadca-Žilina Long Distance Water Supply System. For disinfection by-products the maximum determined concentration is presented that relates to their limit under the Decree no. 151/2004 Coll. and it is expressed in % from that value.

In proportional expression the maximum concentrations of disinfection by-products were ranged from 1,4 to 66,0 % with the lowest content of xylenes and the highest concentrations of chlorobenzene. Other substances contained: 2,5% of toluene, 8,0% of chloform, 10% of 1,1,2,2 –tetrachloroethen and about 50 % of benzene.

6. Chlorites concentrations in the Nová Bystrica-Čadca-Žilina Long Distance Water Supply System

Samples taken from the Nová Bystrica WTP outflow demonstrated rather quick decomposition of chlorine dioxide already after its dosage into the treated water, when concentrations of chlorine dioxide and chlorites reached almost the same values. At the chlorine dioxide concentrations lower than 0,20 mg/l at the outflow from the Nová Bystrica WTP its concentrations ranged from 0,02 to 0,0,5 mg/l already at Krásno u/ Kysuca. For chlorine dioxide concentration of 0,18 mg/l at the outflow from the Nová Bystrica WTP the concentration of chlorites was simultaneously determined at the value of 0,21 mg/l, but already at the Krásno u/ Kysuca water reservoir the chlorine dioxide concentration decreased to 0,02 mg/l, while chlorite concentration increased to 0,35 mg/l. In the Čadca water reservoir the concentration of chlorites decreased to 0,31 mg/l and in Považský Chlmec to the value of 0,27 mg/l. As to course of concentrations it is evident that water disinfection at the stretches Krásno u/ Kysuca water reservoir – Čadca water reservoir and Krásno u/ Kysuca water reservoir – Považský Chlmec water reservoir has been assured mostly by using chlorites.

In a case of higher chlorine dioxide dosage its residual concentrations were also higher at the Krásno u/ Kysuca water reservoir and chlorite concentration maximum values moved in the GWSS towards Žilina and Čadca. To reduce concentration of chlorites below 0,20 mg/l in the entire GWSS it is necessary to use chlorine dioxide dosage in concentrations of 0,25 – 0,27 mg/l at most with respect to own consumption of chlorine dioxide as well as own process of chlorites decomposition. The figures 1 and 2 show courses of chlorine dioxide and chlorite concentrations in GWSS during 02.2002 sampling.

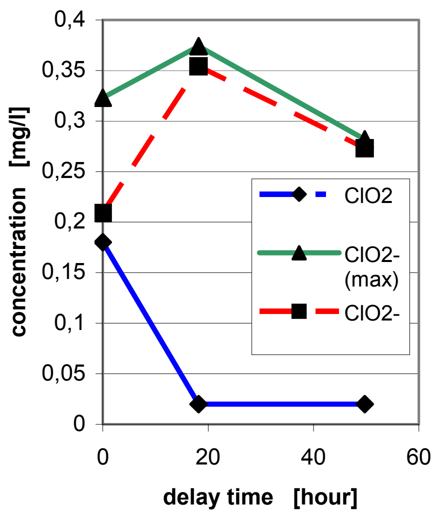


Fig.1 Course of chlorine dioxide and chlorite concentrations in the N.Bystrica-Krásno-P. Chlmec GWSS

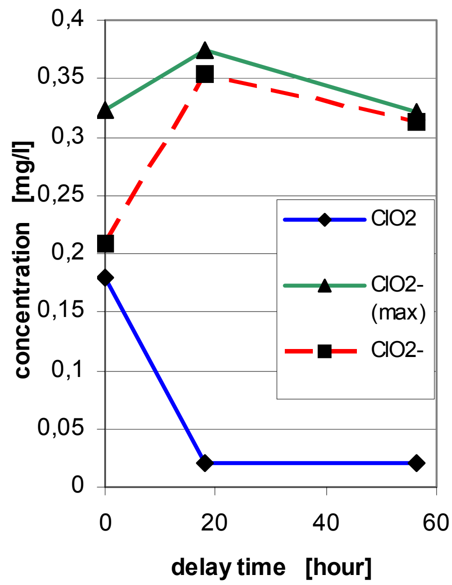
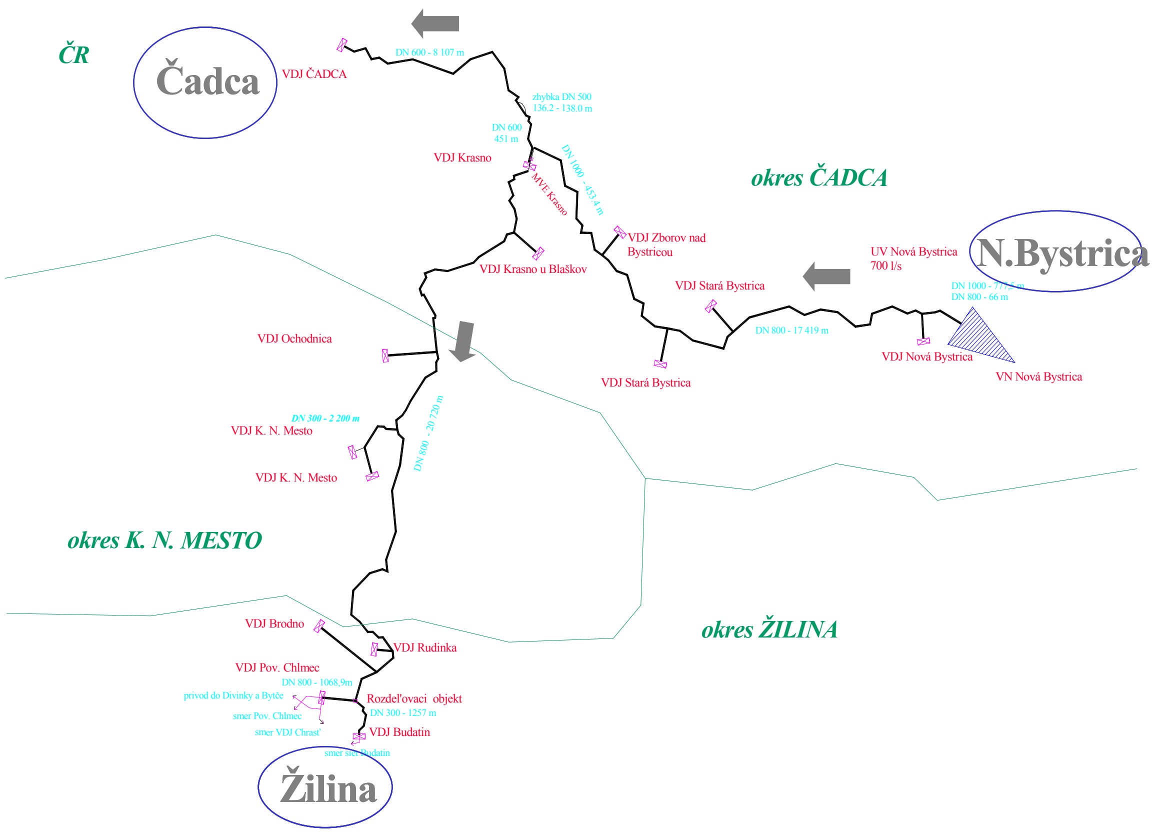


Fig.2 Course of chlorine dioxide and chlorite concentrations in the N. Bystrica-Krásno-Čadca GWSS

The figures show that more rapid chlorite decomposition occurred at the stretch of Krásno u/ Kysuca water reservoir – Považsky Chlmec water reservoir, when their concentrations have decreased by 0,08 mg/l for about 30 hours. At the stretch Krásno u/ Kysuca water reservoir – Čadca water reservoir the concentration of chlorites has decreased only by 0,04 mg/l for about 38 hours. Index ClO_2^- (max) represents maximum available chlorite concentrations in taken water samples without elimination of the effect of chlorine dioxide and its decomposition on increase of chlorite concentrations.

7. Conclusion

Within the monitoring of disinfection by-products at the Nová Bystrica-Čadca-Žilina Long Distance Water Supply System the following substances have been determined: trihalogenmethanes ($CHCl_3$; $CHBrCl_2$; $CHBr_2Cl$; $CHBr_3$), dichlorophenols (2,3-DCP; 2,4-DCP; 2,5-DCP; 2,6-DCP), trichlorophenols (2,4,5-TCP; 2,4,6-TCP), perchlorophenol PCP, benzene and its derivates (benzene, chlorobenzene, ethylbenzene, 1,2-DCB; 1,3-DCB; 1,4-DCB; toluene, xylenes) a other organic substances (2,4-D acid, e.i. 2,4-dichlorophenoxyacetic acid; 1,1,2-trichloroethen; 1,2-dichloroethan; CCl_4; 1,1,2,2-tetrachloroethen), humic substances, alipahtic carboxyl acids (butanoic, pentanoic, hexanoic, heptanoic, octanoic, nonanoic, decanoic, undecanoic, dodecanoic, tetradecanoic, hexadecanoic and octadecanoic acids) and chlorites.



The results show that from the above substances only the following were determined over the detection limit: chlorobenzene, 1,1,2,2-tetrachloroethen, xylenes, toluene, chloroform, humic substances, chlorites, hexadecanoic and octadecanoic acids. The highest concentration was determined for chlorobenzene (6,6 µg/l that represents 66 % of limit under the Decree no. 151/2004 Coll.), chloroform (1,4 µg/l, 1,4 %), toluene (1,3 µg/l, 2,5 %) and 1,1,2,2-tetrachloroethen (1,0 µg/l, 10%). Aliphatic carboxyl acids reached the concentrations lower than 0,50 µg/l. The maximum concentrations of chlorites (NMH 0,20 mg/l) ranged from 0,30 to 0,35 mg/l were detected at sites where the chloride dioxide concentration is very low. Referring to own consumption of chlorine dioxide and course of its decomposition reactions in GSSW it was recommended for providers to use chlorine dioxide dosages in concentrations of 0,25 –0,27 mg/l at most and simultaneously to monitor microbiological and biological water quality, especially at stretches, where distributed water is disinfected only by chlorites. Based on evaluation of obtained results further steps will be proposed for water quality assurance in this GWSS.

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Water Treatment Technology Optimization on aggressive Properties in long-distance Distribution System

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Abstract: The Stakčín water-supply reservoir serves as a water resource for long-distance distribution system with available water capacity of 45 mil.m^3. Surface water from water-supply reservoir is transported by gravitational force to the Stakčín WTP through the steel pipes of DN 1000 mm with the total length of 4471 m. The water is treated by clarification on gallery clarifiers and filtration through the sand rapid filters using 40 % ferric sulfate aqueous solution as a coagulant. Depending on treated water quality the dosage of ferric sulfate was ranged from 4 to 10 mg/l of calcium hydrate in the period from 10.1996 to 10.1997. Water disinfection has been carried out with regard to actual delay of water in pipeline by chlorine-amonisation with dosage of gaseous chlorine with the maximum of 0,4g and 0,2 g of ammonium sulfate. At the site of Ťahanovce housing development, the maximum iron concentrations at particular sampling sites reached the value range from 0,4 to 1,1 mg/l with the mean concentrations ranged from 0,20 to 0,31 mg/l.
Keywords: Optimalization, quality of water, water treatment, transportation

1. Introduction

The Stakčín water-supply reservoir serves as a water resource for long-distance distribution system with available water capacity of 45 mil.m^3. Surface water from water-supply reservoir is transported by gravitational force to the Stakčín WTP through the steel pipes of DN 1000 mm with the total length of 4471 m. The water is treated by clarification on gallery clarifiers and filtration through the sand rapid filters using 40 % ferric sulfate aqueous solution as a coagulant. Depending on treated water quality the dosage of ferric sulfate was ranged from 4 to 10 mg/l of calcium hydrate in the period from 10.1996 to 10.1997. Water disinfection has been carried out with regard to actual delay of water in pipeline by chlorine-amonisation with dosage of gaseous chlorine with the maximum of 0,4g and 0,2 g of ammonium sulfate.

The total length of the Starin-Košice long distance distribution system is 134,5 km and it is made of steel pipes with diameter of 1000 mm. The length of distribution system from the Stakčín WTP up to the water divider is 130,03 km. The water reservoirs along the entire distribution system are built at Snina, Humenné, Vranov nad Topľou, Medzianky, Prešov and Košice with the total capacity of 93 thousands m^3. The pipeline at the stretch from Stakčín WTP to Prešov water reservoir has been lined by bituminous coating (length of 98,4 km), the section between the Prešov water reservoir and Košice water divider is without inner coating (length of 31,6 km).

The actual delay of water in long-distance distribution system from the Stakčín WTP to Košice water divider is 133 hours with the WTP capacity of 313 l/s at a site below the Košice T2 water reservoir, with capacity of 570/l it is 100 hours and with 990 l/s it is 75 hours.

During the operation of long-distance distribution system, there has been observed water quality deterioration by effect of corrosion as a consequence of decreased water consumption and subsequent increase of iron concentrations in a pipeline. Considerably higher concentrations of iron in drinking water have been observed mostly at the end of pipeline in the stretch of the Košice- Ťahanovce water divider.

At the site of Ťahanovce housing development, the maximum iron concentrations at particular sampling sites reached the value range from 0,4 to 1,1 mg/l with the mean concentrations ranged from 0,20 to 0,31 mg/l.

Since the water quality has not reached a desirable level, the Water Research Institute has carried out operational evaluation of aggressive properties of water (corrosion tests) during a period 10.1996-10.1997. The work has been performed within the solution of stage task – "Stability of drinking water during distribution" included in VTP 514-78 "Research of drinking water treatability and environmental aspects of watercourses". Regarding the existing water quality status of the Starina-Košice long distance distribution system as well as steel pipe lining by inner bituminous coating, the equipment for monitoring of aggressive properties of water has been installed in the following sampling sites:

- Stakčín Water Treatment Plant (WTP)
- Humenné Water Reservoir (WR) (34,8 km)
- Prešov Water Reservoir (WR) (98,4 km)
- Košice Water Divider (WD) (130,0 km)

2. Evaluation of the corrosion tests during the period 10.1996-10.1997

During the above period, six corrosion tests with 30 and 60-day exposition were valuated at each sampling site. Considering the duration of corrosion test, they have included all changes in quality of treated water entering the distribution system in connection with changes in quality of raw water flowing into the Stakčín WTP from water-supply reservoir as well as own operation of water treatment technology (dosage of ferric sulfate and calcium hydrate). During the corrosion tests, the long-distance distribution system transported water in volume of about 13 555 000 m^3. During the continuous operation it would represent the average water supply of 430 l/s.

The value range of selected treated water quality parameters during the corrosion tests was as follows: COD_{Mn} 0,9-1,4 mg/l, pH 7,5-8,1, ACIDITY 1,70-1,95 mmol/l, calcium 33-40 mg/l, magnesium 4,3-6,1 mg/l, iron lower than 0,09 mg/l, manganese lower than 0,04 mg/l, ammonium ions lower than 0,05 mg/l, nitrites lower than 0,01 mg/l, nitrates 2,9-3,8 mg/l, chlorides 3,2-4,3 mg/l, sulfates 19,2-39,4 mg/l, turbidity lower than 0,5 NTU and water temperature of 3,5-9,3°C.

During the first tree corrosion tests (10.1996–04.1997) the positive effect of sufficient $CaCO_3$ saturation of water on corrosion rates was observed in the entire length of pipeline. During the $CaCO_3$ oversaturation from 0,05 to 0,15 mmol/kg, the saturation indexes have reached values of 0,0–0,25. Corrosion rates for such stabilized water are shown in table 1.

Following the above results, water distributed by long-distance distribution system might be evaluated as slightly aggressive (aggressiveness category I) without need to propose anti-corrosion measures.

Table 1 Range of corrosion rates in the distribution system during the period 10.1996-04.1997

Period of corrosion test	Range of corrosion rates (μm/year)
10.-12.1996	21-66
12.1996-02.1997	26-64
02.-04.1997	43-61

The water temperature during that period was ranged from 3,5 to 10.1°C with observed increase in the long-distribution system of 0,6–1,3 °C. The iron concentrations reached maximum values of 0,15–0,23 mg/l at Pumping Station (PS) Hanušovce (77,6 km) and Section Valve (SV) no. 18 (85,9 km) respectivelly. At the Košice water divider they reached values of 0,15–0,19 mg/l. The dosages of ferric sulfate ranged from 7 to 10 mg/l and calcium hydrate from 3 to 8 mg/l.

During the next tree corrosion tests (04.-10.1997), the corrosion rates were ranged as shown in table 2.

Table 2 Range of corrosion rates in long-distance distribution system during 04.-10.1997

Period of corrosion test	Range of corrosion rates (μm/year)
04.-06.1997	46-105
06.-08.1997	58-98
08.-10.1997	37-78

The decrease in dosage of calcium hydrate to 2-3 mg/l with dosage of ferric sulfate of 6mg/l during the corrosion test 04.-06. 1997 resulted in insufficient $CaCO_3$ saturation of water (-0,03 mmol/l) in entire length of the long-distance distribution system. During the corrosion test 06.-08.1997 the dosage of calcium hydrate was increased to 4-5mg/l with unchanged dosage of ferric sulfate, e.i. 6 mg/l and it resulted in oversaturation of water at the Stakčín WTP by 0,05mmol/l, while in following parts of distribution system it has continually decreased up to 0,0 mmol/l at the Košice water divider. During the last corrosion test 08.-10.1997 the dosage of calcium hydrate was increased to 5-6 mg/l and ferric sulfate decreased to 4 mg/l that resulted in oversaturation of water to 0,05mmol/l at the Stakčín WTP and it reached balanced values on the level of 0,025 mmol/l in the whole distribution system.

The temperature was ranged from 6,2 to 11,2 °C and there was observed its increase from 2,2 to 4,5 °C. The maximum iron concentrations were observed mostly during the state of insufficient saturation and insufficient $CaCO_3$ oversaturation, respectively, at PS Hanušovce, SV no. 18 and Košice WD (0,23–0,30 mg/l).

As far as treated water stabilization is considered, it is necessary to point out the relations between calcium hydrate and $CaCO_3$ overasaturation as well as corrosion rates. The results of corrosion tests show that decreas of calcium hydrate towards ferric sulfate dosage results in lower saturation of water and there were observed also higher values of corrosion rates. The dosage of ferric sulfate depends on raw water quality and subsequently calcium hydrate dosage depends on ferric sulfate concentrations with regard to formation of optimum conditions for water treatment and assurance of their quality according to requirements on drinking water quality. For water quality treated at the Stakčín WTP it was necessary to take into account a possibility of water stabilization during the calcium hydrate dosage and thus for reduction of aggressive properties of water, especially during the period 04.–010.1997, when corrosion rates at the end of system exceeded the limit of 100 μm/year, there was proposed water treatment technology for achievement of calcium-carbonate balance and state of required $CaCO_3$ oversaturation of water for the purpose to form a protection layer with high affinity to pipe walls.

3. Water treatment technology optimization for assurance of water stabilization

The effect of water treatment technology optimization related to water stabilization at the Stakčín WTP was monitored during the period 02.–10.1999 and it was compared with the same period before technology optimization. (02.–10.1997). During that period four corrosion tests have been evaluated with 30 and 60-day exposition and corrosion devices were placed at the same sites of the long-distance distribution system as during the first corrosion tests (10.1996-10.1997).

Table 3 The mean dosages of lime and ferric sulfate during particular comparative corrosion tests

Period of corrosion tests: 02.-10.1997			Period of corrosion tests: 02.-10.1999		
Corrosion test	Mean dosage of lime [mg/l]	Mean dosage of ferric sulfate [mg/l]	Corrosion test	Mean dosage of lime [mg/l]	Mean dosage of ferric sulfate [mg/l]
02.-04.1997	2,6	7,8	02.-04.1999	4,4	7,3
04.-06.1997	2,7	6,0	04.-06.1999	6,2	7,6
06.-08.1997	4,2	5,8	06.-08.1999	5,5	5,2
08.-10.1997	4,6	5,0	08.-10.1999	6,2	4,3

In compared period 02.–10.1997 of the first corrosion tests, lime dosages were raged from 2 to 6 mg/l and ferric sulfate from 5 to 10 mg/l. However, such dosages were insufficient regarding achievement of needed $CaCO_3$ oversaturation of water, what resulted in higher values of corrosion rates. If the $CaCO_3$ oversaturation of water with a mean dosage of lime and ferric sulfate was ranged between –0,125 and 0,05 mmol/l, it means that water in long-distance distribution system was saturated insufficiently or insufficiently oversaturated by $CaCo_3$, respectivelly, and optimally it reached only low limit of range values included in STS (Slovak Technical Standard) 75 7151 (0,5–0,10 mmol/l). After water treatment technology optimization and also in view of water stabilization the values of water oversaturation reached the range between 0,025 mmol/l and 0,10 mmol/l, while the values of 0,025 mmol/l were determined only in particular cases.

While the mean dosage of lime during the corrosion tests 02.-10. 1997 with the mean dosage of ferric sulfate of 5,0 - 7,8 mg/l were ranged from 2,6 to 4,6 mg/l, the mean dosage of lime during the further corrosion tests 02.-10.1999 with the mean ferric sulfate dosage of 4,3 –7,6 mg/l were ranged from 4,4 to 6,2 mg/l. A comparison of lime dosage for corresponding periods of corrosion tests shows that lime dosages were increased 1,3–2,3 times after the optimization. The increase of lime dosage calculated to ferric sulfate dosage represented 0,27-0,55 mg of Ca(OH)/mg $Fe_2(SO_4)_3$, and higher values were observed especially during the summer period. The increase of lime dosage resulted in decrease of corrosion rates as well as decrease of iron concentration in particular sampling sites along the distribution system.

Figures 1–4 show effects of water treatment technology optimization for selected periods of compared corrosion tests at the Stakčín WTP considering water stabilization on the courses of water oversaturation by $CaCO_3$, saturation indexes and iron concentrations in the long-distance distribution system.

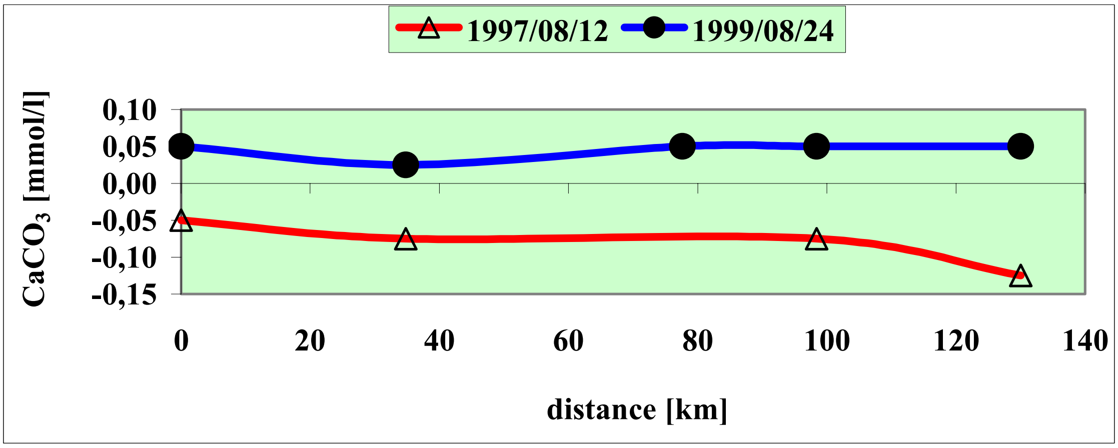

Figure 1 Process of water oversaturation by $CaCO_3$ in the long-distance distribution system for period of compared corrosion tests.

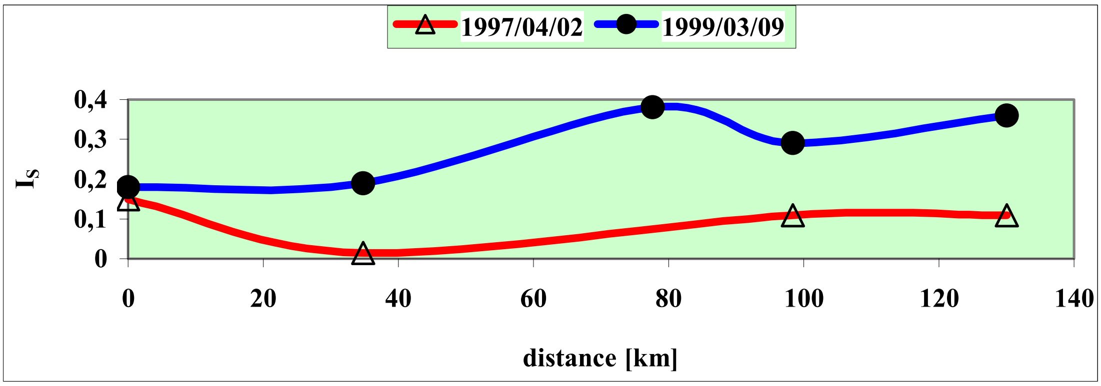

Figure 2 Course of saturation indexes in the long-distance distribution system for selected period of compared corrosion tests

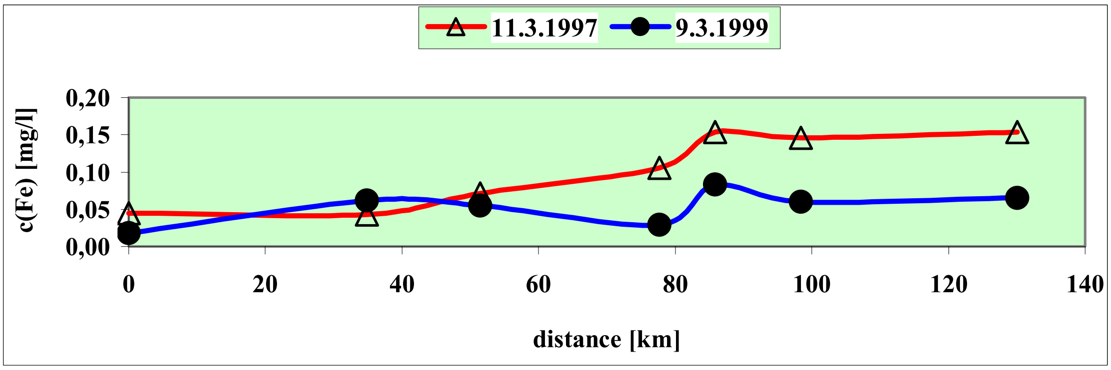

Figure 3 Iron concentration in the long-distance distribution system during the selected period of compared corrosion tests

During the corrosion tests after treatment technology optimization (02.–10.1999) the iron concentration reached maximum value of 0,17 mg/l (Košice water divider –04.1999). During all other samplings the iron concentrations were ranged to 0,10 mg/l at the end of long-distance distribution system.

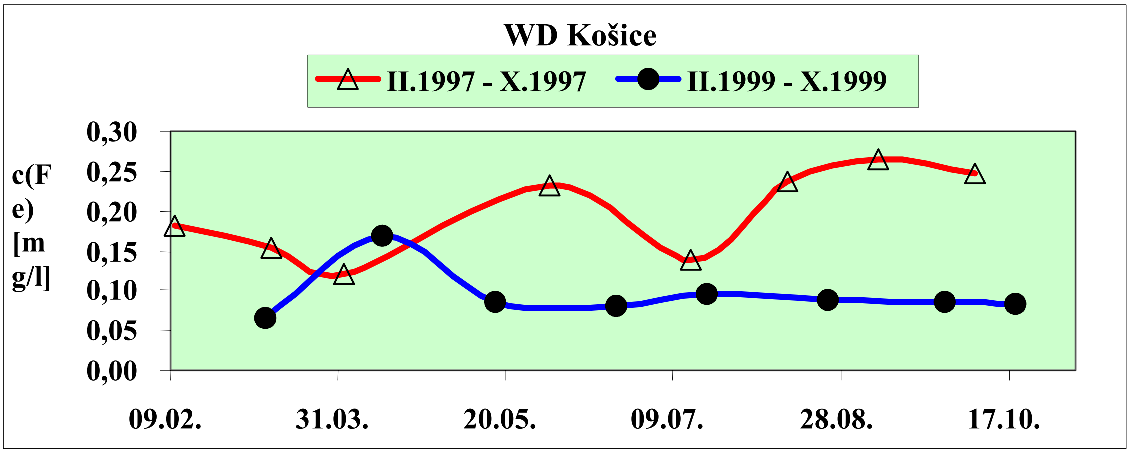

Figure 4 Iron concentration at the end of distribution system at the Košice Water Divider for selected period of compared corrosion tests

Corrosion rates after the treatment technology optimization have decreased during compared periods by 10 -75 % (02.–04.1997), 15-60 % (04.–06.1997), 5-45 % (06.–08.1997) and for the last corrosion test by 15-35 % (08.–10.1997). The range of corrosion rates for particular sampling sites during the corrosion tests 02.10 1997 and 0.2–10.1999 are shown in the table 4.

Table 4 Range of corrosion rates before (02.-10. 1997) and after (02.-10.1999) water treatment technology optimization at the Stakčín WTP

Sampling site	Corrosion rates [μm/year] 02.-10.1997	Corrosion rates [μm/year] 02.-10.1999
Stakčín WTP	45 - 78	31 - 55
Humenné WR	53 - 90	44 - 60
Prešov WR	61 - 83	70 - 80
Košice WD	42 - 105	10 - 54

The most significant decrease of corrosion rates have been observed at the end of long-distance distribution system, where decrease represents 35–75 % compared to values before water treatment technology optimization and the corrosion rates were ranged from 10 to 54 μm a year.

4. Conclusion

Based on evaluation of water treatment technology optimization at the Stakčín WTP and considering water stabilization it can be stated that these measures had considerable effect on decrease of aggressive properties of distributed water, what resulted in significant decrease of the iron concentrations as well as corrosion rates in the long-distance distribution system. Following the categories of aggressiveness, the water distributed by the Starina-Košice system was classified into the 1st category of aggressiveness (moderate aggressiveness of water).

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Chlorine Dioxide Disinfection by Products in the Nová Bystrica-Čadca-Žilina Long Distance Water Supply System

**Jozef Kriš, Karol Munka, Elena Büchlerová,
Monika Karácsonyová, Ľuboslav Gajdoš**

Abstract: In a process of water disinfection it is necessary to distinguish between primary disinfection focused on removal or inactivation of microbiological contaminants from raw water and secondary disinfection focused on maintenance of residual concentration of disinfector in distribution system. Current practice related to disinfection follows two approaches. The paper presents results from stage task solution "Research of physical-chemical changes in water quality during its distribution" at the Nová Bystrica-Čadca-Žilina Long Distance Water Supply System (LDWSS) focused on presence of disinfection by-products by using chlorine dioxide.

Keywords: Disinfection, chlordioxide, by product, water treatment

1. Introduction

In a process of water disinfection it is necessary to distinguish between primary disinfection focused on removal or inactivation of microbiological contaminants from raw water and secondary disinfection focused on maintenance of residual concentration of disinfector in distribution system. Current practice related to disinfection follows two approaches:

- Maintenance of residual concentration of disinfector and effort to find methods for achievement of acceptable concentrations of disinfection by-products.
- Search for new methods of sound management of distribution system without maintaining the residual concentration of disinfector.

In some countries it is required or recommended to maintain residual concentration of disinfector in order to meet limits of microbiological parameters, minimize bio-film formation, prevent from secondary contamination in distribution system and indication, whether contamination occurs as results of disinfector concentration reduction. Other countries follow the opinion that residual concentration of disinfector is not required for good water quality, microbiologically safe groundwater or for some surface water resources treated by multilevel technologies. Another reason is a minimization of disinfection by-product formation and reduction of risk connected with their formation, respectivelly. The residual concentration is not required, because water is treated in order to minimize amount of organic substances entering distribution system. There is an opinion followed that maintenance of residual concentrations of disinfectors in distribution system does not prevent from any significant water contamination and significant decrease or elimination of residual concentration of disinfector is not considered as reliable indication of contamination presence. Changes in pH value and water conductivity might be such indicators. Other authors give reason for that there is no reliable indicator of contamination and the most reliable measure preventing contamination is thorough proposal, operation, maintenance and control of distribution system.

The paper presents results from stage task solution "Research of physical-chemical changes in water quality during its distribution" at the Nová Bystrica-Čadca-Žilina Long Distance Water Supply System focused on presence of disinfection by-products by using chlorine dioxide.

2. Brief characteristics of the Nová Bystrica-Čadca-Žilina Long Distance Water Supply System

The Nová Bystrica-Čadca-Žilina Long Distance Water Supply System supplies northern part of the Žilina Region with water from the Nová Bystrica water supply reservoir. The Nová Bystrica water treatment plant has been constructed in several stages. The first stage was put into the pilot operation in 1983 and to permanent operation in 1987. The designed capacity of water treatment plant is 200 $l.s^{-1}$ at the first stage, with designed capacity enhancement to 700 $l.s^{-1}$ at the second stage and 1060 $l.s^{-1}$ at the third stage of construction. Current operation capacity is 248 $l.s^{-1}$. Treatment technology has been designed as a two-stage technology with technological levels: dosage of chlorinated ferric sulfate, rapid mixing, slow mixing - mechanical and hydraulic, sedimentation, calcium hydrate dosage, rapid filtration and disinfection. Since problems with preparation of primary coagulant have occurred early during the pilot operation, provider has used alternative dosage of aluminium sulfate. Initial water disinfection was designed with using chlorine–ammonization methods. However chlorine dioxide has been used for disinfection since 1997.

The Long Distance water supply system supplies with water 31 municipalities and cities of the Žilina Region with the total number of 164 000 inhabitants. The Long Distance water supply system is made of welded steel pipes with bituminous lining. The stretch Nová Bystrica-Krásno u/Kysuca-Žilina (49km) is made of pipes with diameter of 800 mm and the stretch Krásno upon Kysuca-Čadca (7 km) is made of pipes with diameter of 600 mm. The total length of Long Distance water supply is 607,4 km including 53 water reservoirs and 30 pumping stations. Original disinfection by chlorine-ammonization was sufficient for bacteriological and biological safety of drinking water distributed by the Long Distance water supply, but it gave rise to quality deterioration considering nitrite parameter at the end of Long Distance water supply at Žilina. In addition to this problem another one related to iron presence has occurred in connection with reduced water abstraction from the network in the early 90's. After introducing the chlorine dioxide disinfection at the Nová Bystrica WTP such problems with nitrites have been eliminated completely and iron experienced a rapid decrease of its concentrations.

3. Monitoring of water disinfection by-products in the Nová Bystrica-Čadca-Žilina Long Distance Water Supply System

The monitoring of the Nová Bystrica-Čadca-Žilina Long Distance Water Supply System was carried out in the following periods: 09.2000, 11.2000, 02.2001, 05.2001, 08.2001 and 11.2001. During these samplings the following water disinfection by-products have been determined:

- trihalogenmethanes: chloroform ($CHCl_3$); bromdichloromethan ($CHBrCl_2$); dibromochloromethan ($CHBr_2Cl$); bromoform ($CHBr_3$)
- dichlorophenols: 2,3-DCP; 2,4-DCP; 2,5-DCP; 2,6-DCP
- trichlorophenols: 2,4,5-TCP; 2,4,6-TCP
- perchlorophenol: PCP
- benzene and its derivates: benzene, chlorobenzene, ethylbenzene, 1,2-DCB; 1,3-DCB; 1,4- DCB; toluene, xylenes
- another substances: 2,4-D acid (2,4-dichlorophenoxyacetic acid); 1,1,2-trichloroethen (TCE); 1,2-dichloroethan; tetrachloromethan (CCl_4); 1,1,2,2-tetrachloroethen (PCE)
- humic substances

At the same time (except for samplings carried out in 09.2000 and 11.2000) the aliphatic carboxyl acids have been also determined (butanoic, pentanoic, hexanoic, heptanoic, octanoic, nonanoic, decanoic, undecanoic, dodecanoic, tetradecanoic, hexadecanoic a octadecanoic acid) was monitored. Chlorine dioxide was determined in regular monthly intervals during the period from 07.2001 to 12.2001 and chlorites in regular monthly intervals during the period from 06.2001 to 12.2001.

4. Concentrations of chlorine dioxide in the Nová Bystrica-Čadca-Žilina Long Distance Water Supply System

Chlorine dioxide concentrations at the beginning of Long Distance water supply system were ranged from 0,20 to 0,50 mg/l. The concentrations at the Krásno upon Kysuca water reservoir were determined in range from 0,02 up to 0,25 mg/l. The higher concentrations were observed within its higher entering concentrations. The most frequent concentration was on the level of 0,10 mg/l. The highest concentration at the Čadca water reservoir was determined on the value of 0,15 mg/l., but the concentrations were mostly ranged from 0,02 to 0,05 mg/l. The concentrations lower than 0,02 mg/l were observed at the Považský Chlmec water reservoir (only in one case the concentration reached value of 0,05 mg/l). Based on this course of concentrations it shows that such disinfection used in the Long Distance water supply system results in rather significant reduction of chlorine dioxide concentrations together with formation of chlorites already at 19 km long stretch from the Nová Bystrica WTP to the Krásno u/ Kysuca water reservoir. Following the above facts it can be stated that the stretches from Nová Bystrica to Krásno u/ Kysuca and Krásno u/ Kysuca to Považský Chlmec were disinfected mostly by chlorites.

5. Results from monitoring of disinfection by-products and other organic substances

During the 09.2000 sampling only chlorobenzene, ethylbenzene, 1,1,2,2–tetrachloroethen, xylenes and humic substances were determined over a detection limit. Chlorobenzene concentrations were slightly increased (Nová Bystrica WTP 1,5 μg/l, Považský Chlmec water reservoir 6,3 μg/l, while the limit-MH for drinking water is 10 μg/l) and 1,1,2,2 – tetrachloroethen was detected only in concentration levels of approximately 0,2 mg/l (limit-NMH for drinking water is 10 μg/l). Xylenes, similarly to chlorobenzene, experienced upward trend and their maximum concentration did not exceed 4% of the limit (Nová Bystrica WTP 1,4 μg/l, Považský Chlmec water reservoir 3,8 μg/l, while the limit-MH for drinking water is 100 μg/l). Humic substances in raw water reached the value of 2,0 mg/l, but water treatment reduced their concentration to 0,70 mg/l.

During the 11.2000 sampling only chloroform was determined from THM on the level of 3 μg/l (limit for THM is 40 μg/l) and 1,1,2,2 – tetrachloroethen was detected in the same concentrations as in previous sampling (0,2 μg/l). Humic substances in raw water reached the value of 3,4 mg/l, but at the outflow from water treatment plant their concentrations reached value of 0,80 mg/l.

During the 02.2001 sampling chlorobenzene was detected in concentrations of 3,8-6,6 μg/l with downward trend and its concentration in raw water reached 8,9 μg/l. At the Považský Chlmec water reservoir the 1,1,2,2 –tetrachloroethen concentration reached the value of 0,9 μg/l. The content of humic substances in raw water was 1,2 μg/l and in treated water their content decreased to 0,6 μg/l.

During the 05.2001 sampling only xylenes were detected in concentrations up to 2,1μg/l and humic substances with value of 1,0 mg/l. During the 08.2001 sampling only humic substances were determined (raw water 0,9 mg/l, treated water 0,3 mg/l).

During the 11.2001 sampling only chloroform from THM substances was determined in concentrations ranged from 1,5 to 2,7 μg/l, benzene to 0,5 μg/l, toluene in raw water 1,3 μg/l (

limit-MH for drinking water is 50 μg/l), xylene less than 1,5 μg/l and humic substances (raw water only 0,7 mg/l).

During the samplings the aliphatic carboxyl acids were also determined because of their possible presence in the distribution system for distribution of water disinfected by chlorine dioxide (targeted analysis). 02.2001 sampling has proved that all aliphatic carboxyl acids were below the detection limit. During the 05.2001 only hexadecanoic (less than 0,40 μg/l) and octadecanoic (less 30 μg/l) acids were observed in higher concentrations and the rest was determined in lower concentrations. The numbers from 05.2001 sampling are the same as for 08.2001 sampling. The only difference is that heaxadecanoic acid concentrations were lower than 50 μg/l and for octadecanoic acid less than 0,40 μg/l. The highest concentrations during the 11.2001 sampling were determined for octadecanoic acid (0,30 μg/l).

Table 1 Results from monitoring of water disinfection by-products and other organic substances in the Nová Bystrica-Čadca-Žilina Long Distance Water Supply System

Water disinfection by-product	Maximum concentration	Limit Decree no.151/2004	Percentage limit
Chlorobenzene	6,6 μg/l	10 μg/l (MH)	66 %
1,1,2,2-tetrachloroethen	1,0 μg/l	10 μg/l (NMH)	10 %
Xylenes	1,4 μg/l	100 μg/l (MH)	1,4 %
Benzene	< 0,5 μg/l	1,0 μg/l (MHRR)	< 50 %
Toluene	1,3 μg/l	50 μg/l (MH)	2,5 %
Chloroform	3,0 μg/l	40 μg/l (MH)	8 %
Humic substances	Raw water: 1,2-3,4 mg/l Treated water: 0,6-1,0 mg/l	- -	- -
Aliphatic caroboxyl acids:			
Hexadecanoic acid	< 0,50 μg/l	-	-
Octadecanoic acid	< 0,40 μg/l	-	-
Other carboxyl acids	< 0,10 μg/l	-	-

The table 1 shows results from monitoring of disinfection by-products and other organic substances at the Nová Bystrica-Čadca-Žilina Long Distance Water Supply System. For disinfection by-products the maximum determined concentration is presented that relates to their limit under the Decree no. 151/2004 Coll. and it is expressed in % from that value.

In proportional expression the maximum concentrations of disinfection by-products were ranged from 1,4 to 66,0 % with the lowest content of xylenes and the highest concentrations of chlorobenzene. Other substances contained: 2,5% of toluene, 8,0% of chloform, 10% of 1,1,2,2 –tetrachloroethen and about 50 % of benzene.

6. Chlorites concentrations in the Nová Bystrica-Čadca-Žilina Long Distance Water Supply System

Samples taken from the Nová Bystrica WTP outflow demonstrated rather quick decomposition of chlorine dioxide already after its dosage into the treated water, when concentrations of chlorine dioxide and chlorites reached almost the same values. At the chlorine dioxide concentrations lower than 0,20 mg/l at the outflow from the Nová Bystrica WTP its concentrations ranged from 0,02 to 0,0,5 mg/l already at Krásno u/ Kysuca. For chlorine dioxide concentration of 0,18 mg/l at the outflow from the Nová Bystrica WTP the concentration of chlorites was simultaneously determined at the value of 0,21 mg/l, but already at the Krásno u/ Kysuca water reservoir the chlorine dioxide concentration decreased to 0,02 mg/l, while chlorite concentration increased to 0,35 mg/l. In the Čadca water reservoir the concentration of chlorites decreased to 0,31 mg/l and in Považský Chlmec to the value of 0,27 mg/l. As to course of concentrations it is evident that water disinfection at the stretches Krásno u/ Kysuca water reservoir – Čadca water reservoir and Krásno u/ Kysuca water reservoir – Považský Chlmec water reservoir has been assured mostly by using chlorites.

In a case of higher chlorine dioxide dosage its residual concentrations were also higher at the Krásno u/ Kysuca water reservoir and chlorite concentration maximum values moved in the GWSS towards Žilina and Čadca. To reduce concentration of chlorites below 0,20 mg/l in the entire GWSS it is necessary to use chlorine dioxide dosage in concentrations of 0,25 – 0,27 mg/l at most with respect to own consumption of chlorine dioxide as well as own process of chlorites decomposition. The figures 1 and 2 show courses of chlorine dioxide and chlorite concentrations in GWSS during 02.2002 sampling.

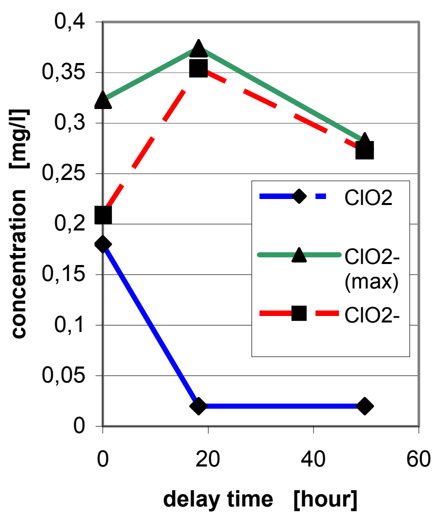


Fig.1 Course of chlorine dioxide and chlorite concentrations in the N.Bystrica-Krásno-P. Chlmec GWSS

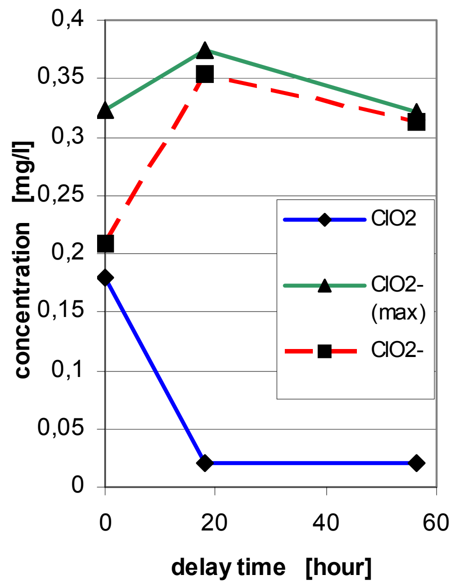
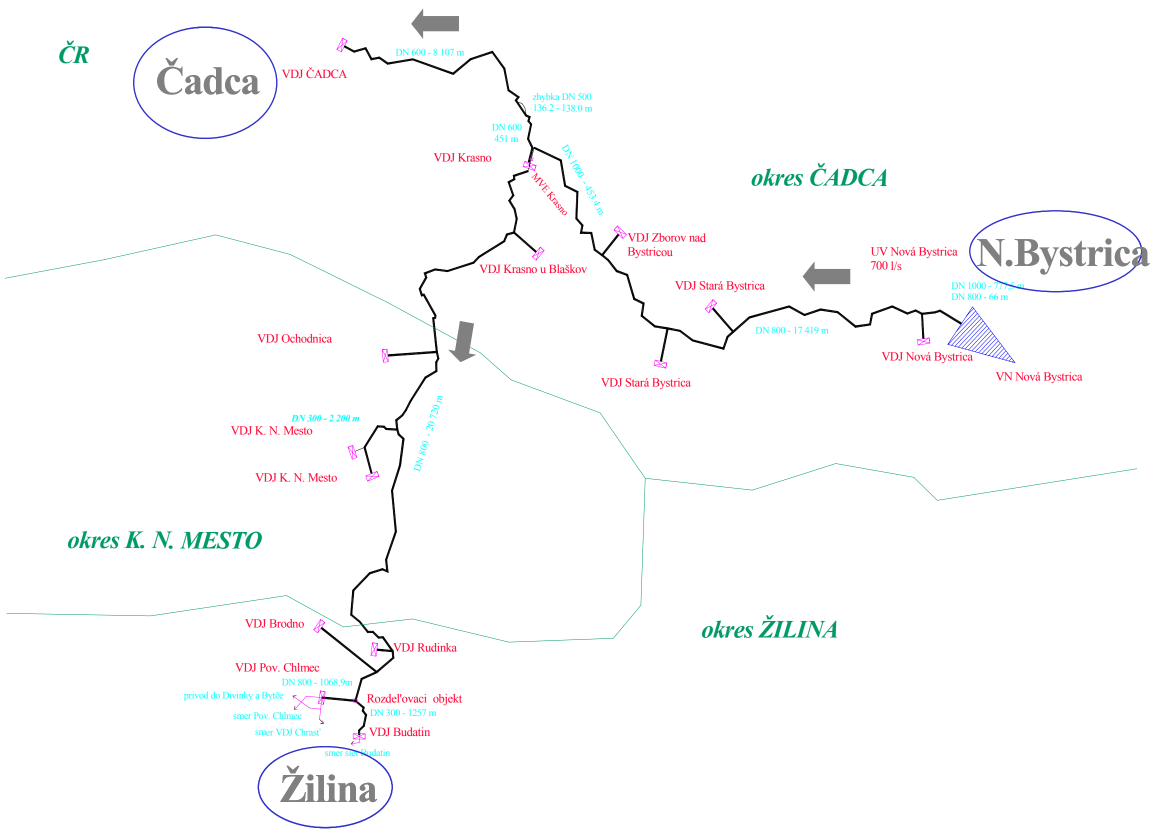


Fig.2 Course of chlorine dioxide and chlorite concentrations in the N. Bystrica-Krásno-Čadca GWSS

The figures show that more rapid chlorite decomposition occurred at the stretch of Krásno u/ Kysuca water reservoir – Považsky Chlmec water reservoir, when their concentrations have decreased by 0,08 mg/l for about 30 hours. At the stretch Krásno u/ Kysuca water reservoir – Čadca water reservoir the concentration of chlorites has decreased only by 0,04 mg/l for about 38 hours. Index ClO_2^- (max) represents maximum available chlorite concentrations in taken water samples without elimination of the effect of chlorine dioxide and its decomposition on increase of chlorite concentrations.

7. Conclusion

Within the monitoring of disinfection by-products at the Nová Bystrica-Čadca-Žilina Long Distance Water Supply System the following substances have been determined: trihalogenmethanes ($CHCl_3$; $CHBrCl_2$; $CHBr_2Cl$; $CHBr_3$), dichlorophenols (2,3-DCP; 2,4-DCP; 2,5-DCP; 2,6-DCP), trichlorophenols (2,4,5-TCP; 2,4,6-TCP), perchlorophenol PCP, benzene and its derivates (benzene, chlorobenzene, ethylbenzene, 1,2-DCB; 1,3-DCB; 1,4-DCB; toluene, xylenes) a other organic substances (2,4-D acid, e.i. 2,4-dichlorophenoxyacetic acid; 1,1,2-trichloroethen; 1,2-dichloroethan; CCl_4; 1,1,2,2-tetrachloroethen), humic substances, alipahtic carboxyl acids (butanoic, pentanoic, hexanoic, heptanoic, octanoic, nonanoic, decanoic, undecanoic, dodecanoic, tetradecanoic, hexadecanoic and octadecanoic acids) and chlorites.



The results show that from the above substances only the following were determined over the detection limit: chlorobenzene, 1,1,2,2-tetrachloroethen, xylenes, toluene, chloroform, humic substances, chlorites, hexadecanoic and octadecanoic acids. The highest concentration was determined for chlorobenzene (6,6 µg/l that represents 66 % of limit under the Decree no. 151/2004 Coll.), chloroform (1,4 µg/l, 1,4 %), toluene (1,3 µg/l, 2,5 %) and 1,1,2,2-tetrachloroethen (1,0 µg/l, 10%). Aliphatic carboxyl acids reached the concentrations lower than 0,50 µg/l. The maximum concentrations of chlorites (NMH 0,20 mg/l) ranged from 0,30 to 0,35 mg/l were detected at sites where the chloride dioxide concentration is very low. Referring to own consumption of chlorine dioxide and course of its decomposition reactions in GSSW it was recommended for providers to use chlorine dioxide dosages in concentrations of 0,25 –0,27 mg/l at most and simultaneously to monitor microbiological and biological water quality, especially at stretches, where distributed water is disinfected only by chlorites. Based on evaluation of obtained results further steps will be proposed for water quality assurance in this GWSS.

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Water Treatment Technology Optimization on Aggressive Properties in Long-Distance Distribution System

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Abstract: The Stakčín water-supply reservoir serves as a water resource for long-distance distribution system with available water capacity of 45 mil.m^3. Surface water from water-supply reservoir is transported by gravitational force to the Stakčín WTP through the steel pipes of DN 1000 mm with the total length of 4471 m. The water is treated by clarification on gallery clarifiers and filtration through the sand rapid filters using 40 % ferric sulfate aqueous solution as a coagulant. Depending on treated water quality the dosage of ferric sulfate was ranged from 4 to 10 mg/l of calcium hydrate in the period from 10.1996 to 10.1997. Water disinfection has been carried out with regard to actual delay of water in pipeline by chlorine-amonisation with dosage of gaseous chlorine with the maximum of 0,4g and 0,2 g of ammonium sulfate. At the site of Ťahanovce housing development, the maximum iron concentrations at particular sampling sites reached the value range from 0,4 to 1,1 mg/l with the mean concentrations ranged from 0,20 to 0,31 mg/l.
Keywords: Optimalization, quality of water, water treatment, transportation

1. Introduction

The Stakčín water-supply reservoir serves as a water resource for long-distance distribution system with available water capacity of 45 mil.m^3. Surface water from water-supply reservoir is transported by gravitational force to the Stakčín WTP through the steel pipes of DN 1000 mm with the total length of 4471 m. The water is treated by clarification on gallery clarifiers and filtration through the sand rapid filters using 40 % ferric sulfate aqueous solution as a coagulant. Depending on treated water quality the dosage of ferric sulfate was ranged from 4 to 10 mg/l of calcium hydrate in the period from 10.1996 to 10.1997. Water disinfection has been carried out with regard to actual delay of water in pipeline by chlorine-amonisation with dosage of gaseous chlorine with the maximum of 0,4g and 0,2 g of ammonium sulfate.

The total length of the Starin-Košice long distance distribution system is 134,5 km and it is made of steel pipes with diameter of 1000 mm. The length of distribution system from the Stakčín WTP up to the water divider is 130,03 km. The water reservoirs along the entire distribution system are built at Snina, Humenné, Vranov nad Topľou, Medzianky, Prešov and Košice with the total capacity of 93 thousands m^3. The pipeline at the stretch from Stakčín WTP to Prešov water reservoir has been lined by bituminous coating (length of 98,4 km), the section between the Prešov water reservoir and Košice water divider is without inner coating (length of 31,6 km).

The actual delay of water in long-distance distribution system from the Stakčín WTP to Košice water divider is 133 hours with the WTP capacity of 313 l/s at a site below the Košice T2 water reservoir, with capacity of 570/l it is 100 hours and with 990 l/s it is 75 hours.

During the operation of long-distance distribution system, there has been observed water quality deterioration by effect of corrosion as a consequence of decreased water consumption and subsequent increase of iron concentrations in a pipeline. Considerably higher concentrations of iron in drinking water have been observed mostly at the end of pipeline in the stretch of the Košice- Ťahanovce water divider.

At the site of Ťahanovce housing development, the maximum iron concentrations at particular sampling sites reached the value range from 0,4 to 1,1 mg/l with the mean concentrations ranged from 0,20 to 0,31 mg/l.

Since the water quality has not reached a desirable level, the Water Research Institute has carried out operational evaluation of aggressive properties of water (corrosion tests) during a period 10.1996-10.1997. The work has been performed within the solution of stage task – “Stability of drinking water during distribution” included in VTP 514-78 “Research of drinking water treatability and environmental aspects of watercourses”. Regarding the existing water quality status of the Starina-Košice long distance distribution system as well as steel pipe lining by inner bituminous coating, the equipment for monitoring of aggressive properties of water has been installed in the following sampling sites:

- Stakčín Water Treatment Plant (WTP)
- Humenné Water Reservoir (WR) (34,8 km)
- Prešov Water Reservoir (WR) (98,4 km)
- Košice Water Divider (WD) (130,0 km)

2. Evaluation of the corrosion tests during the period 10.1996-10.1997

During the above period, six corrosion tests with 30 and 60-day exposition were valuated at each sampling site. Considering the duration of corrosion test, they have included all changes in quality of treated water entering the distribution system in connection with changes in quality of raw water flowing into the Stakčín WTP from water-supply reservoir as well as own operation of water treatment technology (dosage of ferric sulfate and calcium hydrate). During the corrosion tests, the long-distance distribution system transported water in volume of about 13 555 000 m^3. During the continuous operation it would represent the average water supply of 430 l/s.

The value range of selected treated water quality parameters during the corrosion tests was as follows: COD_{Mn} 0,9-1,4 mg/l, pH 7,5-8,1, ACIDITY 1,70-1,95 mmol/l, calcium 33-40 mg/l, magnesium 4,3-6,1 mg/l, iron lower than 0,09 mg/l, manganese lower than 0,04 mg/l, ammonium ions lower than 0,05 mg/l, nitrites lower than 0,01 mg/l, nitrates 2,9-3,8 mg/l, chlorides 3,2-4,3 mg/l, sulfates 19,2-39,4 mg/l, turbidity lower than 0,5 NTU and water temperature of 3,5-9,3°C.

During the first tree corrosion tests (10.1996–04.1997) the positive effect of sufficient $CaCO_3$ saturation of water on corrosion rates was observed in the entire length of pipeline. During the $CaCO_3$ oversaturation from 0,05 to 0,15 mmol/kg, the saturation indexes have reached values of 0,0–0,25. Corrosion rates for such stabilized water are shown in table 1.

Following the above results, water distributed by long-distance distribution system might be evaluated as slightly aggressive (aggressiveness category I) without need to propose anti-corrosion measures.

Table 1 Range of corrosion rates in the distribution system during the period 10.1996-04.1997

Period of corrosion test	Range of corrosion rates (μm/year)
10.-12.1996	21-66
12.1996-02.1997	26-64
02.-04.1997	43-61

The water temperature during that period was ranged from 3,5 to 10.1°C with observed increase in the long-distribution system of 0,6–1,3 °C. The iron concentrations reached maximum values of 0,15–0,23 mg/l at Pumping Station (PS) Hanušovce (77,6 km) and Section Valve (SV) no. 18 (85,9 km) respectivelly. At the Košice water divider they reached values of 0,15–0,19 mg/l. The dosages of ferric sulfate ranged from 7 to 10 mg/l and calcium hydrate from 3 to 8 mg/l.

During the next tree corrosion tests (04.-10.1997), the corrosion rates were ranged as shown in table 2.

Table 2 Range of corrosion rates in long-distance distribution system during 04.-10.1997

Period of corrosion test	Range of corrosion rates (μm/year)
04.-06.1997	46-105
06.-08.1997	58-98
08.-10.1997	37-78

The decrease in dosage of calcium hydrate to 2-3 mg/l with dosage of ferric sulfate of 6mg/l during the corrosion test 04.-06. 1997 resulted in insufficient $CaCO_3$ saturation of water (-0,03 mmol/l) in entire length of the long-distance distribution system. During the corrosion test 06.-08.1997 the dosage of calcium hydrate was increased to 4-5mg/l with unchanged dosage of ferric sulfate, e.i. 6 mg/l and it resulted in oversaturation of water at the Stakčín WTP by 0,05mmol/l, while in following parts of distribution system it has continually decreased up to 0,0 mmol/l at the Košice water divider. During the last corrosion test 08.-10.1997 the dosage of calcium hydrate was increased to 5-6 mg/l and ferric sulfate decreased to 4 mg/l that resulted in oversaturation of water to 0,05mmol/l at the Stakčín WTP and it reached balanced values on the level of 0,025 mmol/l in the whole distribution system.

The temperature was ranged from 6,2 to 11,2 °C and there was observed its increase from 2,2 to 4,5 °C. The maximum iron concentrations were observed mostly during the state of insufficient saturation and insufficient $CaCO_3$ oversaturation, respectively, at PS Hanušovce, SV no. 18 and Košice WD (0,23–0,30 mg/l).

As far as treated water stabilization is considered, it is necessary to point out the relations between calcium hydrate and $CaCO_3$ overasaturation as well as corrosion rates. The results of corrosion tests show that decreas of calcium hydrate towards ferric sulfate dosage results in lower saturation of water and there were observed also higher values of corrosion rates. The dosage of ferric sulfate depends on raw water quality and subsequently calcium hydrate dosage depends on ferric sulfate concentrations with regard to formation of optimum conditions for water treatment and assurance of their quality according to requirements on drinking water quality. For water quality treated at the Stakčín WTP it was necessary to take into account a possibility of water stabilization during the calcium hydrate dosage and thus for reduction of aggressive properties of water, especially during the period 04.–010.1997, when corrosion rates at the end of system exceeded the limit of 100 µm/year, there was proposed water treatment technology for achievement of calcium-carbonate balance and state of required $CaCO_3$ oversaturation of water for the purpose to form a protection layer with high affinity to pipe walls.

3. Water treatment technology optimization for assurance of water stabilization

The effect of water treatment technology optimization related to water stabilization at the Stakčín WTP was monitored during the period 02.–10.1999 and it was compared with the same period before technology optimization. (02.–10.1997). During that period four corrosion tests have been evaluated with 30 and 60-day exposition and corrosion devices were placed at the same sites of the long-distance distribution system as during the first corrosion tests (10.1996-10.1997).

Table 3 The mean dosages of lime and ferric sulfate during particular comparative corrosion tests

Period of corrosion tests: 02.-10.1997			Period of corrosion tests: 02.-10.1999		
Corrosion test	Mean dosage of lime [mg/l]	Mean dosage of ferric sulfate [mg/l]	Corrosion test	Mean dosage of lime [mg/l]	Mean dosage of ferric sulfate [mg/l]
02.-04.1997	2,6	7,8	02.-04.1999	4,4	7,3
04.-06.1997	2,7	6,0	04.-06.1999	6,2	7,6
06.-08.1997	4,2	5,8	06.-08.1999	5,5	5,2
08.-10.1997	4,6	5,0	08.-10.1999	6,2	4,3

In compared period 02.–10.1997 of the first corrosion tests, lime dosages were raged from 2 to 6 mg/l and ferric sulfate from 5 to 10 mg/l. However, such dosages were insufficient regarding achievement of needed $CaCO_3$ oversaturation of water, what resulted in higher values of corrosion rates. If the $CaCO_3$ oversaturation of water with a mean dosage of lime and ferric sulfate was ranged between –0,125 and 0,05 mmol/l, it means that water in long-distance distribution system was saturated insufficiently or insufficiently oversaturated by $CaCo_3$, respectivelly, and optimally it reached only low limit of range values included in STS (Slovak Technical Standard) 75 7151 (0,5–0,10 mmol/l). After water treatment technology optimization and also in view of water stabilization the values of water oversaturation reached the range between 0,025 mmol/l and 0,10 mmol/l, while the values of 0,025 mmol/l were determined only in particular cases.

While the mean dosage of lime during the corrosion tests 02.-10. 1997 with the mean dosage of ferric sulfate of 5,0 - 7,8 mg/l were ranged from 2,6 to 4,6 mg/l, the mean dosage of lime during the further corrosion tests 02.-10.1999 with the mean ferric sulfate dosage of 4,3 –7,6 mg/l were ranged from 4,4 to 6,2 mg/l. A comparison of lime dosage for corresponding periods of corrosion tests shows that lime dosages were increased 1,3–2,3 times after the optimization. The increase of lime dosage calculated to ferric sulfate dosage represented 0,27-0,55 mg of Ca(OH)/mg $Fe_2(SO_4)_3$, and higher values were observed especially during the summer period. The increase of lime dosage resulted in decrease of corrosion rates as well as decrease of iron concentration in particular sampling sites along the distribution system.

Figures 1–4 show effects of water treatment technology optimization for selected periods of compared corrosion tests at the Stakčín WTP considering water stabilization on the courses of water oversaturation by $CaCO_3$, saturation indexes and iron concentrations in the long-distance distribution system.

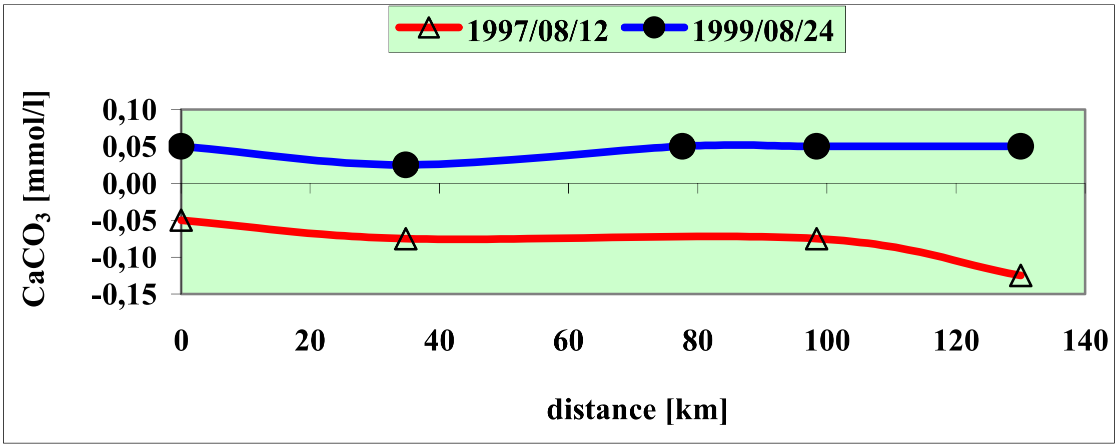

Figure 1 Process of water oversaturation by $CaCO_3$ in the long-distance distribution system for period of compared corrosion tests.

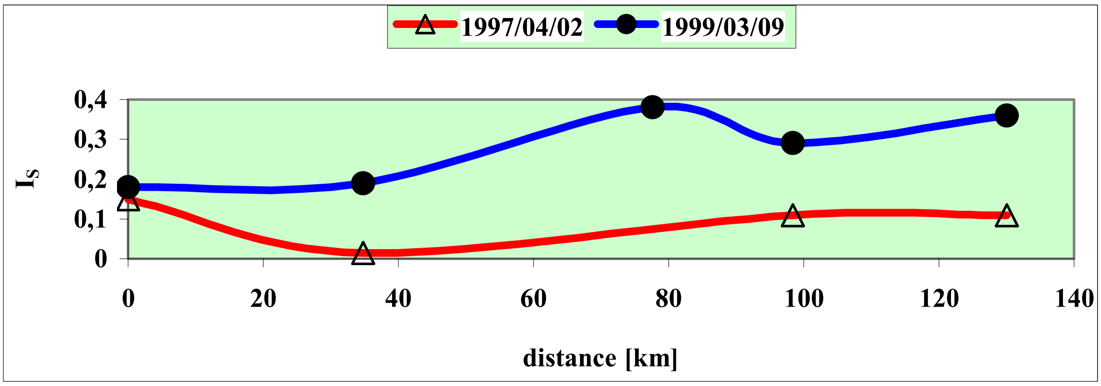

Figure 2 Course of saturation indexes in the long-distance distribution system for selected period of compared corrosion tests

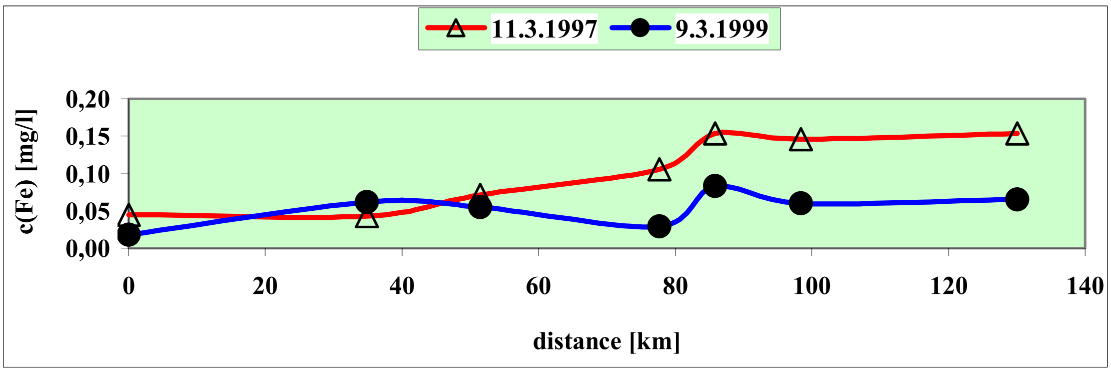

Figure 3 Iron concentration in the long-distance distribution system during the selected period of compared corrosion tests

During the corrosion tests after treatment technology optimization (02.–10.1999) the iron concentration reached maximum value of 0,17 mg/l (Košice water divider –04.1999). During all other samplings the iron concentrations were ranged to 0,10 mg/l at the end of long-distance distribution system.

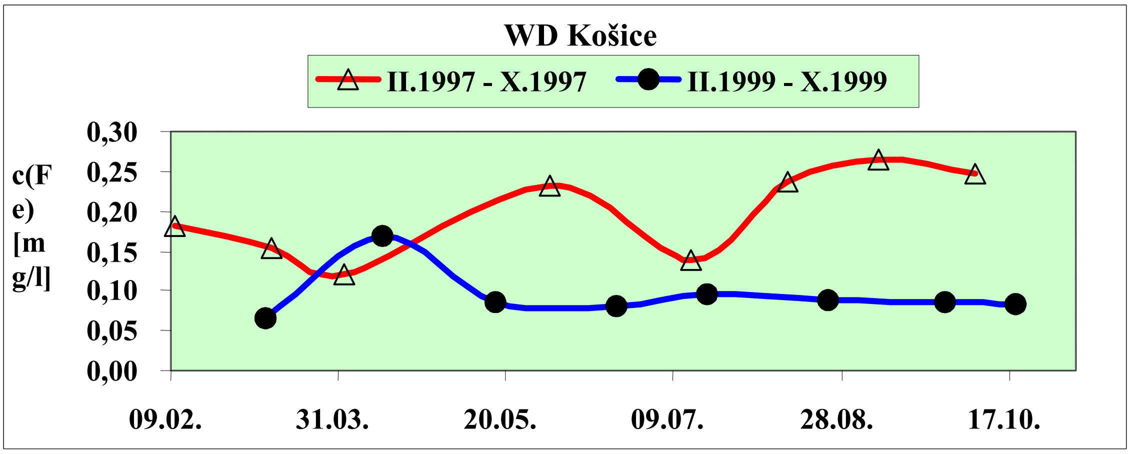

Figure 4 Iron concentration at the end of distribution system at the Košice Water Divider for selected period of compared corrosion tests

Corrosion rates after the treatment technology optimization have decreased during compared periods by 10 -75 % (02.–04.1997), 15-60 % (04.–06.1997), 5-45 % (06.–08.1997) and for the last corrosion test by 15-35 % (08.–10.1997). The range of corrosion rates for particular sampling sites during the corrosion tests 02.10 1997 and 0.2–10.1999 are shown in the table 4.

Table 4 Range of corrosion rates before (02.-10. 1997) and after (02.-10.1999) water treatment technology optimization at the Stakčín WTP

Sampling site	Corrosion rates [μm/year] 02.-10.1997	Corrosion rates [μm/year] 02.-10.1999
Stakčín WTP	45 - 78	31 - 55
Humenné WR	53 - 90	44 - 60
Prešov WR	61 - 83	70 - 80
Košice WD	42 - 105	10 - 54

The most significant decrease of corrosion rates have been observed at the end of long-distance distribution system, where decrease represents 35–75 % compared to values before water treatment technology optimization and the corrosion rates were ranged from 10 to 54 μm a year.

4. Conclusion

Based on evaluation of water treatment technology optimization at the Stakčín WTP and considering water stabilization it can be stated that these measures had considerable effect on decrease of aggressive properties of distributed water, what resulted in significant decrease of the iron concentrations as well as corrosion rates in the long-distance distribution system. Following the categories of aggressiveness, the water distributed by the Starina-Košice system was classified into the 1st category of aggressiveness (moderate aggressiveness of water).

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Accuracy of Measurement of Taken and Produced Water Volume

Optimisation Activities Carried Out at Saur Neptun Gdańsk

Zbigniew Maksymiuk, Andrzej Osiński, Roman Jurec

Introduction

Practical measurement of water quantity is carried out, as a rule, using indirect methods, by registering pressure drop or rotational speed of the mechanism of a driving device – vane-wheels or screw [1]. Direct methods, basing on capacity devices, are rarely applied, usually in control installations. Under circumstances of indirect measurement, possibility of achievement of reliable results depends on meeting additional requirements – first of all adjustment of the instrument to characteristics of the flow (limits of volume variation and time fluctuations) and observance of specific requirements regarding conditions of the instrument's installation [2]. In addition, there arises a problem of consequences of the dynamics of water intake over time. As a result, measurement range varies as well, the initial assumptions becoming outdated.

Measurement instrument, even of a high quality, does not provide a desired measurement accuracy if improperly used. Lack of keeping the requirements specific of measurement instrument, makes the instrument lose its measurement accuracy class [1]. The experience drawn, for instance, from household water meters, reveals that repeated errors are a commonplace and can reach the level of a few tens percent [5].

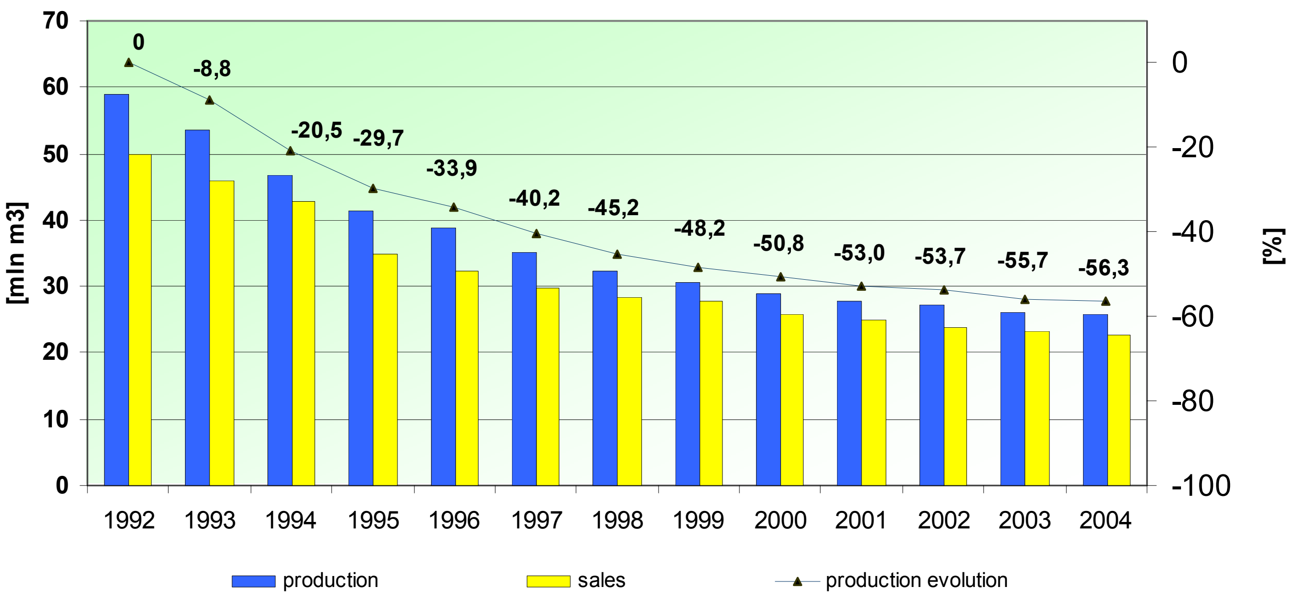
Water supply evolution

The main factor influencing operation of modern water supply and sewage discharge systems is systematic drop of water consumption and sales, and, consequently, deterioration of operating conditions, including system hydraulics. The phenomenon is, on the whole, an objective one, even though its intensity varies over time. In many countries its first symptoms could be noticed as early as in the 1970s, mostly in industry [4]. With consumption by households being still relatively high, total consumption was moderate [3]. The reasons in the background were mostly economic and, to a certain extent, environmental ones.

It is interesting to observe, that within the same period consumption at households was relatively high. Even not long ago, according to a number of forecasts, a certain rise of consumption at households was expected. Yet the final effect of water supply evolution is, actually, a reduction of water demand at all groups of customers. Changes of particular importance took place in the 1990s, and since they are mostly conditioned by technological factors, hardly can a reversal of the trend be reasonably expected [2].

In Poland, the reduction of water consumption process occurred relatively late. It is well-worth mentioning at that occasion that the particularly high figures indicated in the 1977 guidelines [6] never did actually prove as realistic ones, even in their formally reduced version of 1991 [7]. With political and economic transformation having started at the same time, impact of two factors (the economic and the technological one) became evident. Though water prices were made market ones not regularly all over the country, in Gdańsk the average water consumption in households, per capita, got reduced from 208 litres in 1992 down to 108 litres in 2004. Total production of water dropped down by more than 56% within the same period, although the drop slowed down over the last few years (Fig. 1).

Fig. 1 Evolution of water production and sales in Gdańsk from 1992 to 2005



Conditions of measurement

Conditions of measurement are dominated by consequences of the considerable drop of water consumption. Individual opinions may quite significantly differ in specific cases, yet with the drop of consumption by 50-70% against the initial state, incoherence of the applied instruments with the actual range of measurement is a natural consequence. Even greater disproportions can be observed at facilities designed and constructed for future development, particularly when based on the 1977 guidelines [6]. Also in their last version, the indicators stated there are unreasonable [7] (if not just fantastic, as German statistics coming from a comparable period [3] prove).

Yet another problem translating into measurement difficulties is the effect of quality of the taken underground water on condition of the measurement instruments. With supply reduced, relevant slow-down of the flow occurs and, consequently, iron and manganese compounds get precipitated in the sockets of well meters. The result is further deterioration of measurement quality. At the same time traditional angular water meters did not prove reliable in mating with pump sets. When the latter are being put into operation, there arises the phenomenon of water hammer, destroying the water meter mechanism. Yet another problem is the manufacturer's requirements for installation (spatial limitation). Hence measurement conditions can greatly differ from those optimum ones, significant deviations may occure as a result.

The above mentioned factors have contributed to measurement errors differing greatly at various water stations. In most cases errors are within the acceptable range of 5%, however at some stations it exceeds this level. The situation results, among others, in inadequate costs of water seized. According to the 2003 estimates, the amount is nearing PLN 100 000 (25 000€) – a sum significant for the company. At the same time the expenditure is totally unreasonable and bringing no benefits to the company and its customers. Considering that, there has occurred a need to change the policies regarding measurement of the taken volume of water and water production (registration of the volume of treated water supplied to the network).

Current rules of measurement

Considering the observed trends of water intake and results of analyses, actions aimed at improvement of measurement of water taken and supplied to the network have been started. The issue being a complex one, multidirectional efforts are required. It was decided, in particular, to:

- adapt pumping sets to actual explotation capacities of wells;
- introduce "soft start" devices into pump sets installed in the wells;
- replace measuring instruments together with the sockets with ones adapted to the volume and range of flows;
- replace instruments relatively new, but no more manufactured, spare parts being unavailable as a result;
- replace angular water meters mounted in the wells with traditional ones, installation-related requirements being observed;
- replace manual vents, as traditionally used at aerators and iron removers, with automatic ones;

- be guided by the rule that operating work is done cyclically, particularly as far as water meter calibration and control is concerned;
- clean water meter sockets on a regular basis;
- replace water meters at the exit from the treatment plant with flow meters or mechanical water meters of duly improved quality;
- strictly observe time limits for water meter calibration.

Actions realized for individual years are presented in tab. 1.

Table 1 Realization of actions aimed at improvement of measurement quality

Year	1996-2001	2002	2003	2004	2005
Replacement of water meters	259	50	85	50	50 - plan
Water meter cleaning		88	156	157	156
Reconstruction of water meter sockets			10	15	20
Cleaning of wells			10		10

Notwithstanding other actions, measuring of water consumption for various purposes within water intake was started (e.g. filters flushing and other technological needs, social needs etc.). It was decided to record the volume of water produced using electromagnetic flows. At the same time it was made a rule to permanently verify quality of measuring instruments. Indications of raw water meters are confronted with results of registration of water supplied to the network on a daily basis (readings done on a monthly basis have been used so far).

Exceeding of the 5% threshold of losses was adopted as the criterion for the actions taken. Meanwhile, at one of the water stations (Zaspa Wodna) a workstand was arranged for calibration of measuring devices using capacity-based method.

Effects and plans

Realized actions caused a considerable drop of the measurement error of the water meter mounted in wells. Thanks to a the measurement of all-kinds of water use within the treatment stations it is possible to precisely control the amount of water taken up and produced, which enables a quick detection of leaks or installation break-downs in particular at stations without a full-day operating personnel.

Aiming at further improvement of accuracy of measurement devices it is planned to continue actions of reconstruction of water meter sockets, water meter cleaning and calibration between legalization terms.

Moreover a full elimination of angular water meters is planned as well as introduction of electromagnetic flow meters or mechanical water meters of a higher accuracy level.

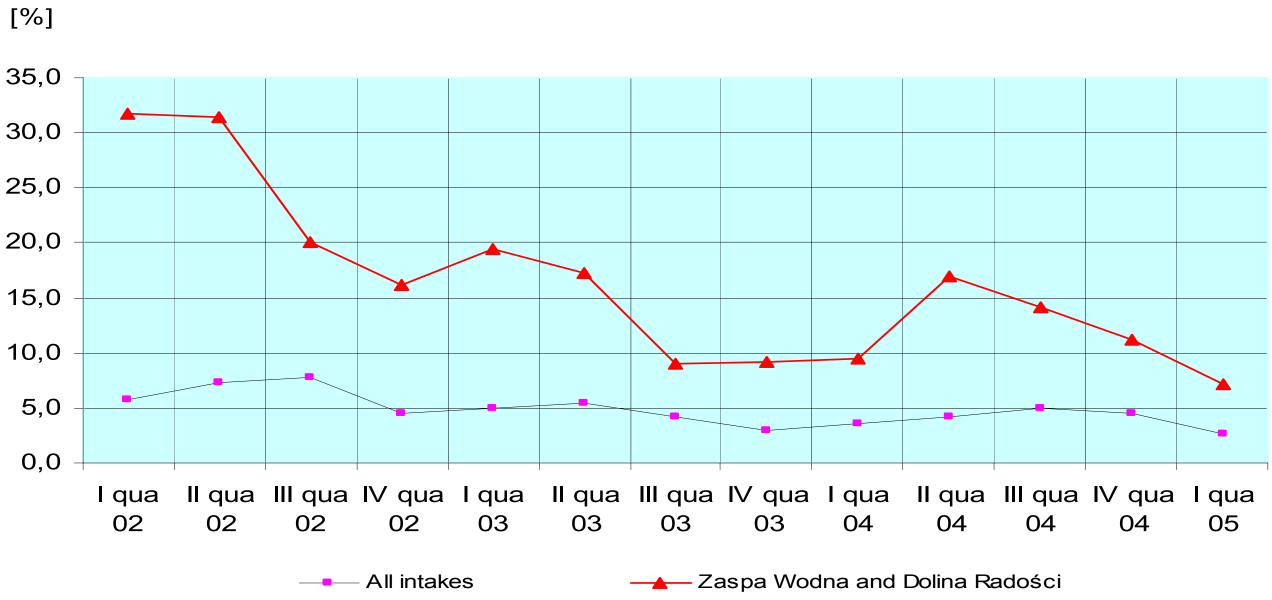
Picture 1 The water meters calibration workstand

Summary

The analyses carried out have shown that it is reasonable to take up actions aimed at improvement of accuracy of measurement of water taken up and produced at individual water intakes. In Gdańsk of particular importance in that respect are two intakes of water of poor quality and limited volume of water seized – Zaspa Wodna and Dolina Radości. A common feature of the facilities is the fact that they were designed in a way exceeding actual needs and, consequently, the quality of recording got poorer in their case. Now that the volume of water taken has got reduced, improvement of quality of water seizure and production has acquired a significant economic meaning. In 2002 errors in particular cases reached savings on the reduction of the charge "for intake" are estimated to amount to PLN 100 000 (25 000€) per year.

General rules of dealing with measurement instruments have been worked out. Replacement of the outdated instruments and adjustment of those to the actual range of hydraulic loads has been started. A schedule of work to be completed by 2005 was drawn up and basis for further control of the devices was created. It was made a rule to start the required actions earlier. The company's own calibration base was established, to deal with the instruments using capacity-related method.

Fig. 2 Average measurement errors of water meters at all graound intakes of Gdańsk and particularlyZaspa Wodna and Dolina Radości



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Hydraulic and Technological Consequences of Reactor Asymmetry

Jerzy M. Sawicki

Abstract: Reactors are usually designed as regular objects, what enables relatively effective operation of the system. But each reactor, even properly constructed, can lose its symmetry, what influences its efficiency, as it is shown in the paper. Two basic methods of reactors description have been considered – a generalised plug flow concept and dispersive model. Both theoretical and experimental results show, that the loss of the reactor regularity worsens its final efficiency, what has also serious financial consequences.
Keywords: hydraulics, reactors, waste disposal

1. Introduction

The fluid-flow technological reactors are usually designed as symmetrical, or at least regular, objects. This feature assures relatively uniform conditions of the process, which runs in the system. However in practice one always should take account for the possible deviations of the reactor geometry from the ideal state. These deviations could be caused by the low quality of work, but also by the unforeseen settlement of the ground.

Preliminary investigations show, that the departure from the symmetry of the fluid-flow object influences its hydraulic characteristics. For example [7], the asymmetrical distribution of a fluid in a pipe tee makes a difference among the components up to 3–4%. Lowering one section of the settling tank overfall may worsen its efficiency, also up to 5% [3].

So, it would be purposeful to investigate the consequences of such inaccuracies on the reactor performance. From the theoretical point of view, especially important would be determination of the system sensitivity to the symmetry loss. For the practical purposes in turn, one should rather look for the relation between these disturbances and the reactor efficiency.

The final results of such efforts strongly depend on the precision of the applied method of the reactor dimensioning. One can define two main groups of such methods:

- technical instructions (which, as a matter of fact, provide facilities only for "copying" of the existing objects);
- physical laws (which enables us designing *sensu stricto*).
- Among the physically based models one can distinguish:
- simplified calculation schemes (e.g. the plug flow model);
- (more or less) precise methods.

2. Reactor asymmetry – the plug flow model

According to the main assumption of the plug flow model, each mass element stays in the reactor during the same period of time:

$$t_{ps} = V / Q \tag{1}$$

(t_{ps} – mean detention time, V – volume of the reactor, Q – discharge). Assuming that the kinetics of the considered reaction can be described by means of the first order model, when [6]:

$$c_k(t) = c_p \exp(-k\, t) \tag{2}$$

(c_p, c_k – initial and final concentration of the reacting component, k – reaction constant, t – time), we can calculate the necessary mean detention time t_{ps} from the Eq. 2, for given reactor efficiency:

$$r = (c_p - c_k)/c_p \tag{3}$$

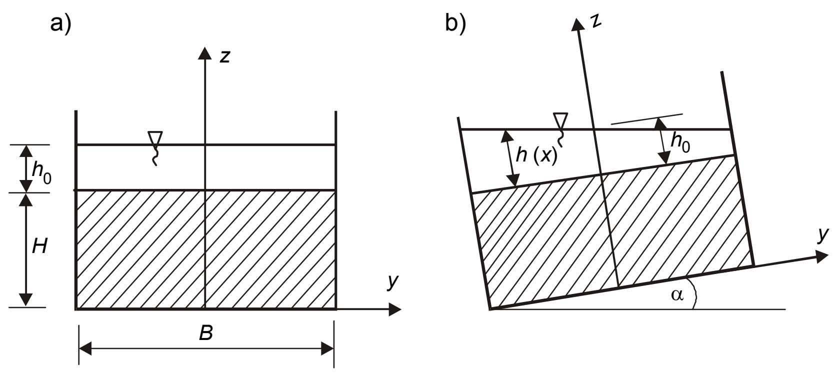

Fig. 1 Front-view of the rectangular reactor (a – regular position, b – position after the damage)

In the case of symmetry loss, the situation becomes more complex. Let us demonstrate this question on an example of a rectangular reactor. When the object is oriented in a regular (horizontal in this case) position (Fig. 1a), its discharge is equal to:

$$Q_o = (2/3)\mu B\sqrt{2g}h_o^{3/2} \tag{4}$$

and the velocity distribution across the reactor u_{xo} is so uniform, that can be replaced by the mean value (Fig. 2a):

$$v = Q/[B(H + h_o)] \tag{5}$$

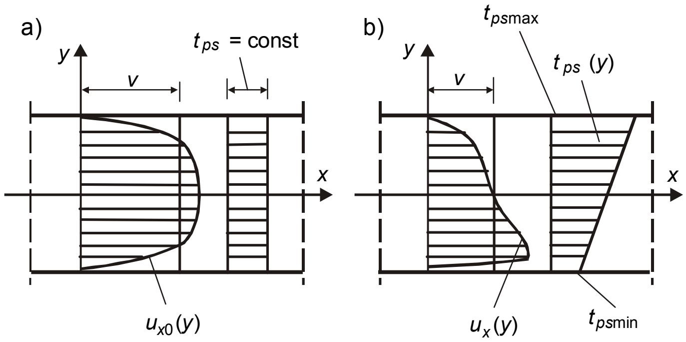

Fig. 2 Disturbances of velocity due to the reactor damage

Mean detention time t_{ps} = const. we can calculate from the Eq. 1, for $V = B\,L\,(H + h_o)$.

However when the reactor went under the damage (e.g. turned around the longitudinal axis – Fig. 1b), the depth of water would vary along the overfall, and the water flow intensity also would vary along the overfall crest:

$$\Delta Q(y) = \frac{2}{3}\mu\sqrt{2g}\ h(y)^{3/2}\,\mathrm{d}\,y\,. \tag{6}$$

The transversal velocity distribution in this case is given by the relation (Fig. 2b):

$$u_x(y) = u_{xo}[h(x)/h_o]^{3/2} \tag{7}$$

and the mean detention time also varies across the reactor $t_{ps}(y)$.

In consequence different parts of a selected (traced) mass element, which enters the reactor in one moment of time, will leave the system after different segments of time (from t_{psmin} to t_{psmax}). The terminal distribution of the tracer concentration is shown in Fig. 3a.

Taking into account the chemical reactions, which decompose the mass of the considered component, dissolved in the flowing water, according to the Eqs. 2 and 3 (Fig. 3b), its terminal concentration $c_k(t)$ can be presented as in the Fig. 3c. The measure of the resultant efficiency of the reactor can be expressed as follows:

$$r_{ef} = 1 - S_K/S_O = S_R/S_O \quad (8)$$

($S_O = S_K + S_R$ – surface areas, as shown in Fig. 3).

Assuming that L = 15.0 m, B = 3.0 m, H = 1,0 m, h_o = 0.05 m, Q = 0.058 m^3/s = 5000 m^3/d, μ = 0.60, k = 0.003 1/s, from the Eq. 1 one obtains t_{ps} = 815 s, what gives the reactor efficiency for the "normal" conditions r_n = 91% (Eq. 2). Considering a deformed object in turn (Fig. 1b), for α = 1.5° one gets t_{psmin} = 575 s, and t_{psmax} = 1286 s, what yields r_d = 87% (Eq. 8, Fig. 3).

This results mean, that even a slight deformation of the reactor has a negative impact on its efficiency. One has to note, that the calculated effect can be stated only under one condition – the plug flow model must be extended, so as to take into account the variability of the fluid velocity, at least across the object. In the simplest version of this model (Eq. 1) it would not be possible, as the total volumes of the regular and deformed systems are practically identical.

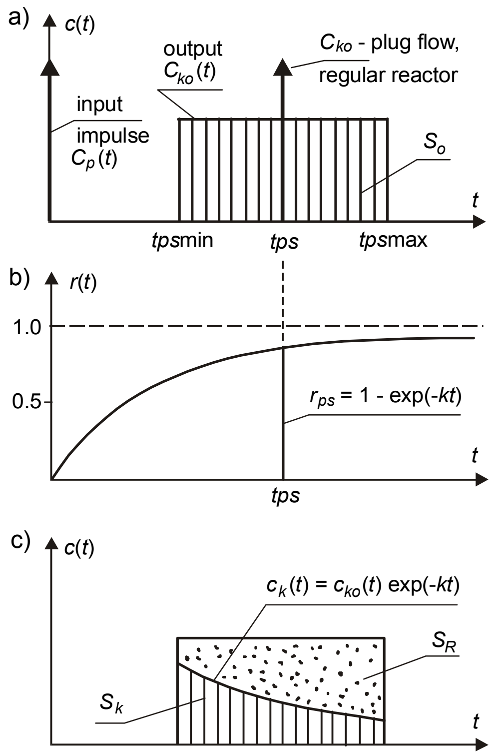

Fig. 3 Impact of the reactor failure (plug flow model)

3. Reactor asymmetry – the dispersive model

The plug flow model, applied above, is very popular, although very primitive, as it does not pay respect to the mass dispersion. This basic unit process can be described both theoretically and experimentally.

In the first step one has to determine the characteristic distribution of the detention time for the mass, dissolved in the flowing fluid. This information is given by the terminal concentration $c_{ko}(t)$ of the conservative substance, introduced into the reactor in the form of an impulse. Total mass of this substance equals:

$$M = \int_0^\infty c_{ko}(t)\, Q \,\mathrm{d}t. \tag{9}$$

In the theoretical attitude this function can be obtained as a solution to the unsteady advection-dispersion [6]:

$$\frac{\partial c}{\partial t} + (\vec{u}\nabla)c = \mathrm{div}(K_D \text{ grad } c) \tag{10}$$

(K_D – coefficient of dispersion). Determination of the advection velocity $\vec{u}$ is a particularly difficult element of this procedure, so one always has to introduce some simplifications, which sometimes go too far (e.g. evidently 3D velocity field is replaced by its 1D model).

Making use of the experimental method in turn, one has to measure the terminal concentration of the conservative tracer, introduced into the system as an impulse.

Assuming that the analysed process is a reaction of the first order (Eq. 2), the removed mass of the considered component equals:

$$M_R = \int_0^\infty c_k(t)\, Q \,\mathrm{d}t = \int_0^\infty c_{ko}(t)\,[1 - \exp(-kt)]\, Q \,\mathrm{d}t, \tag{11}$$

whereas the resultant efficiency of the reactor amounts (Eq. 8):

$$\mathrm{r_{ef}} = \mathrm{M_R / M = S_R / S_O}. \tag{12}$$

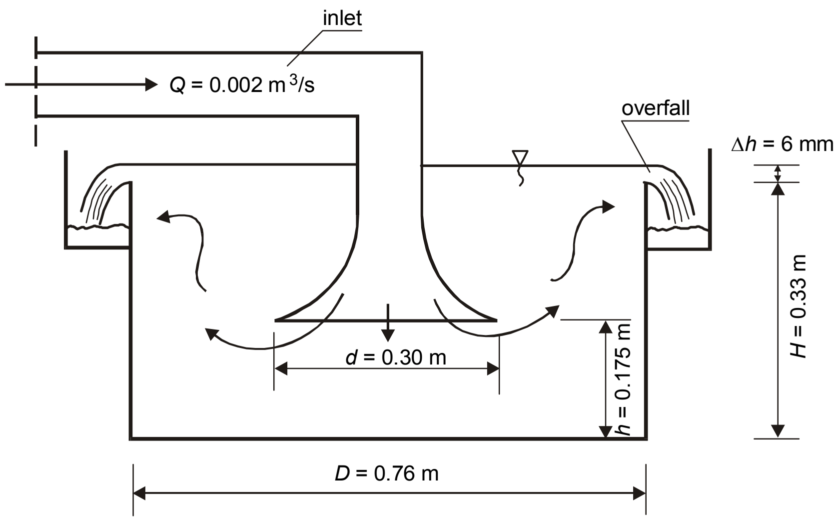

Fig. 4 Laboratory installation

As an example let us consider a cylindrical laboratory reservoir (Fig. 4). Computational concentration of the conservative tracer has been determined from the Eq. 10, for the 1D model of the velocity field:

$$u_r(r) = Q/2\pi rH \tag{13}$$

Initial tracer concentration was equal to c_p = 200 g/dm^3. Coefficient of dispersion was determined from the Elder formula [6], which in this case takes the form:

$$K_D(r) = 0.36\, H\, u_r(r). \tag{14}$$

The result of calculations is presented in Fig. 5, for the reaction constant k = 0.03 (1/s).

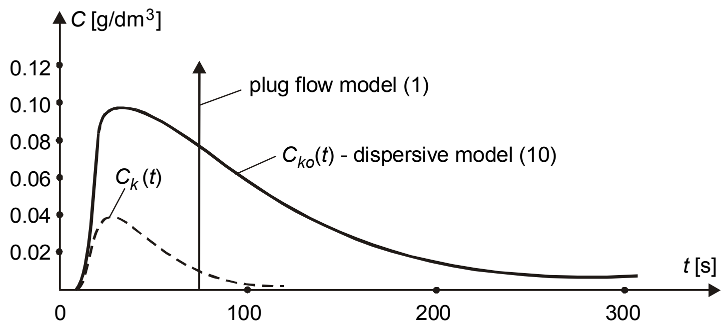

Fig. 5 Dynamic characteristics of the symmetrical reactor

Theoretical effective reduction of the pollutant in such a reactor equals r_n = 85%, whereas the same parameter, determined by the plug flow model – r_p = 89%.

In order to investigate the influence of the possible imperfections of the object, a laboratory experiment was carried out. A cylindrical reservoir (Fig. 4) was performed with a "technical accuracy" (i.e. without any special control, but also without purposeful faults).

During the experiment, for the steady flow the concentration of the impulsively introduced tracer (*NaCl*) was measured in six points around the reservoir perimeter. The results are presented in Fig. 6. As it is seen, the disturbances of the object regularity significantly influence its functioning.

In the next step, the efficiency of the real reactor was determined. The terminal concentration for each measuring point was recalculated by the Eq. 2 into the concentration of a degradable component, and then, according to the Eq. 8, the local efficiency for each sector were computed:

r_1 = 82%, r_2 = 88%, r_3 = 85%, r_4 = 77%, r_5 = 80%, r_6 = 81%.

The mean value is equal to r_d = 82%. As it is seen, this value is by three points worse than the effectivity r_n = 85% (symmetry), and by seven points worse then r_p = 89% (plug flow).

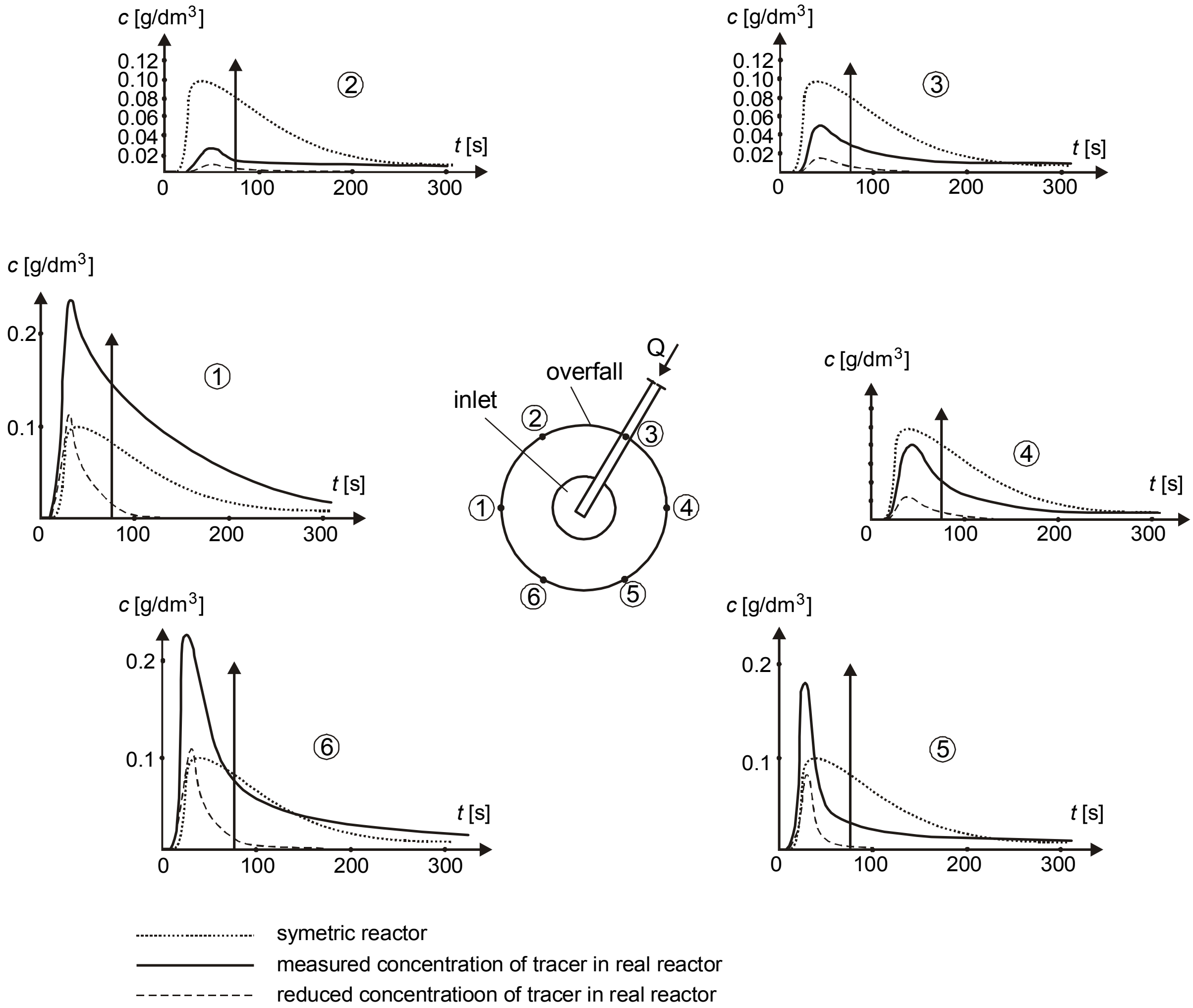

Fig. 6 Dynamic characteristics of the real reactor (considering imperfections)

4. Financial consequences

The superficial discussion of these results may lead to the conclusion, that the influence of the reactor inaccuracies could be neglected, as the resultant efficiency of the real object does not differ very much from the ideal and simplified values.

However one should realise, that in the present situation even one percent of the pollutants reduction has a measurable financial effects.

Let us consider a sewage treatment plant with a discharge Q = 100 000 m^3/d. For the initial *BOD* equal to c_p = 0.4 kg/m^3, let the level of reduction r = 89% fulfil the legal requirements. In this case the total organic load, discharged by this STP into the receiver equals L_{PF} = 4400 kg/d.

For the Polish conditions this means, that the twenty-four hours sewage rate for this legal discharge equals M_{PF} = 13 200 PLN/d (present unit value – 3,00 PLN/kg *BOD*).

Let us assume, that the considered object was designed (as usually) by means of the plug flow model. If so, the real reduction of pollutants for this case is equal to 85%, and the total load amounts to L_{RZ} = 6000 kg/d. So, the legal discharge would be exceeded in this case, and the total sewage rate would increase to M_{RZ} = 45 200 PLN/d (present unit fine – 20,00 PLN/kg *BOD* above the accepted level).

And finally, when the considered object does not work properly (e.g. because of the damage), when r_d = 82%, we have L_{AV} = 7200 kg/d and M_{av} = 69 200 PLN/d.

5. Conclusions

On the base of presented discussion one can state, that the proper dimensioning and realisation of the water and sewage treatment plants has a very important practical meaning, which can be expressed in financial terms.

So, during the process of designing one should apply the properly chosen computational methods.

However, even the most precise model does not guarantee, that the object will be free of some inaccuracies. In order to take this factor into account, it would be purposeful to determine the dynamic characteristics for each existing object, making use of experimental methods (e.g. tracer technology).

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Benchmarking of Water Supply Systems Focused on Water Losses

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Abstract: Modern methods of benchmarking are starting to be use for objective evaluation of water supply level, these methods provide relative comparison of attained results of water companies. Possibilities for improvement of companies create by comparison of selected indicators, such way we can define situation of company considering other similar companies.
Selection of figure of merits is the most important for benchmarking method to give desired results by comparison of them. Important indicators of evaluation of water supply are not only financial indicators, but technical too, technical indicators characterise technical condition of water pipelines, fault liability and with these related water losses. Water losses are significant factor, which affects inefficient exploitation of quality water sources.
Management of water company needs to obtain information about selected indicators to can apply benchmarking methods. For that purpose benchmarking centres are starting to institute in the world.
In this report will be presented new approach of evaluation of water losses indicators from point of view operator of water supply.
There will be described benchmarking method which can be use for evaluation of water supply systems oriented for water losses. In this article will be presented results of evaluation of water supply systems in Slovakia.
Keywords: water supply, water losses, water losses indicators, non revenue water, benchmarking.

1. Introduction

Modern methods of benchmarking are starting to be use for objective evaluation of water supply level, these methods provide relative comparison of attained results of water companies. Possibilities for improvement of companies create by comparison of selected indicators, such way we can define situation of company considering other similar companies.

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In this report will be presented new approach of evaluation of water losses indicators from point of view operator of water supply.

2. Method of Benchmarking

Benchmarking as an important tool of management was first time applied in company XEROX in 1979. From this time method of benchmarking started to apply in different regions and in different levels. There are exist several definitions of this method:

"Benchmarking is continuous and systematic comparison of own (self) efficiency in productivity, quality and production process with companies and organisation which have (peak power) high production."

"Benchmarking is application of systematic methods for comparison themselves with other and finding better ways how to make this work."

"Benchmarking is monitoring other of purpose to learn from them."

Main goal of benchmarking is to build on successful experience of other instead inventing of wheel again. This idea is simple: the most productive way for implementation of change is to learn from positive experiences of other companies. Realisation of benchmarking with the best companies in similar type of activities can help to find way of achievement of success to organisation. Benchmarking is kind of management to accent systematic improvement.

"Learning organisation" try to keep up contact with up-to-date and the best practices in the field by application of benchmarking instead of dependence on out-of-date ideas. Benchmarking is always practised to implementation of improvement. Analyse can be focused on products, processes and/or results. Organisation obtains information for improvement and development what can lead to improvement of efficiency. Benchmarking is not only process of creation, classification and comparison data gathered but it is dynamic process of information exchange to be effective tool (instrument) of change to be better.

Slovak benchmarking information centre (SBIC) like national centre intra-Europe and global network of benchmarking centres was instituted at Ministry of Economy of the Slovak Republic in 1999 in Slovakia.

Main plan of SBIC (www.sbic.sk) is to spread information about benchmarking – important tool of management, to achieve fast increase of competitiveness especially industrial plants.

3. Benchmarking of Water Companies

Acquirement of comparative information and their application in benchmarking became important managing tool for managers in water companies and plants too. First provider of benchmarking for water companies of all the world is IBNET – Water and Sanitation International Benchmarking Network which mediates opportunity for sharing of regional, national and international information. (www.ib-net.org.) It is dynamic database what is continual filling up.

World bank under the name "Benchmarking Start – Up Kit" started with this initiative. On October 2003 Wrc plc (Water research centre, UK) obtained contract for control and development of this initiative specified as IBNET. IBNET provides possibility to create trustworthy global benchmarking network for water companies in developed and developing countries. IBNET will be mediate fast entrance to data in water sector by application presentation of base indicators oriented to web sites of "geographic points".

IBNET helps to share especially international information to support of local benchmarking activities. This tendency has following reasons:

- it is hard to come to an agreement on universal indicators and their detailed definition
- availability of trustworthy data can be limited
- comparison between countries can be effected by different conditions of operations

Because of this was designed central monitoring system – each owner of water distribution system will build up its own monitoring capacities and his data will be made public by internet on the base of voluntaries. If these informations are from users in adequate number, there will be added reference international data to each of the user. IBNET was developed for support of this concept what was described above. IBNET contains:

- main indicators, by means of these indicators operators can build their own measuring and monitoring system
- complete list of these indicators with their definitions
- representation of data – computation
- way to sharing of information

Main benchmarking indicators and their units are still exposed to extensive discussion to obtain minimisation of regional and local influences, which can distort their cancellation value. Because of this it is recommended to express water consumption not only as litres/inhabitant/day but in litres/household/day too because number of households is more accurately entered.

3.1 Benchmarking realised at Operators of Water Infrastructure in Slovakia

On the Department of Sanitary Engineering, Faculty of Civil Engineering, Slovak University of Technology in Bratislava within the scope of solution of grant research work we are making benchmarking of technical figure of merits of water companies. After analyse of availability, representatives and authenticity of base data, technical indicators of water supply and water losses were selected at first analysis stage of solution for comparison. Indicators applied in IBNET could not use for their unavailability.

Applied method of benchmarking make possible to find out momentary state of each water companies compared with others at home and foreign market, this method leads to obtain of new information stimulus and motivate to improve of present stage. Results will be given anonymously, each company has allotted serial number.

At second analyses stage of task which is being smaller operation units like water supply systems of cities and villages in different size categories considering water losses will be submitted to benchmarking. Goal of this task is comparison of water companies with foreign companies. Task of this pilot project is incorporation of Slovakia to world-wide global partnership of network IBNET.

3.1.1 Applied Method

For benchmarking it is usually used applied anonym list of questions for collection of data. Collection of data for comparison selected indicators of water losses in Czech Republic was realised the same way, which organised SOVAK in 2002. In our condition it was not possible to obtain complete background and we had to use database of main indications of water companies, which are published at annual reports. Data during years 1999-2003 were analysed.

Indicators of water losses

International organisation IWA published new approach to evaluation of water losses and in 1996 assigned Task Force to make transparent existing techniques for international comparison of water losses in water supply systems.

Following indicators are recommended to use for evaluation of water losses:

- Unit leakage of non-revenue water:
  - Volume of non-revenue water / total length of pipes (m^3/km/year; l/km/day)
  - Volume of non-revenue water/number of consumer lines (l/service connection/day)
- Infrastructure Leakage Index ILI
  - Volume of real water losses / volume of theoretically necessary water losses (non-dimensional number)

4. Results

Volume of non revenue water, showed by water companies is more reliable indication as showed water losses and so indicators determined with volume of non revenue water were selected for comparison. Index ILI was checked the same way.

Computed indicators for 6 water companies in Slovakia during year 2002 were in the following range:

- unit leakage per km 3 600 – 8 000 m^3/km/year
- unit leakage per consumer line 260 – 700 l/service connection/day
- non-dimensional index ILI 4,5 – 12

Suitable technical condition means water distribution systems with unit leakage up to 4500 m^3/km/year.

From evaluation of work group of IWA for water losses Infrastructure Leakage Index ILI was at intervals 0,8 – 11 for 27 water companies (of the world). If ILI >1,0, it is assumption to intensify effective control of water losses or to speed up repair.

In the figure 1a-d are showed data of selected indicators of water losses per year 2002 for water companies marked by numbers 1-6. Water companies sequence and sign are the same in the chart.

By now data from two selected water companies were prepared in more detailed. Data were prepared for various water supply systems, that were typed to 6 categories according to supplied inhabitants. Averaged values of indicators of water losses for created quantitative categories are presented in Table 1.

Table 1 Average values of indicators of water losses in dependence on number of supplied inhabitants.

Number of supplied inhabitants / indicators		to 1000 inhab.	to 2000 inhab	to 5000 inhab.	to 10.000 inhab.	to 50.000 inhab.	up 50.000 inhab
non-revenue water	%	32,3	31,8	30,9	22,5	27,2	24
unit leakage per km	m^3/km/year	2136	2382	2948	3405	4790	4720
unit leakage per servis connection	l/ser.conn./day	172,3	183,3	204	216,3	365,4	402,9
ILI	-	3,1	3,5	4,1	4,4	6,6	7,2
recovery of pipeline	m^3/m	6,6	7,5	9,5	15,1	17,6	19,7

Summary

Objective view about condition of water supply make only indicators for individual water supply systems (local water distribution or group water supply), e.g. where index ILI exceeds value 40. This will be achieved after evaluation of water supply systems. Benchmarking give possibilities to management of company to compare more selected indicators with

international indicators at the same time and so to identify field which need greater care. Demonstration of benchmarking results of water losses in selected water companies will be presented on the Conference.

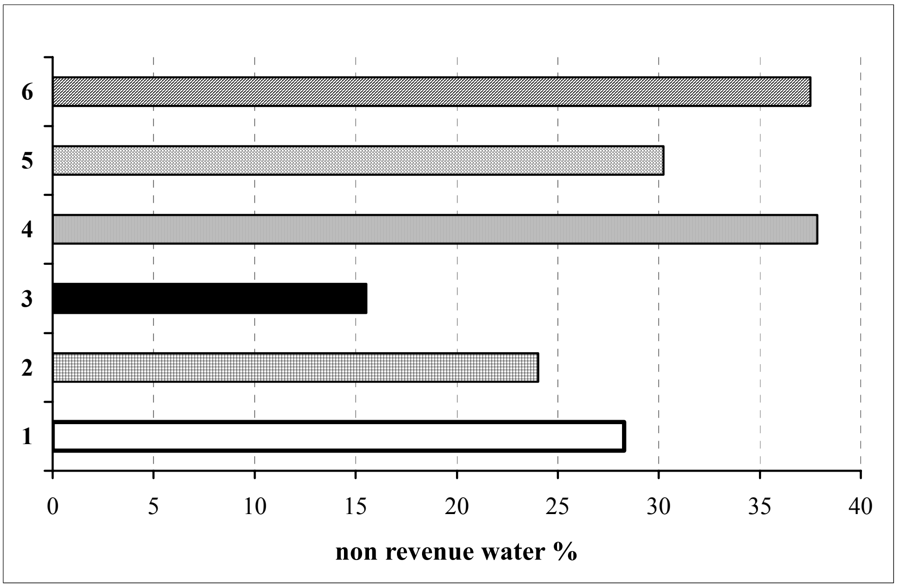

Figure 1a) Values of non revenue water of water losses for water companies 1-6 per year 2002

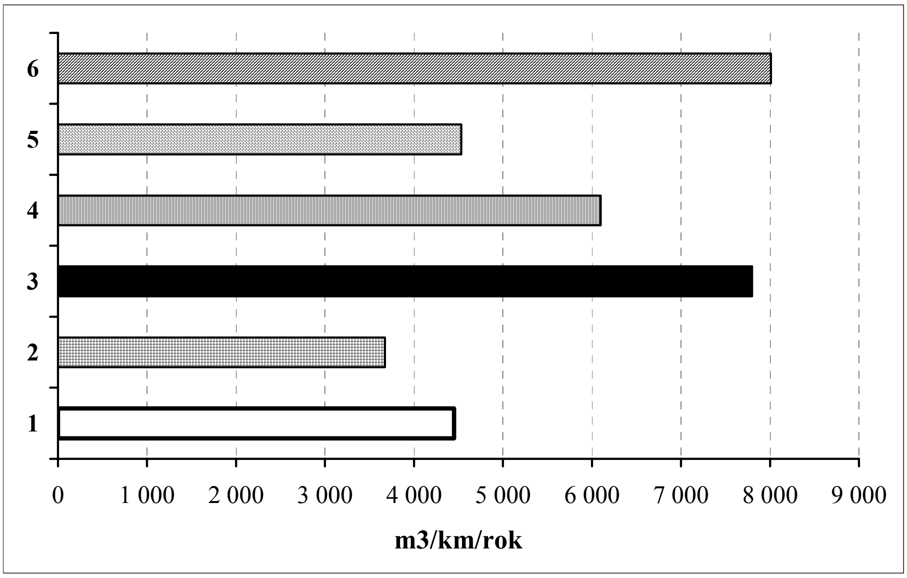

Figure 1 b) Values of indicator m^3/km/year of water losses for water companies 1-6 per year 2002

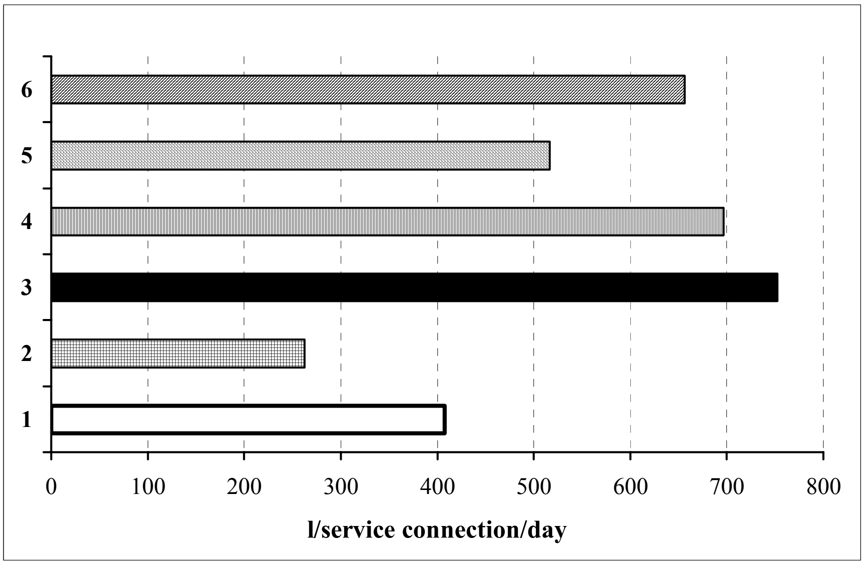

Figure 1c) Values of indicator l/serv./day of water losses for water companies 1-6 per year 2002

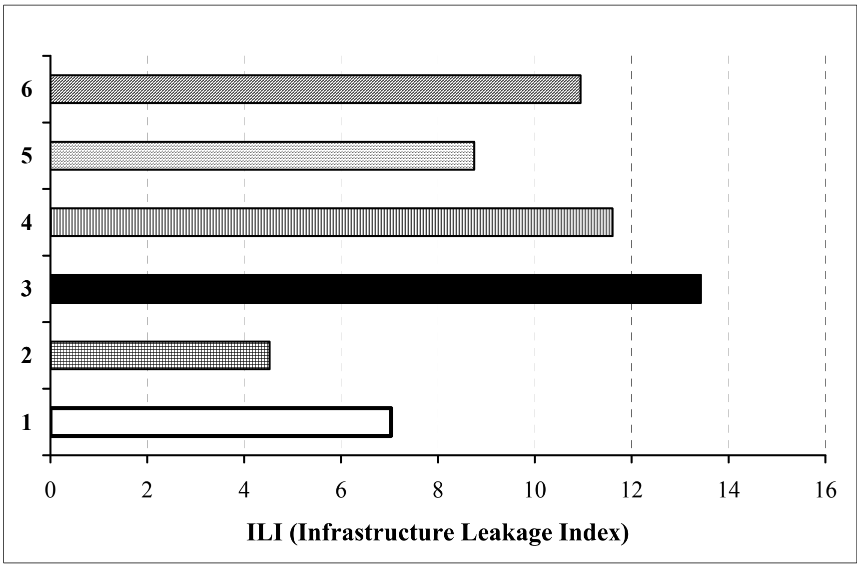

Figure 1d) Values of indicator ILI of water losses for water companies 1-6 per year 2002

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Paper №: **IV.07**

Wastewater Aeration Using by Plunging Water Jet Aerators

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Goran Gjetvaj
Davor Malus

Abstract: This paper gives wastewater aeration analysis by implementing the "plunging water jet" aerators. To clarify the characteristics of such aeration process, physical model was designed consisting of: inlet chamber with constant water surface level and jet-designed outlet (nozzles), and receiving pool representing biological reactor. Variational parameters were number of nozzles, their effective diameter and angle, as well as flow and velocity of resulting water jets.
Besides measurements of hydraulic parameters, the aim was to determine dissolved oxygen enrichment in biological reactor, depending on specific nozzle parameters. One of the goals was to find the optimal combination of such parameters that would result in maximum aeration rate. The study was carried out using the potable water from public water supply system.
Based on resulting aeration efficiency it is possible to estimate the possibility and adequacy of plunging water jet aerators application within the specific site conditions. The final purpose was to reduce the total construction, operational and maintenance costs of wastewater treatment plants, concurrently increasing their treatment efficiency.
Keywords: wastewater treatment, dissolved oxygen, aeration, plunging water jet, biological reactor

1. Introduction

Dissolved oxygen is limiting factor throughout the life cycle in aquatic ecosystems. Concerning the wastewater disposal to water bodies, oxygen enrichment is of significant importance. Natural (aquatic-based) wastewater treatment systems are becoming relatively common practice all round the world. Three most frequently used forms of such systems are lagoons, stabilization ponds and constructed wetlands. Treatment processes involved in these systems are similar to those in natural water bodies (river, lake, sea, etc.). For that reason it is necessary to provide enough detention time of wastewater in the system. Low construction, operational and maintenance costs, construction simplicity and high treatment efficiency make these a good alternative in sanitary wastewaters treatment, especially in small and decentralized rural areas. Their successful application could be expended in highway runoff treatment.

Limited finances of Croatian municipalities are restrictive factor to conventional wastewater treatment plants construction. So, the emphasis is given to alternative, economically beneficial solutions with satisfactory treatment efficiency – natural treatment systems (biological reactors). Enough dissolved oxygen concentration for organic matter oxidation is the key element that will provide successful and effective operation for different forms of treatment systems as well as for natural biological reactors. Next to the natural sources for dissolved oxygen including surface reaeration and photosynthesis, there are several ways to satisfy the required oxygen demand. Due to technological development, numerous solutions and possibilities for dissolved oxygen enrichment in water bodies are present nowdays. Artificial aeration systems in the form of physical facilities such as diffused-air aeration, mechanical aerators etc., are used to improve the overall treatment processes, but require additional power sources which results with increased construction, operational and maintenance costs. Therefore, there is much emphasis on examination of alternative methods for wastewater aeration in biological reactors. Such alternative methods are easily applicable in hilly areas by using available energy in the form of naturally available hydraulic head.

Aeration properties of different hydraulic structures have been investigated for decades. Baylar and Bagatur (1999) investigated the aeration efficiency at sharp-crested weirs having different cross-sectional geometry. Gjetvaj (2005) analyzed the water aeration rate by discharging it over the filter bed (passive aeration pump system). Numerous studies were also conducted observing water jet aeration efficiency (Sene 1988, Bin 1993, Chanson 1995, Cummings and Chanson 1997, Chanson and Brattberg 1998, Chanson and Manasseh 2003).

The present paper describes new experiments into the performance of jet aeration. Aeration experiments were conducted using a hydraulic model designed in the Hydraulic Laboratory at the Faculty of Civil Engineering, Zagreb, Croatia. Model describes the characteristics of plunging water jet aerators and their oxygen enrichment rate. In comparison with earlier investigations based mostly on impact of jet-designed outlet (nozzles) geometry and jet (drop) high, this study investigates dissolved oxygen enrichment in natural biological reactors in dependence of the following parameters - number of nozzles, their effective diameter and angle, as well as flow and velocity of resulting water jets. Every single change of given parameters results with different aeration rate (air-entrainment efficiency). Besides possibility and adequacy estimation of new aeration method (plunging water jet) application, physical model measurements should have resulted with revealing the optimal combination of such parameters that would result in maximum aeration efficiency.

2. Experimental setup

Given analysis is based on the experimental results from physical model specially designed for that purpose. During the experiments, dissolved oxygen concentrations were measured continually with goal of finding out its dependence on variational parameters. Defining the optimal values of such parameters, one would strive to achieve as favorable aeration conditions concurrently increasing wastewater treatment efficiency. Each experiment was carried out using clean water from public water supply system. Main reasons were simplicity of modeling and avoidance of daily discharge fluctuations. Concerning the main aspects of given analysis, it should be pointed out that wastewater has less oxygen solvent ability comparing to clean water. Sanitary wastewater contains soaps and detergents that cause such bounding surface between air bubbles and water resulting with lower aeration efficiency. Consequently, this fact needs to be considered during the computed results analysis and the final conclusion drawing.

2.1. Model description

Natural biological reactors are appropriate method for wastewater treatment in rural areas, dislocated from urban centers. Considering this fact, experiment was carried out under flow rates varying from approximately 1,5 to 2,5 l/s. These flow rates suit the settlements with population number from 850 to 1500 (according to ATV recommendations with specific flow rate of 150,0 l/person/day). With reference to given flow rates and selected size of nozzles, resulting plunging water jet velocities varied between 0,75 and 4,1 m/s. Thompson weir, as one of the most convenient apparatus for low flow rates measuring, was used for that purpose. Measured flow values were then used to calculate jet velocities. Designing the model in scale 1:1, allow direct application of computed result. Model consists of following elements:

- inlet chamber
- jet-designed outlet (nozzles)
- biological reactor

Inlet chamber was used to develop such a head to provide required jet velocities and steady-state regime. Concerning the main goal to determine dissolved oxygen enrichment in biological reactor, water preaeration before reaching the reactor had to bi avoided. Therefore, supply pipes had to be submerged into the inlet chamber and during the initial state of inlet chamber loading special care was taken to reduce the turbulence that might cause water preaeration. Filling the inlet chamber with water was carefully regulated using the flow-control valves.

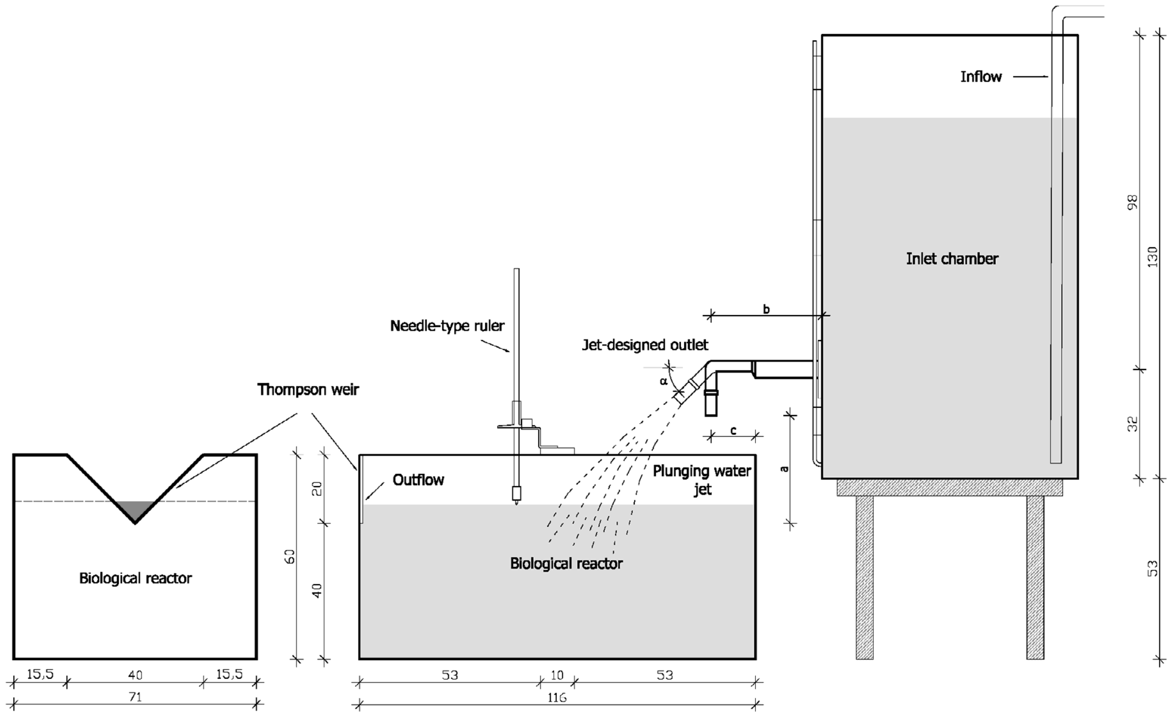


Figure 1 Physical model scheme

Inlet chamber outlet is specially designed to form jet inflow into the biological reactor. That jet-designed outlet is shaped of PVC material (pipes and fittings) and is fixed near the chamber bottom. Outlet dimensioning was performed while keeping in mind two parameters. First was endeavoring to achieve as higher jet velocities, but at the same time to reduce the possibility of pipe clogging (concerning the fact that wastewater contains suspended solids that might settle down and cause pipe clogging). When a water jet impinges downstream water pool (biological reactor) its velocity impacts bubble penetration depth. Higher jet velocities result with deeper penetration and greater turbulent mixing, consequentially increasing oxygen transfer rate. Important factor is necessity to ensure some form of mechanical pretreatment (septic tanks, settling tanks, grit and grease separators), before the wastewater reaches the natural biological reactor. Therefore, outlet pipes diameter shouldn't be smaller than Ø28mm. Following this proposition, outlet pipes Ø28 and Ø50 mm were chosen.

Biological reactor is shaped in elongated parallelepiped form, with one end facing towards the inlet chamber (jet-designed outlet) and other is formed as Thompson weir.

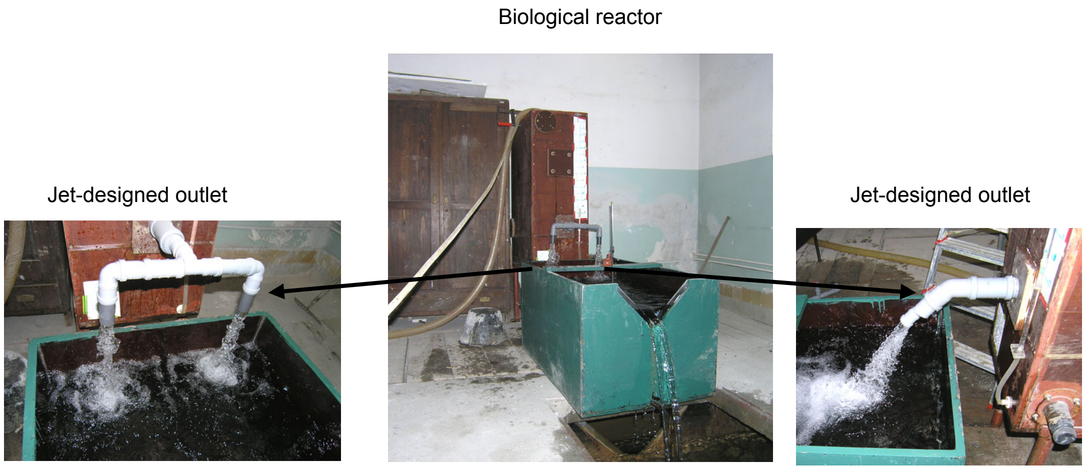


Figure 2 Detailed view of jet-designed outlet

2.2. Measurement program

All investigations were carried out with purpose to determine plunging water jet influence on dissolved oxygen enrichment in biological reactor. Modifying different outlet elements it was trying to find its optimal shape, in fact, find a solution that will result with maximum aeration efficiency. Parameters that were changed during experiment are number of nozzles, their effective diameter and angle, jet flow and velocity. Every change of these parameters describes one scenario whose results have been processed separately. Total number of processed scenarios within the given investigation was 54 (Table 1). One goal was to find optimal combination of such parameters that would result in maximum aeration rate.

Table 1 Analyzed scenarios

Scenarios	Number of nozzles	Nozzle size (diameter), (mm)	Jet flow rate (l/s)	Jet angle
1 - 18	2	28	1,5; 2,0; 2,5	90, 75, 60, 45, 30, 15
18 - 36	1	28	1,5; 2,0; 2,5	90, 75, 60, 45, 30, 15
36 - 54	1	50	1,5; 2,0; 2,5	90, 75, 60, 45, 30, 15

Each element of the model was designed to fit the exact size of natural wastewater treatment system defined with 850-1500 population equivalents. The whole model was sized to provide as higher jet velocities with resulting flow rates between 1,5 and 2,5 l/s. Selected flow rates were controlled by different water levels in inlet chamber.

Variational water levels involve different jet velocities. Flow rates were measured using Thompson weir and needle-type ruler that was positioned at the middle of downstream pool.

Oxygen transfer (aeration) efficiency could be expressed as (Gulliver, 1990):

$$E = \frac{C_b - C_u}{C_s - C_u} \tag{1}$$

where numerator describes dissolved oxygen (DO) concentration increment ΔC (mgO_2/l) expressed as the difference between measured DO values in biological reactor (C_b) and inlet chamber (C_u), and denominator describes initial DO deficit expressed as difference between saturation concentration (C_s), at which equilibrium with the gas phase is achieved, and upstream DO concentration in inlet chamber (C_u). Saturation concentration is function of water temperature, salinity and barometric pressure. Due to negligible changes in water temperature on the model, constant saturation value C_s was assumed, taken from literature (Metcalf & Eddy, 1991). At water temperature 12,7ºC, selected saturation value is 10,6 mgO_2/l. Previous investigations have shown that aeration efficiency is sensitive to water characteristics, primarily its temperature and quality. Temperature effects on oxygen transfer rate are defined with temperature correction factors. Gulliver (1990) applied previous theoretical discussions and developed the relationship:

$$1 - E_{20} = (1 - E)^{1/f} \tag{2}$$

where E = transfer efficiency at the water temperature of measurement, E_{20} = transfer efficiency at 20ºC. The exponent f is described as follows:

$$f = 1{,}0 + 0{,}02103(T - 20) + 8{,}261 \cdot 10^{-5}(T - 20)^2 \tag{3}$$

It was already mentioned that investigation was carried out using potable water from public water supply system. Thus, daily fluctuations of certain input parameters were avoided. In addition, water quality effects on aeration efficiency could've been ignored. Measuring DO concentrations in inlet chamber, insignificant daily oscillations were recorded ranging between 5,50 and 6,75 mgO_2/l.

Dissolved oxygen concentration measurements were taken using digital oxygen meter. Meter was calibrated, prior to each use, following the procedure recommended by the manufacturer. Dissolved oxygen was measured at several different points, selected to best represent the contents of the tank (Figure 3). All measurement points are common to each scenario. Concerning the same point location, measurements were taken at two depths (5,0 and 30,0cm below the water surface). With purpose of reducing the probability of mistake, multiple measurements were taken for each point (including premeasurements in inlet chamber). Therefore, the reliability of the final results increases.

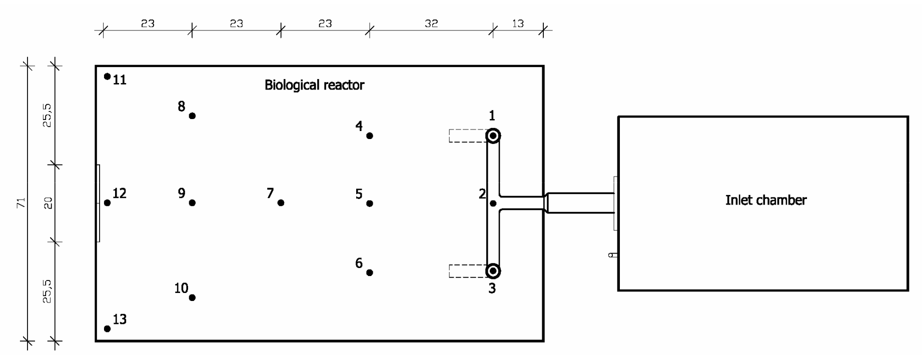
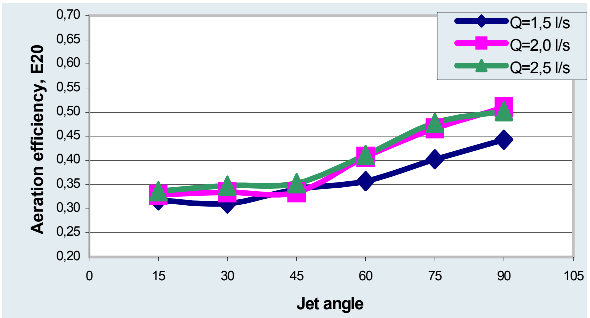


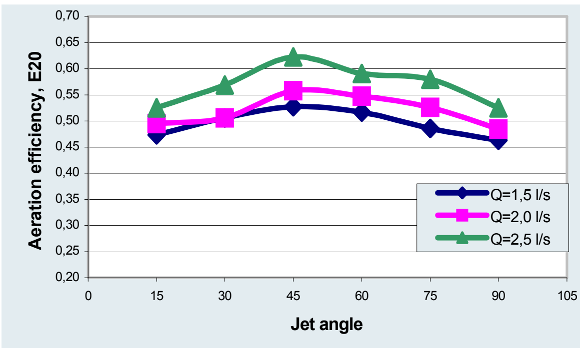
Figure 3 Measurement points layout

2.3. Results

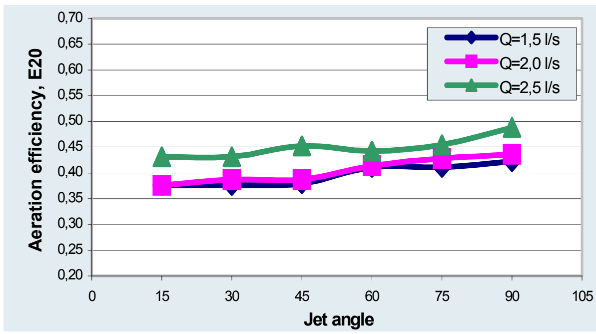
The results of experiments are referred to measured DO concentrations in biological reactor. Obtained measurements have shown aeration efficiency of plunging water jets. Based on measured values, oxygen transfer efficiency, E_{20}, was calculated using equations (1) and (2). Adopting efficiency value, E_{20}, as output parameter is yielding an easier comparison with other aeration methods or with results achieved in different circumstances (different water temperatures, etc.).



a) 2 x Ø28mm



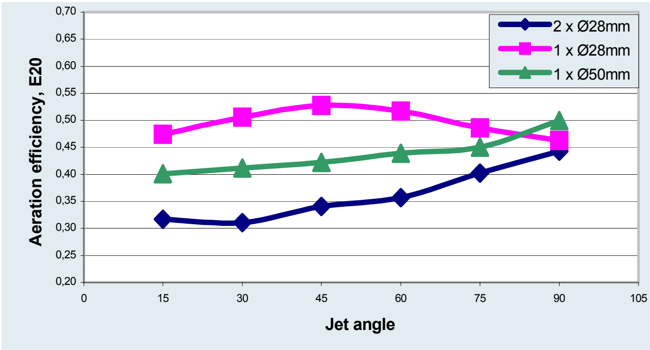
b) 1 x Ø28mm



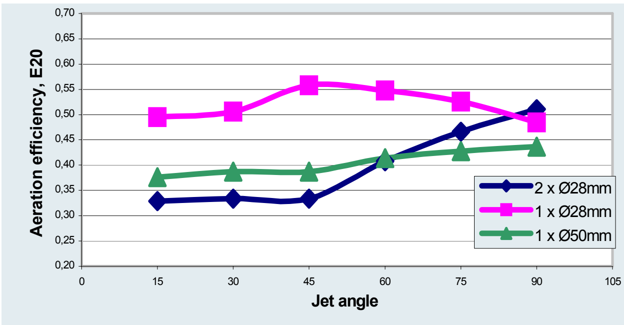
c) 1 x Ø50mm

Figure 4 Variations in aeration efficiency with respect to jet angle and flow rate

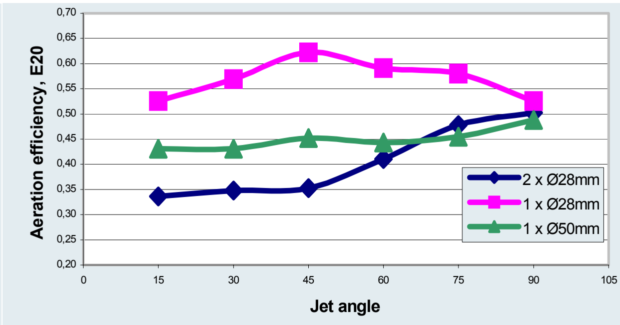
The following sections discuss the E_{20} results referring to measurement point 12, at outlet from biological reactor. The present analysis has shown that flow rate and jet velocity are important factors influencing oxygen transfer rate in biological reactor. The results are presented in graphical view (Figures 4 and 5). Figure 4 shows variations in aeration efficiency with respect to jet angle and different flow rates. Figure 5 also shows efficiency variations, but with respect to outlet (nozzles) properties while the change in flow rate is constant.



a) Q=1,5 l/s



b) Q=2,0 l/s



c) Q=2,5 l/s

Figure 5 Variations in aeration efficiency with respect to jet angle and outlet (nozzle) properties

All of these graphs show proportional relation between jet velocity and aeration efficiency. This observation had been expected since higher velocities cause greater turbulent mixing and deeper jet penetration into the biological reactor.

Thus, air bubbles have longer retention time inside the water column that results with higher aeration rates. On the other hand, jet angle decrement below 30º was observed to breakup the jet. Breaking up into discrete droplets, energy resource (potential) of the jet decreases as well as bubble penetration depth and hence overall aeration efficiency decreases.

Relation between jet angle and aeration efficiency hasn't been strictly determined. It was observed during the investigation that jet angle is indirectly related with outlet properties (number of nozzles and their effective diameter) and resulted jet velocity. Concerning lower jet velocities, jet angle was found to be proportional with aeration efficiency. Greater aeration efficiencies are related to higher velocities, as it is case for the *1xØ28mm*. Jet angle of 45º was found to have the greatest values of oxygen transfer efficiency. Different aeration efficiencies within the given analyzes could be explained by forming different jet shapes. Each scenario is characterized by its unique jet shape, which obviously strongly influences overall efficiency.

Increasing the number of nozzles on account of reduced jet velocities has negative impact on aeration efficiency. Therefore, oxygen transfer rate is reduced within double jets (Figure 2). The greatest efficiency value achieved for the scenarios with double jets was 0,51 with jet angle of 90º and flow rate of 2,0 l/s.

Single jet design with *1xØ28mm* and jet angle of 45º was found to have the greatest values of oxygen transfer efficiency. Maximum efficiency value, achieved during experiments, was E_{20} = 0,62 with flow rate of 2,5 l/s.

In comparison with commercial aerators with their efficiency of 1,2 kgO_2/kWh, plunging water jets have shown to be adequate alternative, especially in hilly areas by using naturally available hydraulic head and thus avoid additional power input. Even if there is need to pump the wastewater to develop desirable head, this type of aeration has shown acceptable efficiency. For instance, observing the same investigation with use of external energy to raise the water head cca 80 cm will result with overall efficiency of 0,78 kgO_2/kWh.

Summary

A series of laboratory experiments were carried out with purpose to measure aeration performance of plunging water jets. From conducted investigation, following conclusions may be drawn:

- Investigation confirms preliminary assumptions about dominant influence of jet velocities on aeration efficiency. Higher jet velocities result with greater efficiency.
- Jet velocities are indirectly dependent of its geometry (shape and diameter). Jet velocity is function of water level in inlet chamber. At steady flow rate, nozzle diameter decrement entails raising the water level in inlet chamber and thus, higher jet velocities.
- Increasing the number of nozzles, at steady flow rate and fixed nozzles diameter, will result with lower jet velocities and thus, oxygen transfer rate decrease.
- Jet angle influence on aeration efficiency hasn't been strictly determined. At higher jet velocities, the greatest efficiency is achieved with jet angle of 45º. At lower jet velocities proportional relation between jet angle and oxygen transfer rate was observed, with greatest aeration efficiency at 90º.
- Jet angle decrement was observed to affect jet breakup with adverse influence on aeration efficiency.
- Single jet design with jet angle of 45º was found to have the greatest values of oxygen transfer efficiency.
- Plunging water jets advantages have also been manifested through better mixing performance inside the biological reactor. The reason for this was found in higher horizontal component of the velocity inside the reactor, which was observed to be favorable circumstance especially for reactors with circular flow.

- Concerning subsequent investigations, a model with wastewater from households is proposed. That is considered to result with higher reliability of the final results on aeration efficiency inside the biological reactor. It is also proposed to carry out an investigation with higher flow rates and jet velocities, as well as to design a model with larger biological reactor and larger number of jets.

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Impact of the Zagreb Wastewater Treatment and Disposal on Sava River

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Davor Malus

Abstract: This paper gives the Sava River water quality analysis of the upper section on its course through Republic of Croatia. Analyzed section covers the area between hydrological stations Jesenice (state border with the Republic of Slovenia) and Galdovo (city of Sisak). Since Zagreb and its nearby settlements wastewaters are not treated, the studied section of the Sava is consequentially heavily loaded with waste (organic) matter. Water quality simulations were undertaken using QUAL2E mathematical model. Detailed analyses of available measurements, based on data from the National Watercourse Quality Monitoring Program, have preceded the simulation (forecasting).

Concerning the input data availability, the given simulation has included the following water quality parameters: BOD-5 and dissolved oxygen concentrations, saturation of water with oxygen and water temperature. Besides the analysis with untreated wastewater effluents (discharges) characterizing current state, the analysis of planned state with treated effluent were undertaken as well. Current state analysis was also used for model calibration. Furthermore, simulations were undertaken for two hydrological regimes of Sava River: the current regime under which Sava maintains (more or less) natural flow condition and the planned regime that assumes the construction of water spillovers (forming river-run lakes). From computed results of autopurification potential of the Sava, the necessity for the planned wastewater treatment plants construction is estimated. The emphasis is on determination of necessity and justifiability of the Zagreb central wastewater treatment plant (CWWTPZ) construction, as well as the impact of the CWWTPZ on the Sava River water quality.

Keywords: water quality, Sava River, CWWTPZ, relevant low flow (Q30,95%)

1. Introduction

Analyzing the Sava River as a significant tributary of the Danube watershed, on its course through Republic of Croatia, the single segments heavily loaded with waste matter are easily distinguished. Special interest is given to the upper Sava course with the urban territory of Zagreb and its nearby settlements as well as Medvednica and Žumberak mountainous streams. The current disposal of untreated wastewaters and runoff has systematically deteriorated Sava River water quality, on the observed location. Natural biological balance of Sava River has been constantly disturbed by significant amounts of organic waste discharge. Concerning the section downstream the discharge point of Zagreb main drainage channel (GOK), special circumstances such as small flows may result with total useless of Sava water for any purpose except for navigation. Consequent upon mentioned a necessity to prevent further deterioration and to protect and upgrade the Sava River water quality for its planned purpose is appearing to become urgent. In accordance with the relevant Water Protection Plans, several wastewater treatment plants have been planned to reduce the total waste loads of the Sava. The scope of this investigation covers the Sava River water quality analyses on the section from the border with the Republic of Slovenia (hydrological station Jesenice) to the City of Sisak (hydrological station Galdovo) in total length of nearly 123,0 km.

The final goal is to forecast (simulate) certain water conditions of the Sava River (from Jesenice to Galdovo) in the near and far future, depending upon the impact of future plants operation, or upon the discharge of wastewaters and runoff. Analyze on the impact of the CWWTPZ on the Sava River water quality will be specially emphasized.

2. Current state condition

The current state of Sava River water quantity and quality, concerning the studied section from Jesenice to Galdovo, has been assessed for low, medium and high flows. It should be pointed out that low flows are relevant for the assessment of watercourse quality and the impact of wastewaters discharge. Namely, wastewater discharge during higher flows has no considerable impact due to significant dilution. There are several methods (norms) to determine the relevant low flows. In Croatia, low flows during 30 consecutive days (any series within a year) with a return period of 20 years and 95% occurrence probability (Q30,95%) are determined as relevant. The relevant low flow will be determined in accordance with elaborates [1][2]. The relevant low flow value at Jesenice Station (beginning point of this analysis) is defined as:

$$Q_{30,95\%} = 68{,}0 \quad m^3 / s$$

Complete analysis comprises the simulations with medium and high flows as well. The values for these flows at Jesenice Station were chosen according to data from elaborate [3]:

$$Q_{mean} = 300{,}0 \ \ m^3/s \ \ ; \ \ Q_{\max} = 800{,}0 \ \ m^3/s$$

Wastewater loads of Sava River have been analysed for "point" and "dispersed" contamination sources. Point sources include all communities with public sewerage system that is used to collect and dispose of the fecal, industrial and runoff wastewater into the Sava River (or its tributaries), regardless whether wastewater treatment plants exist or not. This group of sources includes the mountainous streams that flow directly in Sava, as well.

Dispersed sources include runoff waters collected on agricultural and other surfaces, including the number of cattle on the pertaining area and the number of inhabitants in small communities without the public sewerage system. Regarding the fact that the city of Zagreb has a system of embankments that protect the urban area against flooding, impact of the dispersed sources is insignificant. Runoff that washes out nearby agricultural fields flows into the Sava together with mountainous streams and other tributaries. Small amounts of this runoff still manage to influence the Sava directly as it drains the groundwater. Current quantity and quality of wastewaters and runoff flowing into the Sava have been estimated based on the following elaborates: [1][3][4][5][6][7][8][9][10][11][12][13].

Current state of water quality is given based on data from the National Water Quality Monitoring Program prepared by “Croatian Waters”. Studied section is covered with six hydrological stations on the Sava and three stations on the tributaries Sutla(2) and Krapina(1). Regarding time period of recordings, the 1995-2001 period is relevant. A longer period of time is not relevant due to the Homeland War and changes in water quality that took place as a consequence of war operations. Regarding the use of measurement results, the positioning of sampling stations, as well as sampling times, is not favourable for the preparation of this type of study. Namely, the sampling time is not synchronized with the hydraulic regime of the watercourse, but is determined in advance for regular time periods. Therefore, one particular mass of water passing the individual stations could not be monitored from which organic matter decay rate and watercourse reaeration could be determined. This was considered as a very sensitive part of the analysis and compensation was made with the application of averaged reference data [15][16][17][18][19].

Analysing positions of hydrological stations it is clear that there are no stations downstream from the mouth of the Main Drainage Channel of Zagreb, where the impact of discharged Zagreb wastewaters could be better monitored. The closest station downstream of GOK is situated some 22 km from the discharge point.

3. Evaluation of planned state condition

The year 2015 has been taken as the planned year. In this period it’s been assumed that second-degree wastewater treatment plants would be constructed in all cities on the subject section. In comparison to the current state, the outlet of the Zaprešić plant will be directly in the Sava River. The discharge of the Samobor plant will be into the Gradna Channel, connecting Gradna stream and Rakovica stream, with the discharge point into the Sava common with the Rakovica stream mouth. In Zagreb, wastewaters from the right bank will be connected to the central wastewater plant (CWWTPZ) on the left bank and discharged by GOK into Sava River. The outlet of the Velika Gorica plant has already been constructed at its final position. During the rainy periods with heavy rains, high flows will be relieved from the Zagreb drainage network via Črnomerec and Savica spillovers, as well as number of spillovers connected to GOK.

It is further assumed that upstream of Jesenice Station, wastewater treatment plants of at least second degree treatment would also be in operation, so that the Sava water quality at the Jesenice Station would meet the second degree requirements. Data on point and dispersed sources have been presented according to the same sources as it was done for the current state analyses. Besides the planned steady-state analysis with the current hydrological regime, the water quality state in the case of multi-purpose water spillovers construction is simulated as well. Under such circumstances, the hydrological regime of the Sava would change.

4. Simulations of the Sava River water quality

Sava River water quality simulations were undertaken using QUAL2E mathematical model. The key water quality parameters (BOD-5 concentration, dissolved oxygen balance and the water temperature) have been modelled. Concerning the changed hydrological regime, total phosphorus has also been modelled as a good indicator of the future trophic state. Regarding this case, it can be stated that besides hydrological and hydraulic data on watercourses and some of the data on measured concentrations of key water quality parameters, there are no customized investigations and measurements applicable in environmental modelling.

By establishing a mathematical model, which is based on series of input data that have been taken as averaged values from references, the probability of mistake is increasing. The model was calibrated by checking it for the measured state of the Sava River water quality, carried out according to the Sava water quality-monitoring program [7]. After establishing that the model appropriately describes current state of quality, as recorded at observing hydrological stations, the state for planned loadings of the Sava River was simulated.

Simulation was undertaken for three different state of Sava water flows – low, medium and high flows. Regarding the meteorological conditions in Zagreb, each of the mentioned water flows was analysed during dry and rainy periods in Zagreb. It should be pointed out that during the rainy period this analysis assumes there will be rain in the Zagreb district, and on the other parts of the watershed dry weather would remain. However, there are no data on the probability of occurence of extended heavy rain in Zagreb and low flow of the Sava. Based on past experience, such an occurence has low probability. The hydraulic state characteristic for summer months (May-September) has been modelled. Clearly, such typically summer conditions concerning the environmental aspect would be most unfavourable. Simulation of the Sava River water quality was undertaken for:

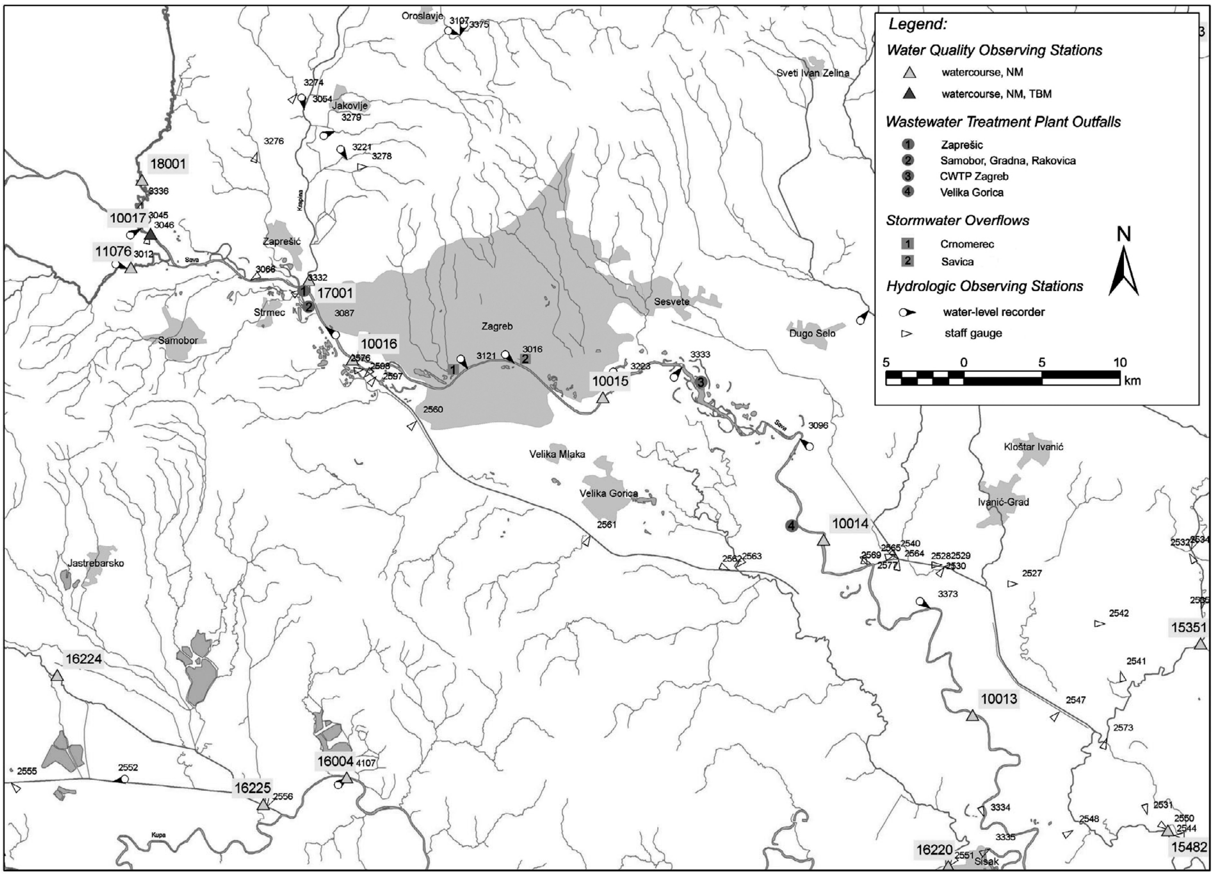

Figure 1 Sava River – studied section

- Current state of loading and current hydrological regime;
- Planned state of loading and current hydrological regime;
- Planned state of loading and changed hydrological regime.

4.1. Current state of loading and current hydrological regime

Regarding the current state analysis, the year 1999 has been taken as the planned year. Simulation results has shown that with the current disposal of untreated wastewaters, the Sava water quality upstream from the GOK mouth can be classified as Category III (according to BOD-5), and occasionally bounding between Category II and III during dry periods with low Sava flows. The Sava water quality would significantly deteriorate after the discharge of untreated wastewaters of Zagreb, and would be classified as Category IV water (according to BOD-5) somewhere to Martinska Ves (cca 50,0 km from the GOK mouth). The status with dissolved oxygen is more favourable, and accordingly the Sava along the entire section can be classified as Category II.

During medium and high flows the Sava meets the present provisions on water categorization as given in the National Water Protection Plan, due to significant dilution.

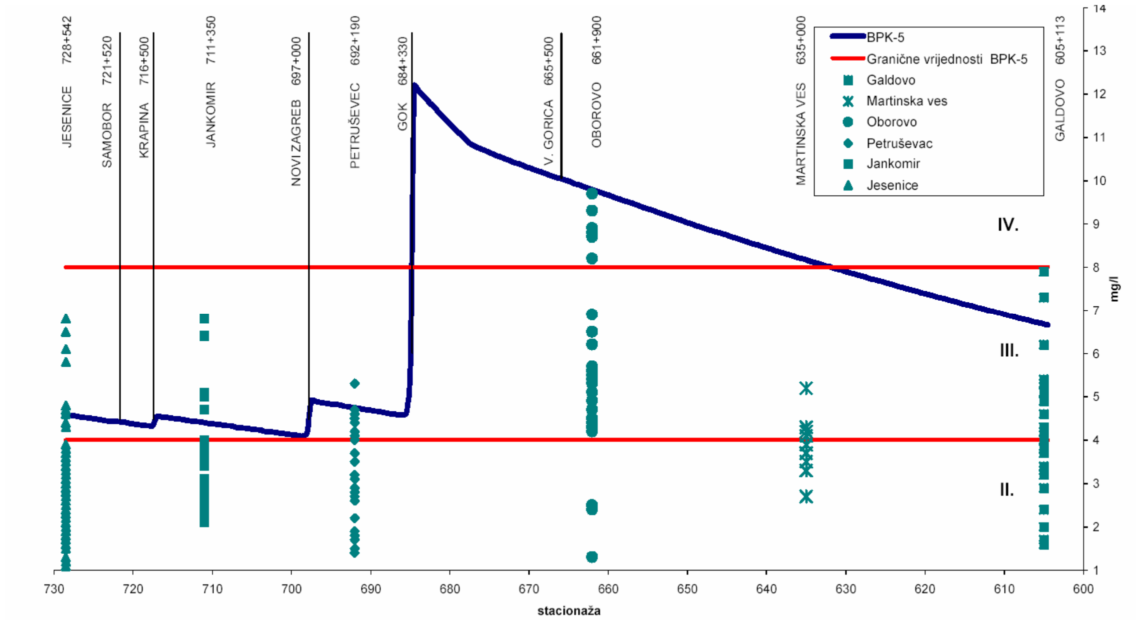

Figure 2 Simulation results for BOD-5 – current state of loading with low Sava flows during dry period

In the case of rain in Zagreb and low flows of the Sava, the water quality status is even less favorable. The whole section downstream the GOK mouth all up to Galdovo would be classified (according to the BOD-5) as Category IV. Consequentially, Sava River would not meet water categorization requirements. Along the entire section, oxygen meets the requirements of Category II, except directly after the GOK discharge point, where it drops to Category III.

During medium flows of the Sava and rain in Zagreb the subject section of the Sava from the GOK discharge point to Galdovo is classified as Category III according to BOD-5. On other parts the Sava is classified as Category II. According to the dissolved oxygen indicator, as well as oxygen saturation, the entire section of the Sava is classified as Category I. During high flows of the Sava and rain in Zagreb, the Sava belongs to Category II, along the whole section.

At the same time, current state analyses were used for model calibration. From the computed simulations it is obvious that the results are residing in the range of measured values from the National Watercourse Quality Monitoring Program.

4.2. Planned state of loading and current hydrological regime

The state of Sava River water quality for the planned waste loadings (year 2015) is forecasted under the assumption that all wastewaters on the monitored section prior to discharge into the Sava are previously secondary treated. Under such circumstances, Category II water characteristics could be maintained along the entire monitored section, with exception of the very short section downstream the GOK mouth, where the Sava during low flows would be classified as Category III, according to the BOD-5 parameter (Figuress 3 and 4).

During medium and high flows, the Sava would meet Category II requirements on the entire section.

In the case of rain in Zagreb and low Sava flows, the Sava would have Category III water quality up to the GOK mouth and Category IV water quality from GOK to Galdovo. On the results basis, the Sava River would not meet water categorization requirements from the National Water Protection Plan according to the BOD-5 indicator. During medium flows and rain in Zagreb, the water quality of Sava River would be Category III, from the GOK mouth to Galdovo. Only during high water flows and rain in Zagreb, Sava River would meet Category II requirements along the entire monitored section.

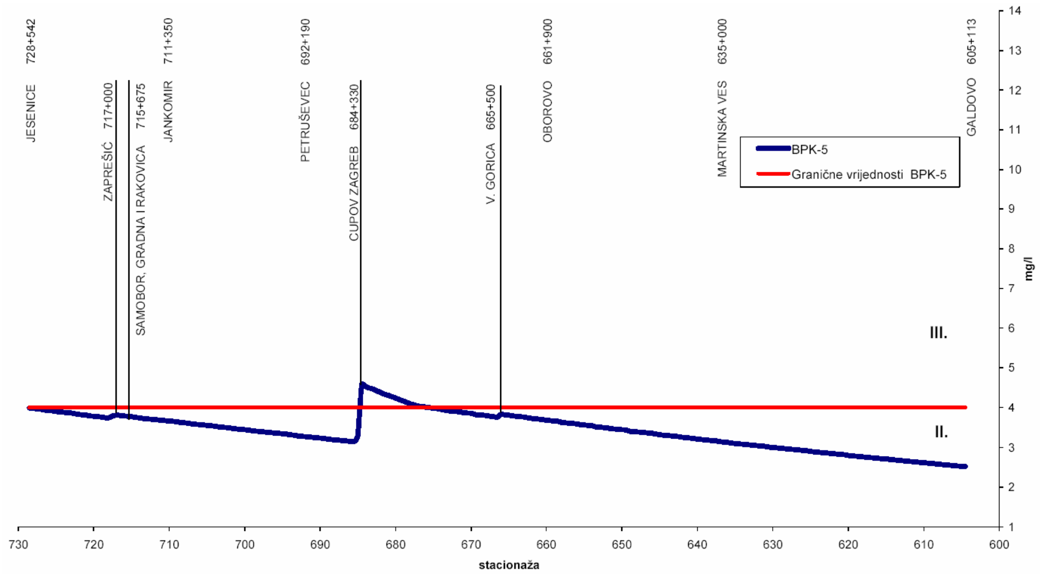

Figure 3 Simulation results for BOD-5 – planned state of loading with low Sava flows during dry period

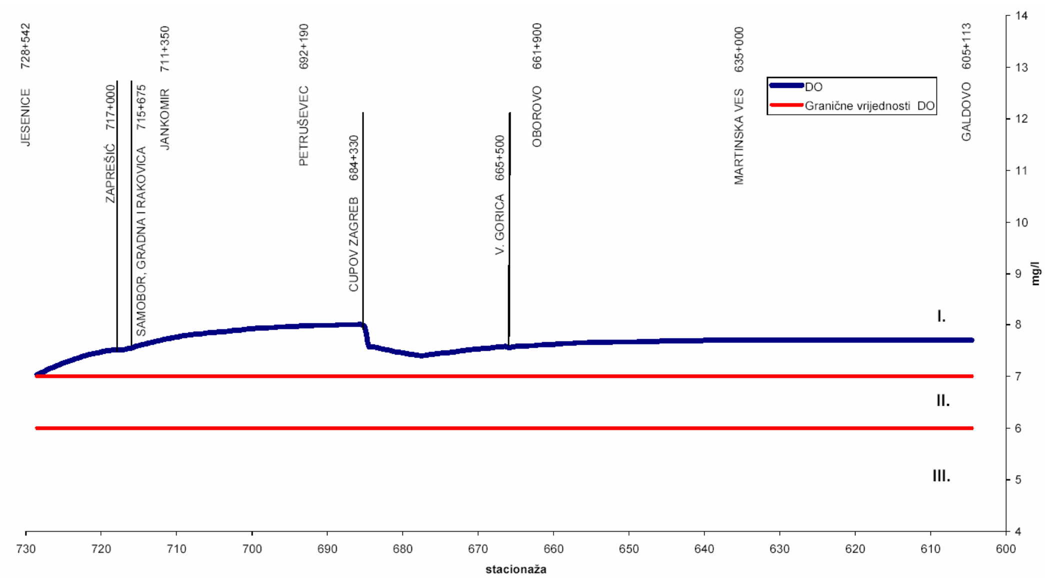

Figure 4 Simulation results for dissolved oxygen – planned state of loading with low Sava flows during dry period

4.3 Planned state of loading and changed hydrological regime

The construction of water spillovers would change the hydrological regime in the watercourse on the monitored section of Sava River. Under such circumstances in the Sava, the water quality was forecast for the future wastewater loading with wastewaters previously treated to the second-degree treatment. During the dry period and low flows of Sava River, the critical indicators are total phosphorus and dissolved oxygen. Regarding total phosphorus, Sava River would be classified as Category IV downstream the GOK mouth. According to dissolved oxygen, Sava is at the same time classified as Category III. Under medium flows, according to total phosphorus, Sava River would be classified as Category III downstream the GOK mouth. For other indicators, Sava River would meet Category II, and Category I requirements.

In the case of rain in Zagreb and low Sava flows, categorization requirements would not been met, downstream the GOK mouth.

Under medium flows, Sava River would be classified as Category III downstream the GOK mouth all the way to Galdovo. Therefore, it would not meet the Sava categorization requirements even at medium flows.

Additionally, the implementation of third-degree treatment on wastewater treatment plants prior to discharge into the Sava has been analysed. In that case, all indicators except dissolved oxygen meet the categorization requirements along the entire section. According to this indicator, Sava should be classified as Category III.

Summary

Based on the undertaken conducted analyses it can be concluded that:

- The current discharge of untreated wastewaters on the section from Jesenice to Galdovo does not meet the water categorization requirements according to the National Water Protection Plan – at the relevant flow ($Q_{30,95\%}$).
- The proposed mathematical model for the simulation of planned water quality conditions in the Sava is appropriately chosen, which is proved by the “simulation” of the existing state, set by the Sava River Quality Monitoring Program.
- The construction of the Zagreb plant (CWWTPZ) would significantly upgrade the water quality of Sava River on the section from Zagreb to Sisak at relevant flows.
- The water quality upstream of Zagreb improves by treating Samobor and Zaprešić wastewater, and, at the same time, contributes to the upgrading of the water quality downstream of Zagreb.
- In the case of rain in Zagreb and simultaneous low flows (relevant) of Sava River, under heavy rain over an extended period of time, the quality of water would significantly drop with regard to loading according to BOD-5. However, even under such circumstances, the value of dissolved oxygen on the whole section would not be lower than 6 mg O_2/l, and there would be no reason to proclaim the “state of

extraordinary contamination" and call for appropriate protection measures. It should be pointed out that such a state of increased loading in the watercourse could last up to one day or less, so unfavourable impact on the living communities in Sava River would not be expected.

- The change in the hydrological regime of Sava River, after the construction of water spillovers on the monitored section, the secondary wastewater treatment plant in Zagreb will not fulfill the water categorization requirements on the subject section.
- The construction of a tertiary treatment plant and water spillovers in the Zagreb area would ensure that most of the Sava River categorization requirements are met. However, it should be pointed out that in this case a reduced quantity of dissolved oxygen in the Sava water can be expected along the entire section from Podsused HEP to Galdovo. This is the consequence of the changed reaeration conditions and dissolved oxygen consumption, in the river. It is stressed that the simulation of water quality conditions after the construction of water spillovers could not be tested on the actual state, as such circumstances on the Sava are not yet existing. It will be necessary to undertake additional investigations after the construction of the first water spillover in the Zagreb area (probably Podsused HEP), in order to confirm the above statements.

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Protecting a Water Course from Gneiss-Quarry Caused Pollution

Mladen Zelenika, Jerko Nuić, Božo Soldo

Abstract: Very fine gneiss debris released from a wet beneficiation plant of two gneiss quarries Mikleuska enters the stream Kamenjaca at a rate of 70 tons/day, together with 700 m^3/day of water. The stream flowers from northern, mostly forest covered, catchment area of approximately 15 km^2 and passes between the two quarries. The Croatian regulation prescribe the classification of waters into five groups according to their quality. Water in this permanent stream was categorized in the State registry of waters as a first-class water. Water quality variables used in the classification includes pH value, oxygen, nitrogen, phosphorus, hygienic indicator bacteria, radioactivity and toxic substance, bat without the water colour, and turbidity. Criteria generated were based on the needs set by relevant kinds of water used and its general environmental condition. In this paper exploration of possibilities for technically, environmentally and economically sustainable exploitation of quarries that would ensure protection of water quality in the stream Kamenjaca is described.
Keywords: gneiss, quarry, suspended solids, sedimentation sump, beneficiation plant.

1. Introduction

Gneiss in adequate sizes of fraction is an important stone-ware for construction of many foundations of major engineering structures or as engineering material. Mikleuska gneiss-quarry is the most important source of this kind stone in the northern Croatia. It is located on southern slopes of Moslavacka gora, 19 km away from Kutina, in the village Mikleuska on the Podgaric-Kutina road. The stream Kamenjaca which passes in its immediate vicinity belongs to the drainage basin of the Lonjsko polje. It is essential to prevent its degradation due to the influx of contaminants from the quarry.

Excavation of gneiss in the quarry does not present a threat to the quality of water in the water course. Since gneiss has a similar mineralogical composition as granite (quartz, feldspar and mica) and its components are mostly insoluble in water, dissolved pollutants have not been detected in the water even after its usage in the wet beneficiation plant nor in the Kamenjaca stream that passes largely through gneiss rocks. However, unacceptably high concentrations of suspended load, close to 133 g/l were measured in the water that leaves wet beneficiation plant (see Figure 1). The suspended load in water from the beneficiation plant used to be discharged into the stream for 8 to 10 hours every working day.

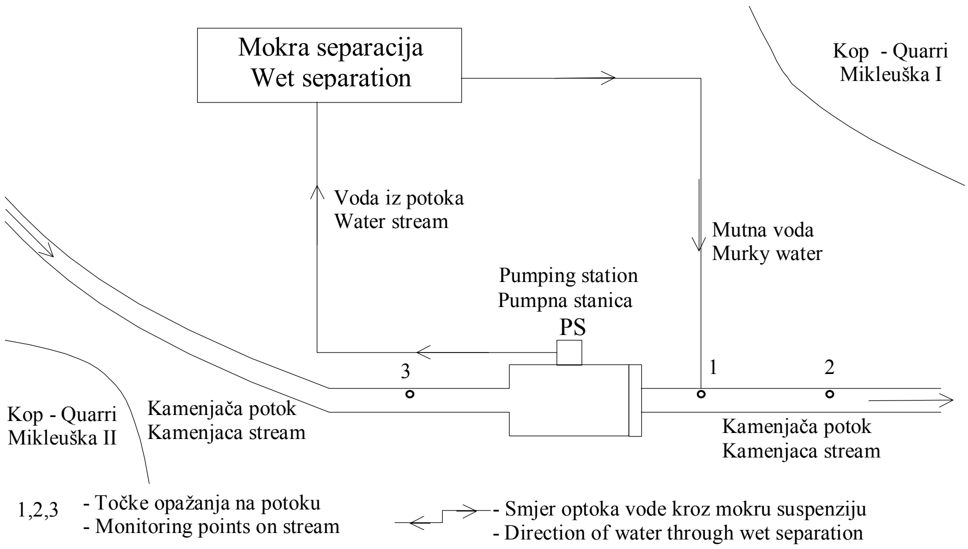

Figure 1 Present solution of water circulation

Due to high production cost and low market prices of the classified gneiss, a more complex and expensive method for purification can not be used. Therefore here is presented an environmentally as well as economically acceptable way for prevention of Kamenjaca water course. The construction of two rectangular sedimentation sumps and a lagoon using gravitational forces is proposed (see Figure 3).

Kamenjaca is a relatively small permanent surface stream with a flow rate between 24 and 47 l/s. Beside on the water quality it was categorized in the State registry as a ″first class water″. Extensive hydrogeological investigation have been performed in the downstream Gracenica and in Lonjsko polje that receives water from the stream Kamenjaca. Kamenjaca is a mountain watercourse being turbid during periods of intense precipitation and snow melting.

2. General data about the quarry

- Intrusion of Moslavina granites in the older metamorphic complex has caused its metasomatical alteration. In this manner few hundred meters to one kilometre wide belt of heterogeneous and homogenous migmatites was formed. Gneiss that has a similar mineralogical composition as granite consists of approximately 45 % guartz, 40 to 45 % feldspars and 10 to 15 % mica. Some other silicates are also present in amounts lower than 1 percent. These compounds are mostly insoluble and do not present a threat to the quality of the water in stream (Braun, 1988).

- There are two quarries producing in the last 30 years approximately 100000 m^3/year of gneiss stone classed in six marketable stone aggregate sizes. One quarry, Mikleuska I is on the left side and the other quarry, Mikleuska II is on the right of the watercourse. Excavation is oriented towards the gneiss massive and planned production level is between 90000 and 120000 m^3 of the stone per year during the next 15 years.
- Owners of the quarry have constructed a 2 meter high **concrete dam** in stream channel at elevation of 155 m near the plateau of the Mikleuska I quarry. Wet beneficiation plant and other quarry structures were constructed near the dam to ensure continuous supply of technical water. Small natural spring at the elevation of 154 m has been captured to ensure availability of drinking water for employees of the quarry.

3. General information regarding pollution

Dissolved harmful compounds have not been detected in samples of rainfall water (that falls flows) and/or accumulates on the open excavation surfaces, nor in the water from the stream that was used in the wet beneficiation plant (Braun, 1988). To confirm this statement water samples were taken during the year 1998 and in January of 1999 from 3 various places in the watercourse, upstream and downstream from the quarry (see observation points 1, 2 and 3 in Figure 1).

- No increase in amount of suspended particles in water of the stream of Kamenjaca was noted when it was compared to the amount of suspended particles in the rainfall waters flowing over plowed land and forest roads. However, precipitated water that flows over surfaces in the quarry should be systematically collected in properly designed retention basins and its quality should be controlled before discharge into the stream.
- Large amounts of suspended stone particles are present in water that occasionally used for washing of the equipment and regularly for washing of finer stone fractions the process of wet beneficiation. Wet beneficiation plant uses 700 m^3 of water from the dammed water course every working day continuously for 8 to 10 hours. Approximately same volume contaminated with small gneiss particles is returned to the stream a few meters downstream from the weir (see Figure 1).
- Environmental impact study for the quarry Mikleuska (Nuic, Zelenika et al., 1999) recommended excavation of useful substance at horizons with elevations between

162 and 155 m in the Mikleuska I quarry and between 175 to 160 m in Mikleuska II quarry to endure efficient reclamation and more efficient use the present and final horizons of activity in the quarry. Excavation of gneiss at elevations lower than present working plateau in Mikleuska I would create an area for the sedimentation sumps and on lowest part of the quarry at elevations between 151.2 to 153.5 m as a lagoon for extended sedimentation of the smallest particles would be created. Sedimentation sumps would be transformed into recreational swimming pools and lagoon into the pond for fish farm.

4. Results of additional investigation

The unit for laboratory examination of Petrokemija d.d. in Kutina performed analysis of more than 30 physical, chemical and biological variables (Dzajo, 1999) in water samples taken from each observation point. Samples of the water that were taken were turbid and did not comply with the present state regulations for quality of a first class water due to the suspended load.

Data that were collected through the examinations of turbid water samples related to sedimentation of suspended solid particles at the laboratory of Geotechnical faculty in Varazdin (Levacic, Stuhec, 1999) are presented in the table 1. These data demonstrate that clearing of water with suspended gneiss particles can be achieved if the sample is immobile or in the conditions of laminar flow.

It was noted that clearing was fastest in a sample taken from the outlet of the wet beneficiation plant was transported in a shorter distance. Clearing was somewhat slower in the fresh sample taken 50 m downstream from the outlet and the slowest in the sample taken from the outlet that was transported for the longest time. Noted differences are probably caused by additional breakdown of bigger gneiss grains during transport in the container and stream. Large amount of serictised feldspars in the rock facilitates this crushing process. For this reason future location of sedimentation sumps should be as close to the wet beneficiation plant as possible.

Table 1 Speed of settling of solid particles suspended in 1000 ml of water used in the wet beneficiation process

Date of the sampling Components in the decanter		Volume of turbid and clear water ml			
		1 hour	3 hour	24 hour	48 hour
Outle Dec. 1998	Coarse residue	100	120	120	120
	Total residue	915	640	500	440
	Clear water	85	360	500	510
Outle Jan. 1998	Coarse residue	80	80	80	80
	Total residue	870	680	430	380
	Clear water	130	390	570	620
50 m down-stream Jan. 1999.	Coarse residue	15	15	15	15
	Total residue	15	150	85	64,5
	Clear water	985	850	915	865

Data in table 1 indicate that a design of the sedimentation sump with appropriate dimensions for removal of suspended solids in justified. In the adequately designed, constructed and maintained sedimentation sumps it is possible to reduce amount of the suspended solids to the required concentration, as measured in samples of water that were taken upstream from the quarry.

Diagrams of granulometric composition of solids suspended in water samples that were taken on observation points 1 and 2 (see Figure 1) are shown in the Figure 2.

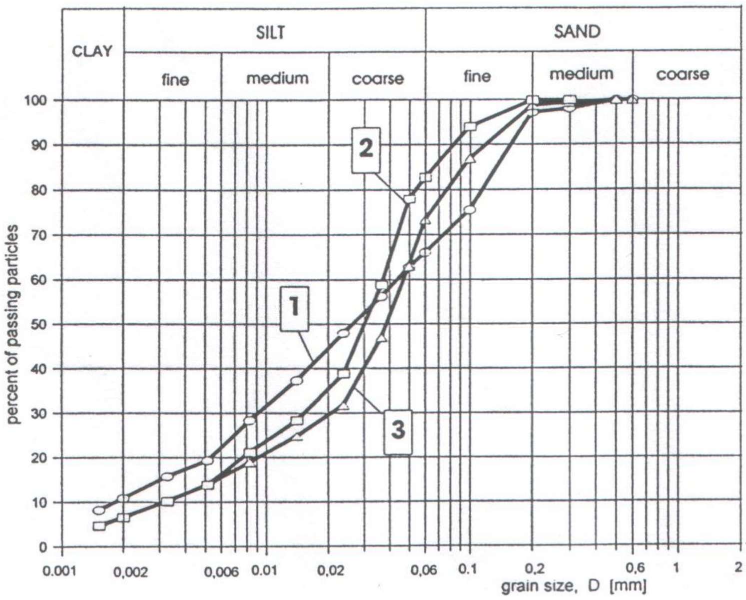

Figure 2 Diagrams of the granulometric composition of suspended solids
1 – Sample of water taken in December 1998. at observation point 1;
2 – Sample of mud water taken in Janury 1999. at obervation point 1;
3 – Sample of water taken at obervation point 2, (see Figure 1)

- As shown in the diagram 1 of Figure 2 granulometric composition of particles in the sample was sand 33,26 %, silt 55,80 % and clay 10,95 %. These values give a total of 133 grams (133 kg/m^3) of dry matter per litre of suspension or 85 litre of residue per cubic meter of suspension.
- The effect of natural dissolution of the turbid water used in wet beneficiation plant can not prevent water quality in the small water stream. Turbid water used in wet beneficiation plant and for washing of the equipment should be treated in properly designed sedimentation sumps and gradually discharged in the water course. If finest particles of colloidal dimensions san not be removed, a special lagoon (pond) for deposition of the effluent that are discharged from sedimentation sumps should be designed. At the times when high concentration of colloidal particles are detected in the water is released from the lagoon the stream it should be diverted back into the wet beneficiation plant (see Figure 3). Such of lagoon water is especially convenient during periods of the low water discharge in the stream.

5. Sedimentation sumps

Release of contaminated water from the wet beneficiation plant to the stream Kamenjaca is the main threat to the water quality. The owner of the quarry is obliged to remove suspended solids from water used in production process. Market prices of quarry products are relatively low, and investments in the equipment for removal of solids are very high (desander, desilter, centrifuges, devices that use flocullators for acceleration of settling etc). Since there is available enough space for construction of an adequate sedimentation sump within the quarry, it has to considered as a possible solution for the pollution problem.

The level of the contaminated water in the wet beneficiation plant is higher than 155 m, and lowest horizon of the exploitation area downstream along the banks of the Kamenjaca has an elevation between 150.2 m and 152.6 m. The planned sedimentation sump should be situated between the elevation point 155.5 m near the wet beneficiation plant and the lowest part of the terrain with the elevations from 151,2 m to 152,5 m. The difference in height of 3.5 m (155.5 m – 152.0 m = 3,5 m) leaves enough space for a settlement zone and ensures gravitational flow of the water through the settlement zone to the lake (lagoon).

Besides size of suspended particles, sedimentation speed is also strongly influenced by the difference in the density between the fluid and suspended particles versus viscosity of the fluid (Linsley et al. 1992), as indicated in the Stokes formula for laminar flow of the fluid with suspended particles.

$$v_t = \frac{g \cdot (\rho_c - \rho) \cdot d_c^2}{18\mu} \quad (1)$$

where: v_t = sedimentation speed of the given particle, m/s; g = gravity acceleration, m/s^2; ρ_c = density of the suspended particles, kg/m^3; ρ =density of the fluid, kg/m^3; d_c = particle diameter, m; μ = dynamic viscosity of the suspension, Pas.

For a given inflow of water from the wet beneficiation plant $Q = 0.20\ m^3/s$, sedimentation sump height h_o, and given time of retention in the sedimentation sump $t_o = 10.80\ s$, it is possible to calculate area of the sump A, volume of the sump V, and flow velocity of particles passing through the sump v_o in accordance with following equations:

$v_o = h_o/t_o$ (2), $t_o = V/Q$ (3), $A = V/h_o$ (4), $v_o = Q/A$ (5)

Based on the above given parameters, and size of the rectangular horizontal sedimentation sump, values of other parameters can be calculated from equation given above.

$V = Q \cdot t_o = 0,020 \cdot 10800 = 216\ m^3$ (6) $A = V/h_o = 216/0,5 = 432\ m^3$ (7)

Mikleuska quarry will be operating for relatively short time, and future use of sumps after ending of the stone exploitation is a very important issue. According to the Sport encyclopedia (1977) typical standard sizes of swimming pool for a sport and recreational use are 25×18×2.2 m or 33.3320×2,2 or 50×25×2.2 m. Excavation of the sedimentation sump is technically and financially advantageous solution for quarry owner. It doesn´t require large investments and it can be performed in crude gneiss with the existing equipment. Sizes of sedimentation sumps are recommended similar to size of standard swimming pool to reduce future expenses associated with the change of the function.

The terrain owned by the quarry has a favourable configuration and size. It extends between the Kamenjaca stream and the wet beneficiation plant. Average content of suspended particles in water, desired frequency of residue removal and standard size of sport swimming pools had a decisive influence on size of the sedimentation sump.

Available space on the useful substance in quarry is large enough for a future sport swimming pool with standard size of 50×25×2.2 m and two smaller swimming pools with size of 25×18×2,2 m. The space on alluvial deposits will be used for a lagoon which can be later used for fish production. If two needed sedimentation sumps are placed in bigger swimming pool with an approximate volume of 2×(55×13×3.5) m can be rationally excavated. It would be divided into two parts with a 4 m wide barrier of the crude gneiss rock. Slotted pipes would be positioned on the bottom of each sump and covered with gravel of appropriate size and geotextile to ensure proper drainage, especially through the coarse deposits in the first ten meters long part of the sump. Verification of the length and width of sedimentation sump is, as follows:

$A = l_o \cdot V$ (8) and length $l_o = A/w = 432/13 = 33{,}3\ m$ or 50 m (9)

$$v_p = Q_1 / A_1 = 0{,}0083/13 \cdot 0{,}20 = 0{,}00319\ m/s \tag{10}$$

Following volume of residue can be deposited in such sedimentation sump:

$$V = l_o \cdot w \cdot h_t = 55 \cdot 13 \cdot 1{,}8 = 1237{,}5\ m^3 \tag{11}$$

of residue.

Residue height is: $$h_t = H - h_c - h_o = 3{,}5 - 0{,}2 - 0{,}2 = 3{,}10\ m \tag{12}$$

where: H = total depth of the excavation, m; h_c = reduction of sump depth due to slotted pipes and gravel, m; h_o = height of the weir into the sump, and could be regulated as needed, m; w = width of the sump; A = surface; l_o = length of each sump.

Laminar character of flow in the sedimentation sump with given size and velocity of the flow must be verified, since efficient sedimentation of suspended particles is possible only under laminar conditions. Reynolds equation of the laminar flow was used and value of Reynolds number was calculated, and compared with the value of 580, which is a critical value and indicated transition from laminar to the turbulent flow (Agroskin, 1969).

$$R_{eR} = \frac{v \cdot R}{\mu/\rho} = 344 < 580 = R_{eR\ critical} \tag{13}$$

where: v_p = average speed in the sump = 0,00319 m/s; R = hydraulic radius, R=A/O=0,2 m; μ = viscosity of water with suspended particles =0,002 Pas; ρ =density of the water suspended particles =1080 kg/m^3

Calculation above has confirmed laminar character of the flow since value of the Reynolds number was only 344. For this reason one sport swimming pool with standard size of 50×25×2.2 m with two sedimentation sump and two sport pools with size 25×18×2.2 m for the drying of the residua are proposed (see Figure 3).

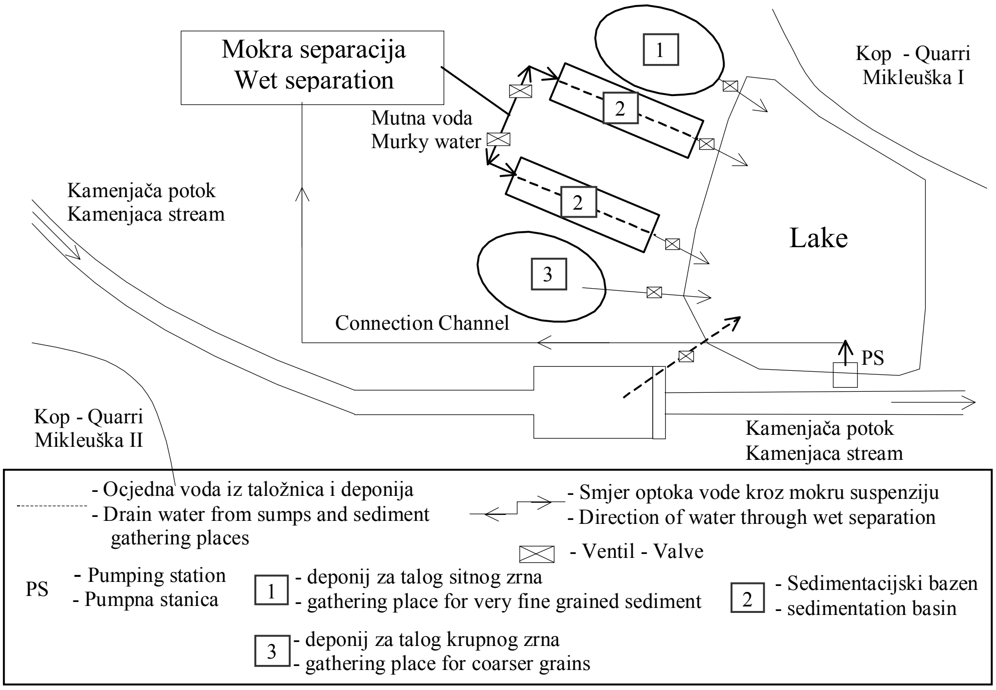

Figure 3 Proposed solution of water circulation

The small lake (lagoon) with an area of approximately 10000 m^2 positioned downstream from the sedimentation sump is proposed as an additional measure for protection of the Kamenjaca stream. The humus from the location of the future lagoon will be removed and used in the reclamation of the abandoned areas of the quarry. Absence of humus layer from the bottom of the lagoon may increase infiltration of the water into the alluvial aquifer and raise level of groundwater in wells located downstream from the quarry and in the village Mikleuška. Rise of the water table is an additional favourable effect of the proposed lagoon.

If quality of water on the outlet from the lagoon to the stream still raises some concerns among the ecologists, reuse of lagoon water into the wet beneficiation plant is recommended (see Figure 3).

Figure 3 shows locations of the sedimentation sump, the lagoon and the residua disposal area. Due to the expected segregation of particles in the sedimentation sump two depots for sediments are suggested. Sedimentation of the coarser grains that could be sold as a building material is expected on the first 10 to 15 meters of the sedimentation sump while accumulation of finer particles is expected in its remaining part. For this reason two disposal areas are planned for sun drying of the coarser grained and fine grained sediments. Coarse grained material will be sold as technical stone on the marker and fine grained fraction will be used to from a laminar water flow in the lagoon, filling of the holes in the quarry and for levelling of appropriate finalization planes in the quarry areas prior to their enrichment with humus and planting of plants.

Before its distribution to consumers, coarse-grained fraction will be additionally dried on a particularly designed disposal area and released water will be taken back the lagoon. Residue consisting of the smallest particles (mud) will be dried in appropriate pools. Installation of drainage devices in the pools, similar to the one in active sump is recommended.

Water in the lagoon will be directed towards the stream along an extended channel, which will be gradually formed from sediments of finest particles. Size of the channel should ensure laminar flow and efficient settling of the finest colloidal particles. After cessation of activity in the quarry, as was suggested in the study by Nuic, Zelenika, et. Al. (1999), sedimentation sump should be transformed into swimming pools of standard size and lagoon into a fish farm creating an economically favourable effect.

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The Reliability Assessment of Water Treatment Subsystem of Wrocław City

Izabela Zimoch

Absract: The proper Water Treatment Plant (WTS) operation influences on reliable operation of the whole Water Supply System (WSS). The treatment processes of drinking and communal water take place in a technological consisted of the parallel-connected facilities of the same type. Such a technological series structure guarantees that in case of shutting down of a device, the system would proper operate. During emergency periods water may be directed to the rest of operating facilities, without any risk of WTP operation failure. Such operations are connected with the changes of technological parameters of unit process during water treatment or with the decrease of the rate of treated water flow. The above mentioned cases are not situation of full subsystem failure. However, significant decrease of water quantity may influence on WTP operation.

More important water supply systems in the world are designed and evaluated, just after the usage of the above mentioned theory achievements. Reliability theory is very important while projecting evaluation of WTP supply systems. It prevents from elements failure caused by eventual mining damage. It may decline the risk of supplying the noxious water.

In the paper the practical usage of probabilistic evaluation model of water treatment plant technical operation on the example of Water Treatment Plant "Mokry Dwor" and "Na Grobli", which are water supply systems for Wrocław city. The evaluation based on the real exploitation data in the years 2001-2004 and published data, includes the determination of reliability index estimators of water treatment facilities and the stationary availability index of water intake subsystem of Wrocław city.

Keywords: Water Supply System, Water Treatment Plant, reliability theory,

1. Introduction

The reliability of Water Treatment Plants (WTP) is a feature based on water production (in given service conditions) of proper quality and quantity dependently on buyers needs. The variation of water quality is random function describing set of chemical, physical and bacteriologic features of raw and treated water. The abovementioned variation influences on such features of water production subsystem (WPS), (Zimoch, Wieczysty, 2001):

- complicated engineering-technological structure,
- random character of factors influencing subsystem operation,
- the lack of distinct limit between efficiency and inefficiency or partial efficiency,
- occurrence of various types of failures such as: full, not full, sudden, gradual, dependent, independent, detectable, undetectable and difficult to detect etc.,
- the progress in automation and control of subsystem operation.

The paper was written within the framework of realization of research project KBN 5T07E 044 25 under the title “Determination of reliability operation model of water supply system (WSS) in the aspect of secondary water contamination in a water pipe network”.

2. Water Supply System in Wrocław

Wroclaw, the capital of Lower Silesia is the city situated on the Odra River in the southern-west of Poland. The population of Wroclaw is about 700 thousands of inhabitants. It is on the biggest and most important cultural and scientific centers of Poland. There are a lot of industrial branches (electro machinery, chemical, typography etc.). Such strongly developed place requires large amount of water to meet needs of its inhabitants and industry. The beginnings of Wroclaw 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 appears about next water taps i.e. buildings used to take water: The Grate Scooper next to Dolne Młyny, the Maciej Scooper, the Cat Scooper and the House 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 city 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 city 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 water supply system of Wroclaw consist of two subsystems- water supply subsystem (WSS) and water distribution subsystem (WDS). The water supply systems in Wroclaw take water from the Oława River, which is supplied by switching water system from the Nysa Kłodzka River. The distribution water subsystem is connected with two big water treatment plant WTP: “Na Grobli” and “Mokry Dwor”, which are separate technological lines. The „Na Grobli” plant treats infiltrated water, the „Mokry Dwor” plant – surface water, and the „Leśnica” plant – underground water. The primary assignment realized by water treatment plants is to maintain continuity and reliability of supplying water of standardized quality and required pressure. Plants continuously supply water to meet city demand. So far, during exploitation of Wroclaw water supply system occur two failure situations. One – in July 1997 during the flood. Then technological devices had been flooded and plants stopped supplying water. And the second – during manganese catastrophe in 1906, when quality of water rapidly made worse, because of abundance of manganese compounds (caused by content of iron).

Water treatment plants have total production capacity at the level of 300 000 m^3/d, what is a double value of water consumption. Pumping clean water by plants and urban zonal pumping station ensure essential pressure in water-pipe network.

The plants are successively modernized. Priority of investments is improving water quality. During realization much is made with regard to conditions of the environment protection. There are chosen technologies that minimize number of produced sediments and interference in environment. In 2002 it was initiated two-stage process of water treatment. It was started irrigating water-bearing area with water cleaned in the “Mokry Dwor” plant, what will let make intensification of natural way of cleaning water in wet ground areas. Ecosystem of water-bearing areas is place of rigorous protection.

In treatment plants all devices and technological process system are built with at least 100% of reserve. That ensures holding constant supply during emergency situations. Reliability of water supply to the city may be threatened only in case of unusual events. However, possessing two big treatment plants even in those cases decreases effects of potential threat.

Total length of Wroclaw water-pipe network is 1,800 km. This network has a ring system. It consist of about 200 km of main network, 1,200 km of distribution network and about 30 thousands of terminals of length 390 km. Distribution subsystem has ring system. Owing to this there exists possibility to deliver water to specified place with different “waterways”, what matter while cut-off fragments of the network caused by failure or modernization.

3. The “Mokry Dwor” Water Treatment Plant

The “Mokry Dwor” WTP has started to operate on 15 September 1982. The Plant was designed for the production of 200 000m^3, at present the amount of water is 70,000m^3 a day. Water is supplied from the Oława River in Czechnica by two pipes ∅ 1200. Nowadays the plant has two functions: treats water to irrigate water bearing area (not full technological process) and completes water production for the city (full technological process).Water intake from water bearing area supplies the second water treatment plant WTP “Na Grobli”

Water in the Oława River is contaminated both mechanically (turbidity, suspension), chemically (iron, manganese and ammonium compounds) and biologically (bacteria, diatoms, plankton, blue-green algae) and is characterized by often changes of concentration. To tread water properly according includes the following unit processes: coagulation, filtration, disinfection by ozone, and final disinfection by chlorine and its compound (chlorine dioxide).

Raw water comes into a fast-mixing chamber. A coagulant (PAC-10WA) is dosed to the pipe before the chamber with the use of pumps Milton-Roy Maxroy B145 at a delivery 900l/h, automatically steered by a SCD system. The SCD steering system keeps the same coagulant concentration. The solution is supplied to the pipes by static mixers. Then, water is directed to slow-mixing chambers, where flocculation process takes place. Then water goes to setting tanks. There are 12 slow-mixing chambers and 12 three-level setting tanks, where contaminations settle at the bottom. The setting tanks are equipped with drift fenders of ZICKERT system, which work automatically according to a given operation scheme. In this system sludge is scraped to the outlet and then to sewage ponds.

From the settling tank, water flows to rapid filters. There are 24 sand-gravel, rapid, gravitational filters at a efficiency 400m^3/h. Only 12 of them are in use. The proper filtration process requires rinsing every 24-72 hour. The filters are rinsed by air using two blowers of DR700T type at a delivery 3000 Nm^3/h and by water in counter-current using two pumps IFV 400 DDTL (delivery 2250 Nm^3/h). Rapid filters washings are led to two Dorra settling tanks. Over sediments water is pumped to the beginning of the whole technological system i.e. before fast-mixing chamber at the delivery 400 Nm^3/h. Sediment is pumped to the sediment ponds using Sarlin pumps (efficiency 200 m^3/h).

After filtration, clean water flows to two ozone tanks, where ozone and sodium hydroxide is added (to correct the pH).The “Mokry Dwor” WTP has the installation of ozone production consisted of: a main air filter, a filter sponge, a compressor VM125 12 of capacity Q=400÷960 Nm^3/min (3 pieces), a cooling water and freon bed (2 pieces), two absorbing driers, three ozone generators at a capacity 11kg O_3/h and 4 pumps to cool.

Because ozone works locally, to prevent distributed water from bacteriological contamination water is finally disinfected using chlorine and chlorine dioxide. These compounds are added before a clean water tank. Chlorine water is produced at the treatment plant and dosed using chlorinators. Disinfection agent dosing are let automatically by concentration analyzer of chlorine or chlorine dioxide.

After disinfection water from the clean water tank is delivered by two pipes ∅ 1200 with pumps of II degree pumping station. The present capacity of clean water tank equals 45,000 m^3. The equipment of the II pumping station consist of 10 pumps including 4 pumps WAFA 50P17 (Q=1650 m^3/h, Hp=5.2 m) and 6 pumps type ABS Z22 (Q=2000 m^3/h, Hp= 60m). Moreover, there are four anti-hammer water tanks at capacity 5÷6 m^3 and pressure 10 atm, which are to stabilize water distribution subsystem operation. The II degree pumping station works automatically and is steered by required pressure in distribution system.

Except for technological line of water production, there are other objects and facilities at the “Mokry Dwor” WTP connected with sediment from settling tanks, rapid filters and sewage from a chemical building.

4. The "Na Grobli" Water Treatment Plant

The plant has been started to operate on 31 July 1871. The plant was designed for the production of 120 000m^3/d, to supply water for 240 thousands inhabitants of Wroclaw. Nowadays it produces about 70 thousand m^3/d. It treats infiltration water. According to development plan of WTP Wroclaw, which goal is improving quality of service, there is forecast of soon modernization the "Na Grobli" WTP. After modernization efficiency of the plant will be 90 000. m^3/d.

The plant is supplied by two pumping plants of infiltration water. They scoop water from water-bearing areas, supplied by water initially cleaned after process of coagulation and sedimentation in the "Mokry Dwor" WTP. The basic function of supplying the WTP fulfils a pumping plant Świątniki with efficiency of about 84,000 m^3/d, supplied with underground water from water intake Radwanice with efficiency 17 000 m^3/d. The WTP is also supplied by a pumping plant Bierzany with efficiency of about 12 000 m^3/d. In the case of disable the Radwanice pumping plant resulting from occurred failures, efficiency of „Na Grobli" WTS will decrease to 72 000 m^3/d. It comes from fact that pumping stations Świątniki and Bierzany takes water from the same water-bearing area. Water to the plant is pressed inside two water-pipes ϕ 850 and one water-pipe ϕ1000. Taken water has attributes of underground water – elevated content of iron, manganese, carbon dioxide. It is characterized by stable quality with regard to physicochemical and microbiological meaning. Water treatment is done in processes of: preaeration, water stabilization, filtration and disinfection.

After WTP modernization water treatment technology will be enlarged of surface coagulation on accelerated filters, intermediate ozonization and sorption an active carbon. In the technological system will be working two intermediate pumping stations. The first treatment process will stay unchanged. Raw water will be still aerated in an iron remover, consisting of 48 aerating towers. Those devices are functionally divided into 12 sections, consisting of 4 towers each. Next, through intermediate pumping station water will be pass onto sand filters, with possibility of simultaneous coagulation in a filtration deposit. Technological string will be equipped with 16 acceleration filters. After process of filtration with the aid of intermediate pumping station water will flow through ozonization chambers and carbon filters (12 pieces including 4 reserve ones) to clean water tanks. Final stage of water treatment, before entering into the distribution network, will be disinfection with chlorine dioxide and/or chlorine.

In the water treatment technological system was designed water stabilization using sodium hydroxide solution.. That process will be two-stage, before sand filters and after carbon filters.

5. Reliability of technical operation of water production subsystem

Water treatment plants are renewable objects of the WSS. To estimate reliability of their operating it is commonly used two-parameter method, where reliability measure is any combination pair of two elements coming from a three-element set *{K,Tn,TP}*, (*K*-availability factor, *Tn*- mean renewal time, *Tp*- mean operation time). Availability factor *K* determine probability that the WTP at any moment t, enough far from start of operation and in given exploitation conditions will be available; it means will do its task, which is water production for inhabitants and industrial of required quality and indispensable amount. Mean renewal time *Tn* is defined as expected value of object failure time *E(Ton)* , while expected value *E(Top)* of operation time *Tp* is determined by mean object operation time between following failures. Renewal and operation time are random function and create fluxes of failures and renewal. In practice it is assumed that fluxes are the simplest Poisson fluxes being characterized by: stationary and no time sequence. For WTP objects, when known is only number and failure removing time, availability factor presents:

$$K = \frac{\mu}{\lambda + \mu} \qquad 5.1$$

where: μ –failure intensity [1/h],
λ –repair intensity [1/h].

It is complex factor taking into consideration failure rate as well as object renewal ability (Wieczysty and others, 2001).

Determining reliability level of WTP (or other objects) there are used reliability diagrams made basing on engineer solutions of a given station. Exploitation connections between particular technical objects of WSS present reliability structures as: serial, parallel or threshold. These structures present reciprocal connection of WTP elements from a point of view of its failure results on the whole system operation.

The paper tries to determine operation reliability of urban water supply subsystem of Wroclaw containing plants “Mokry Dwor” and “Na Grobli”. Calculation was done basing on structural reliability diagrams of an existing water treatment technological string in the “Mokry Dwor” WTP and assumed designing solutions of the “Na Grobli” WTP modernization. Water treatment process in the WTP are done in independent technological strings (described above), which from reliability point of view forms a serial structure. In every mentioned stage of the water production, unit treatment processes are realized on technological systems, forming so called threshold or parallel structure of objects operation. The above analysis allows creating global calculation reliability diagrams of the plants.

For created reliability structures of the „Mokry Dwor” plant, it was performed calculation basing on exploitation data for observation period in years 2001÷2004. Failure time analysis of particular elements was performed basing on the failure records of the objects in “Failure logs”. The records contain information concerning technical objects failures i.e. type, number, date and hour of occurrence and failure time, as well as effects on WTP operation. Moreover, they contained information of scheduled engineer inspections and preventive maintenances of technological devices. This information allows determining reliability characteristics for particular objects and classifying them. In case of the “Na Grobli” WTP reliability analysis was performed basing on design solutions of its modernization. Fundamental reliability factor was assumed basing literature data (Kwietniewski and others 1993; Wieczysty and others, 2001;Zimoch, Wieczysty, 2001;). Obtained analysis results are presented in tables 1 and 2.

Table 1 Values of reliability parameters of technological system of the "Mokry Dwor" WTP

The "Mokry Dwor" WTP arrangement	Availability factor K
Primary pumping station	0.999999369
Coagulation	0.999999504
Filtration	0.999853158
Ozonization	0.996771189
Water disinfection system	0.999998691
Secondary pumping station	0.999999942

Table 2 Values of reliability parameters of technological system of the "Na Grobli" WTP

The "Na Grobli" WTP arrangement	Wskaźniki gotowości K
Primary pumping station	0.996869566
Aeration	1.0
Circulating pump on FP	0.999912354
Contact filtration	0.999998874
Circulating pump	0.999912354
Ozonization	0.999964486
Carbon filters	0.999999919
Water disinfection system	0.999999629
Secondary pumping station	0.999990958

Performed reliability calculation give final result in a form of determined a availability factor of water treatment plants of Wroclaw:

$$K(\text{"}MokryDwor\text{"}WTP) = 0.996622335 \quad 5.2$$

$$K(\text{"}NaGrobli\text{"}WTP) = 0.996648849 \quad 5.3$$

$$K(WSSWr) = K(\text{"}MokryDwor\text{"}WTP) \times K(\text{"}NaGrobli\text{"}WTP) = 0.993282503 \quad 5.4$$

$$K(WSSWr = 0.993282503 > K_W = 0{,}991328855 \quad 5.5$$

where: K_W – required reliability factor for water supply subsystem for settlement over 500 thousands inhabitants having two independent water supply systems (Wieczysty and others, 2001).

6. Summary

Reliability of engineer operation the "Mokry Dwor" WTP was determined basing on exploitation data as reliability factor is K=0.996622335>K_W=0.995654988. It satisfy reliability conditions, what means that the plant at any moment with probability over 0.99 will operate correctly by producing required amount of water and of adequate quality.

The most unreliable element of the "Mokry Dwor" WTP is water ozonization system. It is very important element of technological string, because on its correct operation depends final high quality of water passed to the distribution network, what is guaranteed by Municipal Water and Sewage Company in Wrocław. Reliability of that arrangement is 0.996771189. Calculations have shown that high engineer reliability characterize coagulation and secondary pumping station. It comes from high reservation of elements in this arrangement, what is essential for operation reliability of engineer systems.

Analysis of design solution of the "Na Grobli" WTP modernization has shown that engineer solutions of technological water treatment system satisfies reliability standards. The most failure elements arrangement of the "Na Grobli" WTP is a raw water pumping station, what is caused by a little number of reserve elements and a fact that pumping stations are fed from the same water-bearing region.

Performed probabilistic estimation has shown in addition that the WSS of Wroclaw satisfies required reliability level.

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The Water Distribution Subsystem Failures Affects on Krakow's Water Supply System

Izabela Zimoch

Abstract: The increase of regulation requirement regarding water quality delivered to consumers challenges administrators of water supply systems. Operations intend to improve quality of water and reliability of its supply cannot only limit to subsystem of water production. Nowadays the level of water treatment technology allows reaching good quality of water, much better than formal requirements. Nevertheless the majority of subsystems of water distribution do not fulfill criteria that allow maintaining such quality during delivering it to a consumer. This paper presents the results of preliminary reliability analysis of Krakow water distribution system operating. The analysis was made on the ground of exploitation information, which was concerned the real failures of water distribution system in the year 1996-2003. The reliability estimation contained calculations of basic parameters such as: average operation time without failure, failure intensity value and average repair time. This analysis of water distribution system of Krakow city includes investigation results of dependency of water-pipe network failures upon their diameters, materials and importance consumer's water supply. The failure reasons and effects, which appeared in water distribution system, are showed in this article too.
Keywords: water distribution subsystem, reliability analysis, availability factor

1. Introduction

Application of the reliability theory in engineer system evaluation in Poland and in the world is very common and contains different fields of economy as: military, transport or building. Using that theory for analysis of water supplying systems or some subsystems or elements is already acknowledged as standard. Nevertheless it is still a field of science that is not fully recognized.

The inspection and reliability estimation of a distribution system is a complicated and difficult question. It requires considering a wide range of random events. First of all it results from multifunctionality of such system: supplying water to buyers in needed amount, quality, under proper pressure and at the any moment. Moreover structure of the system, multitude of different elements consisting of, is another aspect that influences a difficulty of estimating reliability. Nowadays, water demand in the system has considerably decreased, the reliability inspection are performed in particular to estimate weak points of the water supply system, where frequency of accidents and type of effect is the greatest. The above inspections gives in addition possibility of marking out such point of the system where exists a potential risk of secondary water contamination, caused by change of network running hydraulic parameters resulting from lack of water demand stability in the system. Small reliability and occurring damages may lead not only to disable operation of water supply system (WSS), but also can risk consumers health and lead to other damages of water-pipe enterprises**.** Qualitative and number estimation of negative events lets draw more precise conclusions and use an engineer performance having in view eliminating them to improve exploitation conditions of system functionality. Hence, reliability estimations of WSS, corresponding not only to engineer structure but also allowing for qualitative aspect of the product, determine essential information for effective management and exploitation of WSS.

This paper is a try of presenting reliability estimation of a water supply system. Analysis was performed for distribution systems of Krakow city. Basic of investigation was a wide base of information related with damages of the water distribution system in exploitation period 1996-2003. Reliability estimation has included determining basic parameters such as: parameter of failure flux, mean time of running without damage or mean time of repair. It was also analyzed different dependencies between possibility of pipe damage and their diameter, material and if their aim was supply water to a buyer. Elaboration consist also of results of causes and effects analysis in the pipe network.

The paper was written within the framework of realization of research project KBN 5T07E 044 25 under the title “Determination of reliability operation model of water supply system (WSS) in the aspect of secondary water contamination in a water pipe network”.

2. Water distribution subsystem of Krakow city

The beginning of the water-pipe network of Krakow is dated to 15 February 1901. The network was then running in the system with the initial reservoir, from which water was gravitationally delivered to city network by cast iron pipeline ∅750 of length 2,870 meters. At the beginning of the last century a substance of water-pipe network in Krakow were pipelines of diameter ∅700 ÷ 425 mm. At the starting of operation of the water-pipe in the city there were only 206 home service pipes and 43 street wells, giving possibility to inhabitants of Krakow of general access to water. At the end of the first year of exploitation, total length of the water-pipe network was 80,967 m (Rafalski L. and others, 1993).

Now the subsystem consists of the complex water-pipe network with numerous reservoir of pure water. Krakow is divided into separate water-pipe zones fed from independent sources, which are 4 top water intakes supplied from rivers: Raba, Rudawa, Dłubnia and Sanka and one underground intake placed in Mistrzejowice. Localization and engineer solutions of supply systems with water purification plants ensure (on normal conditions of exploitation) reliable operation of the water supply subsystem to the city. The present Krakow WSS is classified as so called systems with excess that means the system has a reserve of not used production capacity in the level of 91% with regard to actual water requirement in the city. In the case of a stoppage of water supplies from one intake (e.g. as a result of contamination) there is possibility of a damage supply to some part of city using remaining sources .

Division into supply zones results from configuration of city terrain, determining the position and the range of the zones. In Krakow there are 13 different pressure zones incorporating areas from 225 m above sea level to 366 m above sea level in northern region of the city. The reliability of water supply to the inhabitants of high zones is ensured by hydrophones and intermediate sewages (Archival materials, 1996-2003).

Actually (at the end of 2004) total length of the water-pipe network in the area served by Krakow's Water Company is 1,849 km. The extensive water distribution system of Krakow consists of transit network of pipes ∅1400, it amount totally to 18 km length of the system. The main network of the system is pipes of diameter ∅1200 - ∅350 and total length 265 km. The greatest part of the system (in regard to length) is distributing pipes of diameters from range ∅325 - ∅80, which total length is 1100 km of water-pipe network. The last linear element of the discussed system is service pipes, being a system of 466 km pipes of diameter ∅100 - ∅25. With regard to over 100-year exploitation of water-pipe network, it is characterized by essential age differentiation. In the city there still exist pipelines from the first decades of the distribution system operating period, of the age over 50 years

The many-year exploitation and development of the water-pipe network contributed to its material differentiation. The most significant part in material structure take steel pipes, being 32% of total length of the network (600 km) and next pipes made of cast-iron (26%, 479 km), PCV (23%, 426 km) and others materials (19%, 344 km). The integral part of the water-pipe network is all kinds of utilities. Plumbing fixtures allow using them according to its destination and makes easier servicing, controlling and exploitation of the system.

Efficiency of the system distribution functionality depends also on exploitation of the network stilling-reserve reservoirs. The reservoirs are designed for storing excess of water and next to supplement water supply in the periods of increased requirement. The additional task some of the reservoirs is stabilization of pressure in the supply areas, which changes regarding to different part of a day, depending on water usage in the city. In the first years of last century it has started running the first network reservoir Kościuszko of capacity 5 thousand m^3. The reservoir in 1987 was developed and now it has total store ability at the level 25 000m^3. With the water-pipe network developing it was increasing the amount and capacity of the reservoirs. Now in Krakow there run 11 reservoirs complexes of total capacity 276.2

thousands m^3. They are mainly round terrain reservoirs with single- or multi-chamber system of reinforced concrete construction. Particular attention should be paid to the greatest reservoirs complex Siercza storing 158.5 thousands m^3 of water from the Raba supply system, what is almost a daily requirement in the city. So much treated water stored in the city raises reliability of the WSS of Krakow and decrease a water-supplying enterprise's risk of not fulfilling its primary function - supplying water to buyers (Archival materials, 1996-2003).

3. Factors of the reliability analysis

Most of objects and devices consisting of the water supply system are classified as so called renewable elements that mean such that can be taken in processes of running and renovation. A determining exploitation condition, definite as reliability states, is a ground of selection and estimation of suitable reliability factors of these objects. Two reliability states are distinguished like: state of running, it means full ability and state of partial or complete inability. Taking into consideration above exploitation states and specification of objects and water-pipe devices in the reliability analysis of their operating there are chosen from under named defined factors (Kwietniewski M. and others,1993).

Mean operational time between failures is defined as expected value of random variable T_p' determining operational time between following failures:

$$T_p^{'} = \frac{1}{k+z}\left(\sum_{i=1}^{k} t_{pi} + z \cdot t\right) \qquad 3.1$$

where: k – number of operational periods of failing objects,
t_{pi} – value of ith operational period [d],
t – observation period [d],
z – number of operational periods of unfailing objects, and: z = N-M, and N is a number of tested objects, and M is a number of failing elements.

Mean time of being failed is an expected value of random variable T_o' determining time of renewal, considering expectation time and time of removing occurred damage:

$$T_o^{'} = \frac{1}{n_o}\sum_{i=1}^{n_o} t_{oi} \qquad 3.2$$

where: n_o – number of failures during tested period,
t_{oi} – time of lasting ith renewal [h].

Parameter of failure flux characterizes reliability of renewal and two-stage objects, it is an unconditional probability of occurring an object failure in time period Δt, independently whether at the beginning of period it operates correctly or is failed, in case of linear objects:

$$\omega^{'}(t)=\frac{n(t,t+\Delta t)}{L\cdot\Delta t} \quad 3.3$$

where: n(t, t +Δt) –failure number in period Δt,
Δt – length of time period that observation period was divided [year],
N – number of tested objects
L –length of tested pipes [km], (Kwietniewski M. and others,1993).

Availability factor is probability that object, system or subsystem will be ready to operate at the moment t. In practice this factor means probabilistic estimation of object, system or subsystem availability in the range of execution of a given task:

$$K_{g}^{'}=\frac{T_{p}^{'}}{T_{p}^{'}+T_{o}^{'}} \quad 3.4$$

where: $T^{'}_{p}$ – mean time of operation between failures [d],
$T^{'}_{o}$ – mean renewal time[d], (Kwietniewski M. and others,1993).

Operating probability of an element R(t) is defined as probability that in the interval of time (0, t) between following failures, beginning from starting operation after failure, object will not fail. Flux of water-pipe objects failure is flux without results, single and stationary, and renewal process is Poisson process, for which operating time has exponential distribution. With such assumptions object operating probability has form:

$$R(t)=\exp(-\omega\times t) \quad 3.5$$

Mean repair intensity is a parameter defines number of inability removed a time unit:

$$\mu^{'}(t)=\frac{r}{t_{n}} \quad 3.6$$

where: r –failure number of renewed elements ,
t_n – summary time of repair of tested objects in observation period [h].

Practical use of abovementioned formulas in reliability estimation of operation of WSS base on indispensable information obtained from water-pipe exploitation. Source date are failure protocols, failure cards, exploit books, operating books of a machine, log of standby service, registers of failures etc. Above documentation should contains data such as: date and hour of failure, repair, overhaul etc., lasting time, description of the event containing type of failure and its effects for the subsystem or whole WSS. Unfortunately, information obtained form exploitation are not always enough, what considerably makes difficulty to perform full analysis. They are not systematic and sometimes allow only stating a fact of failure without any numerous data useful in finding appropriate factors.

4. Reliability analysis of WDS's operating for Krakow city

The water distribution subsystem of Krakow is a large and extensive one, with complicated topology and different types of utilities. Relatively frequent failures are caused by many-year exploitation of the pipeline, by negative influence of environment and often by low quality of materials, of which the pipeline was made after World War II. In the course of the year Water Company in Krakow registers about 2000 failures of the water-pipe network. Performed analysis of pipe network failure has shown that most common reason of pipeline failures is corrosion damage. Moreover it was stated dependency between intensity of failures and season of the year. Relatively the largest number of failures falls to autumn-winter period

Three independent repair brigades, servicing water distribution regions: Centrum, Podgórze and Nowa Huta, remove damages and their effects in the city area. Every region has its own Water-pipe Service, which is due to remove damage after announcement. Efficiently working the Water-pipe Services increase the reliability of WSS operating by minimizing time of elements being failure. Efficient intervention reduces also effects of failure and risk of the enterprise, which operates extensive and complicated WSS.

Performed reliability analysis paid attention to estimation and classification of operating conditions of water supply system of Krakow. Above studies were performed using archival system exploitation data in years 1996-2003. 1,849 km of water-pipeline were analyzed. Studies were bidirectional. First direction of study contained reliability analysis of water-pipelines in regard to performed function in water distribution system in the city (dividing them into distribution, main and transit networks and terminal elements). Second aspect of study was preliminary reliability estimation of separated linear elements of the distribution system.

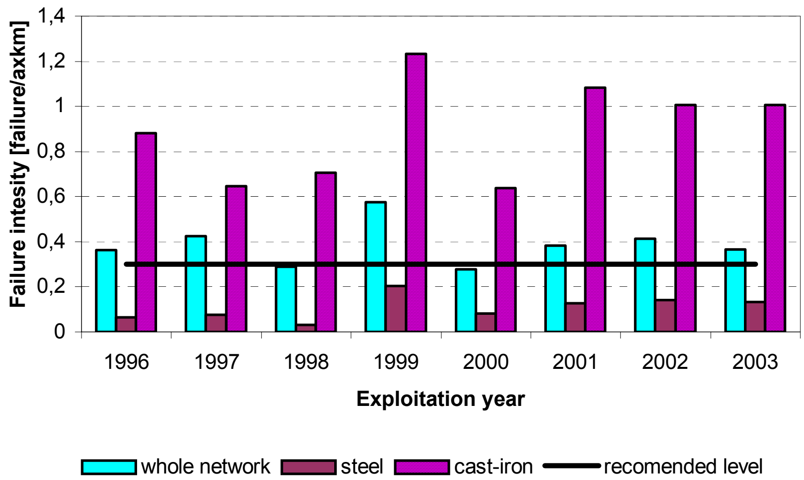

Figure 1 The damage intensity value of main networks in the years 1996-2003

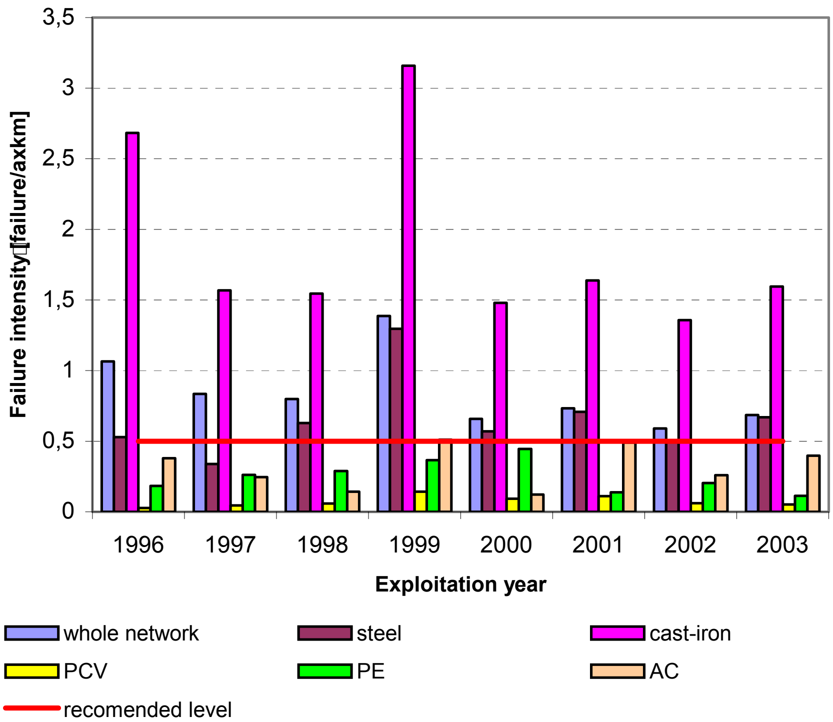

Figure 2 The damage intensity value of distributive networks in the years 1996-2003

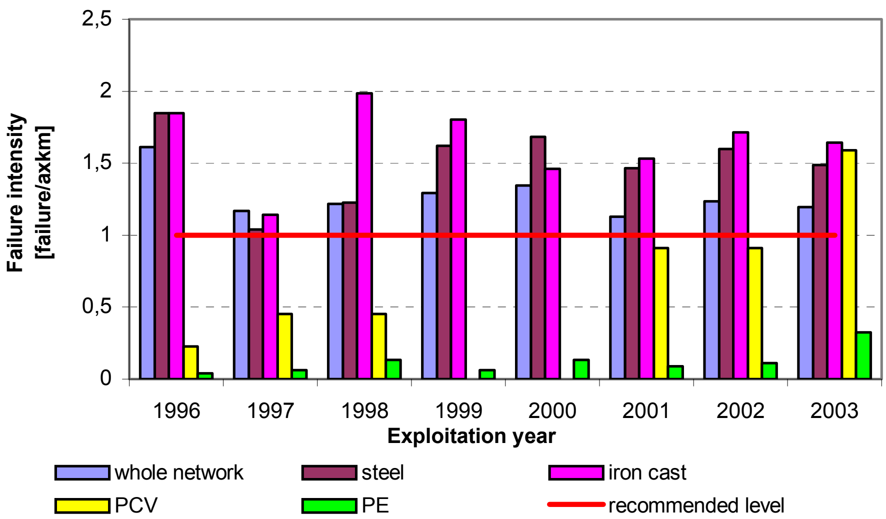

Figure 3 The damage intensity value of home terminal in the years 1996-2003

Considering specification of studied water-pipeline objects, two states of their operation and Poisson character of renewal process, in reliability analysis of that elements were found following factors: mean operating time between failures T'_p, parameter of failure flux $\omega'(t)$, renewal intensity $\mu'(t)$, mean being failure time T'_o and availability factor K'_g and operation probability of object $R(t)$. The results of performed analysis are contained in diagrams figures 1 - 3 and in table no 1.

Table 3 Comparison of value of reliability parameters of water distribution system of Krakow city

Reliability factor	Unit	Network type		
		Transit and main pipes	Distributing pipes	Water-pipe terminals
Mean failure intensity	failure/axkm	0,362	0,844	1,273
Mean operating time	d	52,38	22,85	48,06
Mean being failure time	h	9,78	8,36	8,19
Mean repair intensity	h^{-1}	0,10	0,12	0,12
Availability factor	-	0,982586	0,867265	0,98169
Operating probability	-	exp(0,0191xt)	exp(0,0438xt)	exp(0,0208xt)

5. Summary

According to the recommended level of failure intensity factors the smallest failure intensity characterizes main and transit network. Failure rate of distribution network is over twice greater (0.844 failure/axkm) and home terminal even three times (1.273 failure/axkm) with regard to failure intensity of main and transit networks (0.386 failure/axkm).

The main part of failures of studied objects is cast-iron pipe and then steel ones, asbestos cement (AC), network made of PE and PCV. Parameter of failure intensity of cast-iron is $\lambda^*(t)>0.890$ failure/axkm. Cast-iron pipelines make about 26% of length of water-pipe network, and so often accidents bring about that make the greatest exploitation problems.

The most exposed to corrosion are steel pipes, for which failure intensity was $\lambda^*(t)=0.376$ failure/axkm.

On reliability of cast-iron pipes decides mainly joints failures. During the analysis period such failures has been 4,027. In the Krakow distribution system, cast-iron pipes are old, approaching the expiry term of working life (36% of pipes in the network are 25-50-year-old). Majority were connected with pipe bells traditionally tighten with cord and aluminium foil, which failure intensity is greater then connections using new technologies.

From a reliability point of view, it is possible to state that despite the high failure intensity parameter, the water-pipe network generally was characterized by high availability, because stationary factor of availability for pipes of the main and transit network was 0.982586 and for terminals 0.98169. In the case of the distributing network value of that parameter reach a level 0.867265, what results from high failure-rate of pipes with diameter 100 and 150 mm.

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Theme V

Surface and Groundwater Resources (Including Floods and Droughts)

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Impact of Potential Natural Vegetation of a Mountainous Catchment on the Runoff Regime

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Zora Papánkova, Oliver Horvát

Abstract: Distributed hydrological rainfall runoff modelling was applied for estimating changes in the runoff regime using an extreme land use change scenario. The effect of the change of the present land use to the historical potential natural land use was simulated using climate data from 1981-2000. The upper Hron river basin, which has an area of 1766 km^2 and is located in central Slovakia, was selected as the pilot basin. The WetSpa physically-based rainfall-runoff model with distributed parameters was used for modelling of runoff from rainfall and melting snow. Parameters of the model were estimated from three digital map layers: land-use map, soil map and digital elevation model. Changes of the long-term mean annual runoff, mean monthly discharges and maximal mean daily discharges with a return period from 5 to 100 years were compared for present and potential natural land use.
Keywords: Distributed rainfall-runoff model, land use change scenarios, runoff regime

1. Introduction

Recent extreme flood events in Europe have originated public discussion on the question whether the frequency and severity of floods have been increasing and if such changes could be attributed to anthropogenic influence. The effect of deforestation on runoff is another important issue, which is frequently discussed between environmentalists, hydrologists and water resources managers. This paper presents partial results of a wider research project, in which the impact of land use changes on runoff was investigated. A distributed rainfall-runoff model, which is parameterized on the basis of satellite imagery, digital terrain models, and diverse digital maps, was applied to the Hron catchment in Central Slovakia. Two types of land uses are investigated here: the present land use, which is used for model validation and as a baseline for comparison, and an almost total afforestation of the basin, which corresponds to the historical potential natural land use.

The influence of land use on runoff generation is very complicated; as land use and soil cover have an effect on interception, surface retention, evapotranspiration, and resistance to overland flow. The influence of land use changes on storm runoff generation has been frequently studied in the recent past. Due to the complexity of the processes involved, the magnitude of their impact on runoff components and subsequent flood discharge into the river system is still highly uncertain (Niehoff et al., 2002). Most scientists used either conceptual rainfall-runoff models (e.g. Koehler, 1992) or distributed physically-based rainfall-runoff models (e.g. Parkin et al., 1996) for their investigations. In Slovakia, several physically based hydrological models with distributed parameters were recently used for assessing land use or climate change impact on runoff and snow melting processes and for simulating sediment transport, e.g. Wasim, Topmodel and UEB-EHZ (Kostka and Holko, 2002), WetSpa (Papánková, et al., 2005), and AGNPS (Pekárová, et al., 2004).

Both modelling approaches have their merits and limits. Conceptual rainfall-runoff models use less parameters, but describe the transformation of rainfall to runoff with simple concepts. However these simplifications prevent the transfer of measured physiographic properties and state variables directly into modelling parameters. Distributed physically-based rainfall-runoff models use a high number of parameters, which may be difficult to determine and the modelling concepts often do not describe the occurring runoff generation processes adequately because they use equations based on small scale physics and these are applied on a grid scale. In an ideal case, model parameters are measured or estimated from catchment characteristics.

To overcome some of these problems and also to gain insight into the processes in the catchment, others investigated the influence of land use change on storm runoff experimentally by identifying the dominant runoff processes in catchments (e.g. Naef et al. 1999). The effects of land use change on the hydrological responses of catchments, particularly those connected to forest management, have been documented by smaller watershed scales (e.g., see Bosch and Hewlett, 1982; Harr, 1986; Stednick, 1996).

In this paper it is expected, that distributed parameter models are suited to predict the hydrologic effect of land use change especially if their parameters have adequate physical interpretation, and the model structure allows for correct representation of their spatial variability. The availability of spatially distributed data such as digital elevation model, land use, and soil information makes the use of distributed models convenient. Changes in the runoff regime in the Hron river basin due to expected radical land use change are estimated using the modified WetSpa model. The impact of the historical potential natural vegetation on runoff formation is estimated, in particular changes in the long-term mean annual runoff and its components, mean monthly discharges and changes in maximal mean daily discharges are discussed.

2. Methodology

2.1 Description of the WetSpa rainfall-runoff model

WetSpa is a grid-based distributed hydrologic model for water and energy transfer between soil, plants and atmosphere. It was developed at the Vrije University in Brussels by Wang et al. (1996), further developed by De Smedt et al. (2000), and Liu et al. (2003). In the model four zones are considered in the vertical direction for each grid cell: the vegetation zone, root zone, transmission zone and saturated zone. In these zones following hydrologic processes are modeled: interception, depression, infiltration, evapotranspiration, percolation, water balance in the root zone, surface runoff, interflow and ground water flow. The model computes hydrographs of all runoff components for any location in the catchment and in the channel network. The spatial distribution of water balance components can also be assessed.

The total water balance for each grid cell is computed in four control stores: the plant canopy, the soil surface, the root zone, and the saturated groundwater aquifer. At the surface the water balance is differentiated for the vegetated, bare-soil, open water and impervious parts of each cell in order to account for the non-uniformity of the land use.

A mixture of physical and empirical relationships is used to describe the hydrologic processes in the model. Rainfall excess is calculated using a moisture-dependent modification of the rational formula. The runoff coefficient is the function of the land cover, soil type, grid slope, the rainfall depth and the antecedent moisture content of the soil.

Both percolation and interflow are assumed to be driven by gravity. Percolation from the root zone is a function of the hydraulic conductivity, the moisture content and a soil pore size distribution index. Interflow is assumed to occur in the root zone only when the soil moisture is higher than field capacity. Darcy’s law and a kinematic approximation are used to estimate interflow in each cell as a function of the hydraulic conductivity, soil moisture content, grid slope, and the vegetation root depth.

The Thornthwaite and Mather method is used to estimate actual evapotranspiration from the land surface. It is based on potential evapotranspiration, vegetation type and stage of its growth, and soil moisture content of the soil. Total evapotranspiration is the sum of evaporation from the land, intercepted water, depression storage and from groundwater.

The groundwater flow is modeled by a semi-distributed sub-catchment scale scheme, special knowledge about the bedrock is not required. Outflow from a linear reservoir is used to estimate groundwater discharge from each underground sub-catchment.

WetSpa routes surface runoff and interflow from each cell in the watershed to selected control points (cross sections in the river network) using an approximation of the the diffusive wave method. A two parameter (the mean travel time for each cell and its variance) approximate solution of the diffusive wave approximation was proposed by De Smedt et al. (2000). It uses the form of an instantaneous unit hydrograph relating the discharge at the end of a flow path to the available runoff at the start of the flow path. The mean travel time and its variance for each grid cell are considered to be spatially distributed, and can be obtained with GIS methods along the topographically determined flow paths as a function of flow celerity and dispersion coefficient. The groundwater flow is added to runoff at the sub-catchment outlets to produce the total streamflow.

The WetSpa model was chosen in this study because it has a rather simple structure and runoff generation is parameterized in each grid cell by its slope, land use and soil properties. All model equations are carefully chosen to maintain a certain physical basis and preserve simplicity. Apart from the large number of physically-based parameters, which are derived from the physiographic properties of the catchment, the model also uses 11 global parameters, which have to be estimated or calibrated.

Input data and model parameters are prepared in an ArcView GIS extension, the whole processing spatial distributed data is set to this interface. The minimum necessary spatial map layers of the physiographical characteristics are a digital elevation model, a map of the land use types and a map of the soil types. From these maps of other physiographical characteristics are derived.

The hydrological and climatic data are prepared in text format. Daily or hourly total precipitation values from rain gauge stations, the mean daily or hourly values of temperature and the mean daily or hourly measured discharges at the basin outlet are needed.

2.2 Pilot basin and input data

In this study the upper Hron River basin up to the Banská Bystrica gauging station, which has an area of 1766 km^2, was selected as the pilot basin. The upper Hron River basin is located in central Slovakia; the minimum elevation of the basin is 340 m a.s.l.; the maximum elevation is 2004 m a.s.l.; and the mean elevation is 850 m a.s.l. The present land use is described in Table 1. The digital elevation model (DEM) with a resolution of 100 x 100 m was interpolated from digitalized contour lines of the Basic Map Work of the Slovak Republic (1:10 000). The model input and parameter layers derived by the ARCView extension include the map of flow accumulation, flow direction, generated stream network, slope map, and a map of the hydraulic radius and delineation of the basin into sub-watersheds. The land use map originated from the thematic mapping of Slovakia within the CORINE project. The land use was divided into 14 categories that are required for the WetSpa (Liu and Smedt, 2004).

Daily total precipitation was measured at the 20 rain gauge stations, while mean daily temperature was measured at 6 climatic stations. The model was calibrated up to the outlet gauging station at Banská Bystrica for the period 1981-2000 in a daily time step. The Nash-Sutcliffe coefficient achieved for the whole 20-year period was 0.732.

2.3 Land use change scenarios and their impact in the Hron River basin

Designing scenarios is a widespread technique in business, planning and policy consulting, because it offers the opportunity for assessment of the present and possible future situations and the mitigation of mismanagement (Niehoff, 2002). The extreme land use scenario created for this paper, the historical potential natural vegetation in the catchment, was selected in order to assess the potential change of runoff in the catchment due to all existing present antropogenic activities. The present land use, when compared to the natural, is characterized by urbanization, deforestation, by changes in the structure of forests and agricultural land near the rivers and on smaller slopes. The potential natural land use scenario is representing the land use closest to that of a natural, pristine landscape, with almost the whole basin area covered by forest. Land-use categories of the scenarios are summarized in Table 1.

Using theses scenarios, runoff from rainfall and snowmelt was simulated in daily steps for the 1981-2000 period. The resulting changes in runoff were evaluated by comparing the simulated discharges and their statistical characteristics for the existing land use (baseline scenario) and the scenario consisting of historical potential natural vegetation.

In order to assess flood regime changes due to the land use changes, a statistical flood analysis was undertaken. For both land use scenarios maximum mean daily discharges for return periods of 2, 5, 10, 20, 25, 50 and 100 years were estimated by the DVWK/101 (1999) method used in Germany. The GEV distribution was chosen (and tested) as appropriate. To estimate the parameters of theoretical distribution the method of probability weighted moments (PWM) was used.

The comparison between mean daily discharges for the potential natural and the existing land use was expressed by values of the long-term mean discharges and their components (Table 2); the long-term mean monthly discharges and their statistical characteristics (Table 3) and in N-year annual maximum floods (Table 6.). The changes in runoff as opposed to the existing land use are summarised in Table 4., Table 5. and Table 7.

Table 1 Summary of basic land use categories expressed as percentage of basin area for both scenarios

	Percentage of catchment area (%)	
Category of land use	Actual land use	Potential natural land use
Deciduous forest	9	85
Mixed trees	25	0
Evergreen coniferous trees	29	12
Urban	3	0
Agricultural land	17	0
Grassland, bush and other	17	3

Table 2 Long-term mean discharges [m3.s-1] and its components for both scenarios

Land use	Surface runoff	Interflow	Baseflow	Total runoff
Actual land use [$m^3.s^{-1}$]	3.86	7.70	13.82	25.39
Potential natural land use [$m^3.s^{-1}$]	0.32	3.66	13.79	17.76
Actual land use [$mm.year^{-1}$]	68.98	137.53	246.77	453.28
Potential natural land use [$mm.year^{-1}$]	5.72	65.28	246.11	317.11

Table 3 Long-term mean monthly runoff and its variability for both scenarios

Month	Mean runoff [$m^3.s^{-1}$]		Variability – Cv ( )	
	Natural land use	Actual land use	Natural land use	Actual land use
1	17.37	20.50	0.70	0.78
2	16.70	19.96	0.54	0.64
3	27.35	35.18	0.48	0.59
4	38.06	52.87	0.46	0.60
5	27.36	39.63	0.46	0.63
6	20.03	31.02	0.59	0.72
7	12.23	18.19	0.61	0.75
8	7.16	12.67	0.62	1.38
9	6.72	13.85	0.92	1.42
10	9.42	15.94	1.10	1.27
11	14.84	23.89	1.03	1.09
12	16.10	21.16	0.70	0.79

Table 4 The change in long-term mean annual runoff and its components

Difference	Surface runoff	Interflow	Baseflow	Total runoff
Scenario – Baseline [$mm.year^{-1}$]	-63.3	-72.2	-0.7	-136.2
(Scenario – Baseline) / Baseline [%]	-92	-53	0	-30

Table 5 Change in long-term mean monthly runoff and its variability

Month	Mean runoff [$m^3.s^{-1}$]		Variability – Cv ( )	
	(Scenario – Baseline) / Baseline	Scenario – Baseline	(Scenario – Baseline) / Baseline	Scenario – Baseline
1	-15.29	-3.13	-9.73	-0.08
2	-16.33	-3.26	-15.20	-0.10
3	-22.25	-7.83	-18.46	-0.11
4	-28.02	-14.81	-24.15	-0.14
5	-30.97	-12.27	-28.08	-0.18
6	-35.42	-10.99	-17.05	-0.12
7	-32.79	-5.96	-18.44	-0.14
8	-43.53	-5.52	-54.93	-0.76
9	-51.46	-7.13	-35.43	-0.50
10	-40.88	-6.52	-12.94	-0.16
11	-37.89	-9.05	-5.50	-0.06
12	-23.92	-5.06	-10.42	-0.08

Table 6 N-year values of annual maximum mean daily discharges

Design floods [$m^3 s^{-1}$]							
N [years]	2	5	10	20	25	50	100
Actual land use	115	157	194	240	257	319	395
Potential Natural land use	60	72	79	84	87	92	97

Table 7 Changes in N-year values of design floods as opposed to the actual land use (in $m^3 s^{-1}$) and percentage

Changes in design floods [$m^3 s^{-1}$]							
N [years]	2	5	10	20	25	50	100
Natural land use [$m^3 s^{-1}$]	-55	-85	-115	-155	-170	-227	-298
Natural land use [%]	-48	-54	-59	-65	-66	-71	-75

The comparison between mean discharges for the historical potential natural land use scenario and the existing land use suggests that the almost complete afforestation of the basin could lead to a very significant decrease in mean runoff values and also to the reduction of their variability. The average annual runoff depth would decrease almost 140 mm year^{-1}, which represents a difference of about -30% from the existing state. From the runoff components, largest decrease was estimated for surface runoff, i.e. -63 mm year^{-1} and -92%, compared to the existing state and the interflow, which had a decrease of -72 mm year^{-1} and -53% from the present state. For this scenario, there was no apparent change in the baseflow. Decrease in runoff is also predicted in the mean monthly values, especially for the months of April, May and June in absolute values and in the summer and autumn months in relative values. Large decrease in design floods, as opposed to the actual land use, can be seen for the historical natural land use scenario especially for high return periods (-298 $m^3 s^{-1}$; -75 % for the return period of 100 years).

The removal of forest cover is known to increase stream flow as a result of reduced evapotranspiration and increase peak flows as a result of higher water tables (e.g. Mattheusen, et al., 2000). Simulation results tend to confirm this tendency, the expected change in flood runoff seems to be large. However given the fact, that almost 35% of the area was afforested (including impervious urban areas), it could be considerer as acceptable, since the mean annual maximum flood decreased to one half. The large decrease of floods with higher return periods can partly also be attributed to statistical extrapolation.

3. Conclusions

Results of simulating changes in runoff due to land use change showed that the scenario representing the land use closest to that of a natural, pristine landscape, when almost the whole basin area is covered by forests, could have a significant effect on changes in runoff and design floods in the upper Hron river basin. For this scenario, a significant decrease in total, surface and sub-surface runoff and interflow as well as design maximal mean daily discharges were indicated. However it is necessarily to realize that these consequences could also be attributed to the parameterization of the forests and grassland land use types in the model (root depth, interception capacity, surface roughness), which affect the process of forming partial runoff components in the model. Therefore the same land use change could have a different manifestation in other rainfall runoff models.

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Regional Methods for Design Flood Computation in Slovakia (Review and Comparison)

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Abstract: The recent floods in Central Europe have resulted in concerns about the reliability of flood frequency estimates in the region. As a consequence, also the currently used regional flood frequency estimation methods are being re-visited. In the study several regional flood frequency estimation approaches were compared with annual and seasonal floods using flood data from 251 basins in Slovakia. Traditional regional regression formulae between flood quantiles and the basin area and other physiographic basin characteristics were considered and the Hosking Wallis methodology was also applied in estimating floods at regional scale. Subjective delineation of regions was complemented by the use of objective regionalisation approaches based on more complex climate-soil-vegetation control characteristics and regional frequency analysis. A comparison of flood quantiles computed by regional approaches with statistical reference values was performed. The applicability of the compared methods for design purposes was discussed.
Keywords: seasonality analysis, design values, engineering hydrology, N-year maximum discharges, pooling scheme

1. Regional flood formulae for estimation of design floods in Slovakia

The tradition of the application of empirical methods and regional flood estimation procedures for the determination of design flood discharges on small-ungauged catchments in Slovakia dates back to the first half of the last century.

The computational methods have been based on the indirect determination of 100-year flood discharges from catchment size in geographically contiguous zones (regions). Envelope curves have been applied in some cases to arrive at a safe relationship for practical engineering applications.

The most frequently used regional formula is:

$$q_{100} = \frac{A}{(F+1)^n}. \quad 1.1$$

where q_{100} is the 100-year maximum specific discharge [$m^3 * s^{-1} * km^{-2}$], F is the catchment area [km^2] and A, n are regional parameters.

To make the formula flexible and to account better for local runoff-forming conditions different from the average type, diverse multiplicative correction factors (e.g. for the catchment shape, percentage of forested area, presence of lakes and swamps, etc.) have been proposed in the past.

Floods with shorter return periods are computed with regional frequency factors from the 100-year discharge as:

$$Q_N = a_N . Q_{100}, \quad 1.2$$

where Q_N is a design flood discharge with a return period of N years, a_N is a regional frequency factor, Q_{100} is a 100-year design discharge computed from the regional formula (1.1).

This type of formula has been employed with slight modifications (usually in the correction factors and regional parameters) by several authors, e.g. by Bratránek (1933), Dub (1940, 1957), Čermák (1954), Halasi-Kun (1968, 1980), Dub, Němec (1969), HP ČSSR, III. (1970). For the most popular formulae of Dub, Halasi-Kun, the Czechoslovak Hydrometeorological Institute - HPIII method: 8, 7 and 26 regions were suggested, respectively.

The Slovak Hydrometeorological Institute (SHMI) has adopted this approach for the regional estimation of design discharges for the catchment areas greater then 20 km2 since 1954. The last regionalisation was revised in 2003 and published as a guideline by SHMI. Together 65 regions were derived for the whole territory.

2. Methods based on the results of the revision of the traditional methods for regional flood frequency analysis in ungauged catchments in Slovakia

In Kohnová, Szolgay (1995, 1996) a comparison of design floods computed by the versions of traditional regional formulae with statistically derived 100-year floods from new hydrometric data from 261 small and mid-sized catchments was discussed. The comparison has shown that the most popular formula of Dub can be still regarded as an envelope curve to the statistically derived values in almost all regions. Other formulae show similar behavior.

Other regionalisation schemes based on different discriminating regionalisation concepts (such as catchment geomorphology, boundaries of the main catchments of Slovak rivers, etc.) have been tested using the traditional form of the formulae in Kohnová (1997) and Kohnová, Szolgay (1998). It has been also proposed to reduce the heterogeneity in the genesis of floods by separate analysis of rainfall and snowmelt-induced floods. Probably due to the high heterogeneity of runoff-forming factors, no acceptable regional regression curves could have been defined, and the envelope curve approach has to be accepted again. This in turn has introduced a safety factor into the formulae. Despite the fact that the safety factor has been reduced in comparison with the traditional approach, it has become apparent that a regression between flood discharge and catchment area alone is an inadequate basis for a formula without a regional safety factor. New engineering and water resources planning tasks, like river restoration, wetland protection and design have also called for approaches not without a safety factor.

3. Methods based on regional regression

In Szolgay, Kohnová (1997) and Kohnová, Szolgay (1998) regional estimation of the mean annual maximum seasonal flood and its coefficient of variation from catchment characteristics by regional regression was attempted in continuous hydrologic regions. Flood quantiles were determined from the two parameter lognormal distribution rather than using regional frequency curves derived traditionally with the help of frequency factors from the estimated regional 100-year flood. The approach was not recommended for practical use, since the variation of the estimated dependent variables in the control catchments was high for practical applications.

As the next logical step, the methods of delineation of regions were proposed to be changed. For the high core mountain region, volcanic mountain region and flysh region of Slovakia four methods of regionalisation were proposed and tested and formulae for computation of the mean annual seasonal floods and their standard deviation were derived in Kohnová, Szolgay (1999, 2000).

In the first method, called Traditional method (I), the whole study area was considered to be a compact region as it is suggested by the tradition of flood analysis in Slovakia and also for comparison.

Secondly, it was attempted in the Geomorphological method (II) to divide the study area into continuous geographic sub-regions based on similarities in the geomorphological properties of the landscape.

In the third approach - the Variability method (III), logical regionalisation principles were applied. The idea of geographical regions was abandoned and pooling of catchments with existing flood observations was attempted into groups (regional types) with similar values of the coefficient of variability of flood frequency curves of the individual sites.

In the fourth method, the Physiographic method (IV), physiographical properties of basins were used as variables to define regional types with similar physiographic characteristics. Cluster analysis was based on several combinations of catchment characteristics with little correlation among each other.

A multiple regression method was used to find the formulae for the estimation of the first two moments of maximum discharges. The mean (Q_{pr}) and the standard deviation (S_d) of the maximum annual seasonal flood series were estimated by a regional regression equation of the form:

$$Q_{pr} = k.A^a.B^b.C^c... \quad 3.1$$

$$S_d = i.U^u.V^v.W^w... \quad 3.2$$

where i, k, a, b, c ... and u, v, w … are regional parameters, and A, B, C ... and U, V, W…are climatic and physiographic catchment characteristics.

The two-parameter distribution functions (e.g. lognormal, Gumbel) were used to estimate the design flood values. The comparison of values of relative differences $(X-Y)/Y$ between Q_N computed using the selected distribution (X) and from the regional formulae (Y) showed, that the distribution of the relative differences seemed to be slightly skewed. This again suggests a need for further analysis. Although all methods give comparable results, Method (III) and (IV) performed as the best ones and were recommended for practical application.

In 2005 SHMI derived a regression formula for direct estimation of 100-year specific discharges ($q_{100.max}$) for whole Slovakia and calculated in 340 water gauging stations with minimum record length of 20 years the residual deviations between values of q_{100max} derived from the regression formulae and using statistical analysis according the DVWK methodology (1999). According to the residual deviations values the territory of Slovakia was divided into 10 regions, Podolinská (2005).

4. Methods based on the Hosking and Wallis methodology

The growing number of gauging stations in small basins with longer records made it possible to test how some of the new concepts of regional homogeneity and regional flood frequency analysis reported in the literature (e.g. Acreman, Sinclair, 1986, Zrinji, Burn, 1994, Meigh et al., 1997, Hosking, Wallis, 1997 and FEH, 1999) perform in the specific physiographic conditions of Slovakia. In these methods the concept of regions does not refer only to contiguous zones identified by geographical boundaries, but also to a group of catchments with similar properties with respect to the analysed phenomena. These methods were consecutively tested in conditions of Slovakia for practical applicability and compared with the traditional approaches. Detailed summaries of the recent results of these efforts were published e.g. in Čunderlík (1999), Kohnová and Szolgay (1999, 2000, 2002), Solín (2002).

To pool catchments into pooling groups following physiographic and climatic catchment characteristic were derived from digitalised maps and DEM using GIS: the catchment area, the gauge datum, the length of the river network, the mean catchment slope, the slope of the mean stream, the catchment shape coefficient, the mean aspect of the, the mean catchment elevation, the percentage of forested area, the long-term mean annual runoff, the an index of infiltration capacity of the soils, the time of concentration according to Kirpich, Hradek and Nash, the areal averages of the maximum daily precipitation amounts from the period 1901-1980, the maximum daily precipitation amounts with return periods of 2, 50 and 100 years, the snow water equivalent with the return period of 50 years and the snow cover high with the return period of 10 years.

About 70 different combinations of these characteristics were tested for pooling the catchments into homogenous pooling groups separately for summer floods, winter and annual floods. Only combinations with little correlation between the selected catchment characteristics were used. No unique combination of acceptable characteristics was found.

For estimation of design discharges the index-flood method was applied in homogenous pooling groups. To select the appropriate regional distribution function, the goodness of fit test (Hosking and Wallis (1997)) and L-moment ratio diagrams for all pooling groups were used. As the index-flood - the mean maximum flood (Q_{pr}) was accepted.

A stepwise multiple regression was used to determine the relationship between the climatic and physiographic basin characteristics and the index flood values in the pooling groups. In order to minimize the effect of multicollinearity, attention was paid to the choice of predictors with a low mutual dependence. The values of the multiple correlation coefficients ranged between 0.6 and 0.95 for the resulting combinations of predictors.

5. Comparison of 100- year design maximum discharges derived using various approaches in catchments Hájovňa Slače-Vyčomá and Myjava-Myjava

In following example the variability in design flood estimation in Slovakia using various methods was demonstrated. After catastrophic floods in the year 1999 in catchments Hájovňa Slače-Vyčomá and Myjava-Myjava values of 100-year design flood estimation were estimated in these catchments using various approaches. The results are presented in Fig. 1 and 2.

It can be seen that the traditional methods tend to overestimate the statistical values in most cases. From the practical point of view, when compared with the statistically based design values, with the use of these methods safe design criterion in is met ungauged catchments in almost each case (without taking into account the uncertainties associated with the statistical values). The degree of safety of such a design is, however, arbitrarily defined, and it changes from site to site and method to method. Moreover, the use of such methods makes return period based risk and economic analysis virtually impossible, since the “actual return period” remains unknown.

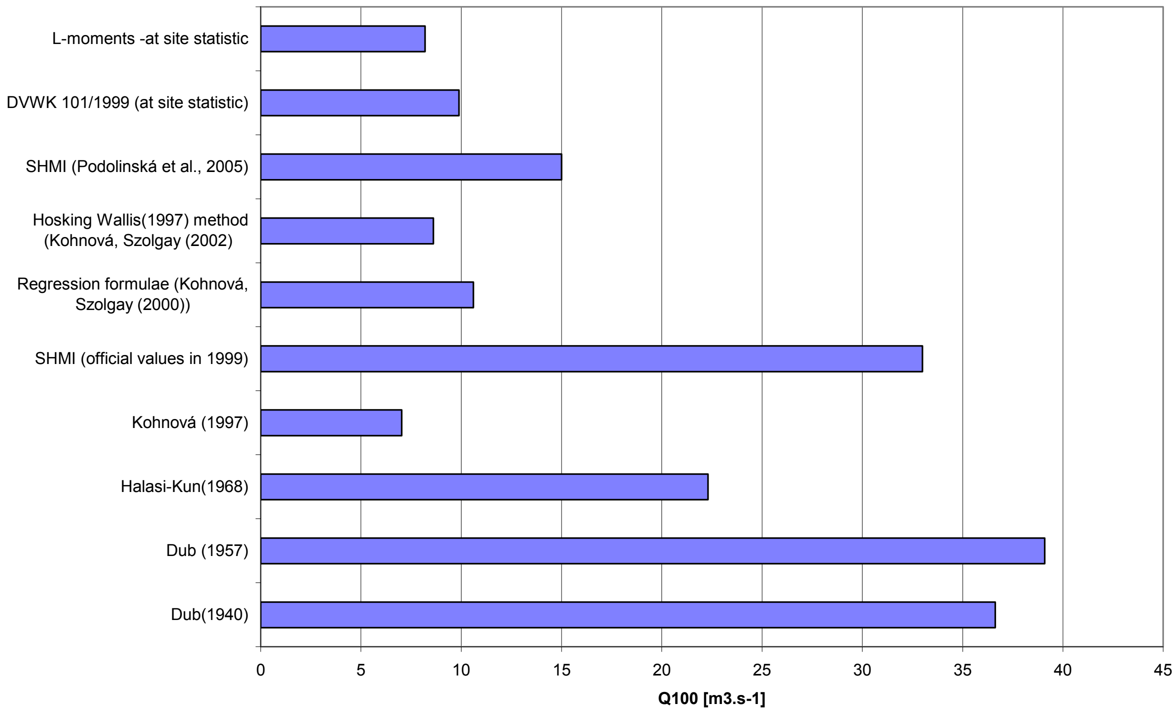

Figure 1 100-year design floods estimated in Hájovňa Slače -Vyčomá using various methods

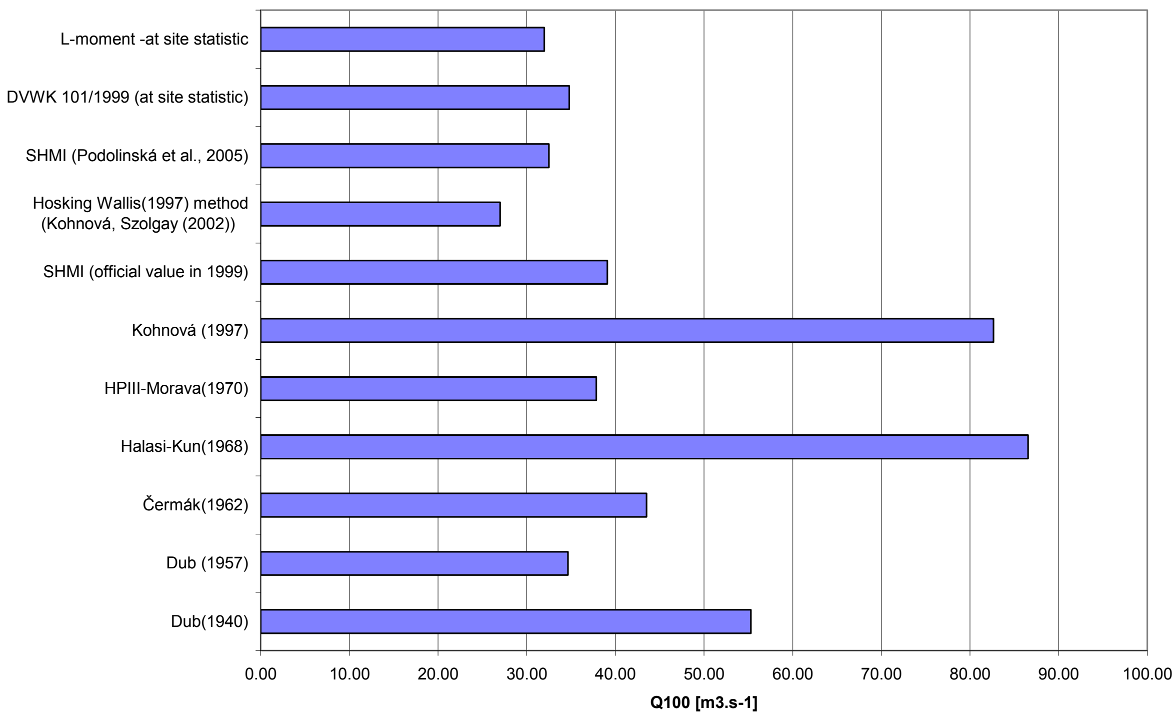

Figure 2 100-year design floods estimated in Myjava-Myjava using various methods

6. Conclusions and summary

Concerning the practical applicability of new methods base on L-moments and of the traditional methods in engineering design in Slovakia, the following conclusion were made: the traditional methods are not generally applicable to ecological river training and river restoration design, since they tend to overestimate the design discharges in almost all the basins. For these tasks, the modern regional flood frequency methods tested in this study, which do not include the regional safety principle (the envelope curve concept), but allow for site specific over and under-estimation resulting from the regional average, seem to be appropriate. Concerning the choice of the method, those with the lowest variability over the regional reference values should be preferred in practical applications. The degree of design safety associated with these (and similar) methods could be acceptable, if some measure of over and underestimation is comparable with the uncertainty associated with statistical methods. For flood protection and hydraulic engineering design where a high degree of safety is required, the traditional methods should be preferred used, since under-design cannot be compensated for by over-design.

Several questions remain to be investigated in the future, such as the preference for a particular pooling approach under specific physiographic conditions, quantification of statistical uncertainty under which the concept of regional homogeneity can be used for design purposes, etc. Inclusion of other variables controlling the variability of flood formation, and describing the intensity of overland flow formation, upper layer permeability, catchment storage, etc., should be further investigated in order to test the practical applicability of the concept of regional homogeneity based on physiographical characteristics.

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Geothermal Waters in Recreational Facilities of Slovakia

Jozef Kriš

Abstract: Geothermal resources represent that part of geothermal energy of solid, liquid and gaseous phases of the Earth's crust, which can be economically exploited from the Earth's surface and used by available technologies for purposes of power engineering, industry, agriculture, recreation and rehabilitation. The source of this energy is a residual heat of the Earth as well as heat releasing in radioactive decay of rocks and during the movement of lithospheric plates accompanied by volcanic activity and earthquakes. From this point of view, the geothermal energy is considered as renewable resource of energy. It mostly occurs in a form of hydro-geothermal resources such as geothermal water and steam. In addition, the heat of dry rocks is also included in to the geothermal energy resources. All of these energy resources are classified among the non-traditional alternative sources of energy.

Heat from the inside of the Earth is transferred to the surface and permanently emitted into the space. The mean value of surface heat flux density is q = 70mW/m^2. Concerning that the Earth's surface area is 5,1 * 1014 m^2, heat losses represent about 42 * 406 MWt.

Keywords: Geothermal water, recreational facilities, technical problems

1. Introduction

The use of mineral, but especially geothermal waters for recreational and sport purposes is now based also on scientific knowledge. The construction of bathing facilities is inevitable with regard to worsening natural conditions for bathing, swimming and recreation near water. It is important to fully utilize natural conditions for the accumulation of geothermal waters on the Slovak territory and use their higher natural temperature and evident healing effects on human organism for the construction of swimming pools at locations of such reservoirs. Another indisputable advantage is a possibility to extent bathing seasons of summer swimming pools together with more effective return on investments.

2. Present state of geothermal water use

At the present time there are 172 public swimming pools with the total number of 404 pools in Slovakia, including 146 pools with thermal water and 258 without thermal water. According to valid hygienic criteria, thermal swimming pool is defined as swimming pool comprising at least one pool filled by geothermal mineral water of portion higher than 50 %, regardless of whether there is successive circulation of water or not. Eventually the portion of geothermal water might be lower than 50 % in a case of exceeding the mineralization limit, i.e. 5000 $mg.l^{-1}$. Mineral or thermal water can be used in public swimming pools only if water temperature does not exceed temperature of 28 °C in swimming pools (recreational swimming), 25 °C in sport swimming pools and 37 °C in relax pools (for sitting). The total mineralization is allowed to limit of 5000 $mg.l^{-1}$ and strictly without H_2S content.

The content of CO_2 is determined by the maximum value of 500 $mg.l^{-1}$.

The main criterion for assessment of existing and proposed thermal swimming pools is an extension of bathing season length or their year-round availability from 280 to 360 days, respectively. Exploitation of geothermal energy is a cost demanding process and that is why the cost-effectiveness plays significant role in this issue. Suitability criteria for utilization of such resources for recreational purposes and tourism are as follows:

- Water temperature; depends on swimming pool type
- Yield of geothermal resource; they are suitable with regard to construction of swimming pools, for particular attendance categories, allow required water replacement in pools and assure hygiene of bathing
- Content of mineral substances dissolved in water; they have healing effects. On the contrary, they give rise to incrustation and corrosion of equipment and have adverse effect on environment.
- Area location; to existing or planned settlements, transport options, attractiveness, etc.

Based on mentioned facts, the following resources of geothermal energy intended for purposes of recreational swimming pools are economically profitable (Franko):

- with minimum water temperature of 35–40 °C
- with minimum yield of well over 10 $l.s^{-1}$
- specific capacity of well by uncontrolled spillway from wells over 0,1 $l.s^{-1}$
- maximum mineralization to 10 $g.l^{-1}$ and with appropriate composition of salts and gases
- depth of well ranged from 3000 to 4000 m
- geothermal gradient with value more than 30 $°C.km^{-1}$
- heat flux over $60mW.m^2$

Present knowledge on construction and operation of thermal swimming pools indicate significant capacities in a comprehensive use of energy from geothermal water. Despite positive results, unsolved problems related to construction, comprehensive use of geothermal energy, exploitation, transport, treatment and disposal of geothermal water still remain. The non-uniform approach to solution of such problems prevails in practice and usually leads to unsatisfactory results. The major reasons of this state can be summarized as follows:

- appropriate localization of well with regard to yield of well, temperature, mineralization, receiving body (suitable method for geothermal water disposal), capacities for construction of recreational area including operational-technical facilities for catering, regeneration and year-round accommodation, etc. Nowadays, the complex large bathing areas so-called “AQUAPARKS” are built all over the world. They comprises systems of pools for sitting (35– 37 °C), recreation (24–28°C), swimming(24-26 °C) and children’s pools (28–30 °C) with waves, as well as pools with toboggans, slides and other attractions such as waterfalls, fountains, etc.,
- timely urban and water management conception based on capacity and real possibilities of region
- using the thermal water for heating of year-round operations (heat exchangers, heat pumps and recuperators – regenerative air heaters), improvement of calorific balance by covering of swimming pools out of recreational seasons,
- application of solar collectors near cold thermal water, etc.

Table 1 List of swimming pools

<table>
<tr><th colspan="2" rowspan="4">Region</th><th rowspan="4">Number of public swimming pools</th><th colspan="7">number of pools</th></tr>
<tr><th colspan="7">of which :</th></tr>
<tr><th rowspan="2">thermal</th><th rowspan="2">Non-thermal</th><th rowspan="2">total</th><th colspan="2">In operation</th><th colspan="2">Out of operation</th></tr>
<tr><th>Public swimming pools</th><th>pools</th><th>Public swimming pools</th><th>pools</th></tr>
<tr><td>1</td><td>Bratislava</td><td>13</td><td>0</td><td>30</td><td>30</td><td>9</td><td>23</td><td>4</td><td>7</td></tr>
<tr><td>2</td><td>Trnava</td><td>23</td><td>33</td><td>32</td><td>65</td><td>22</td><td>64</td><td>1</td><td>1</td></tr>
<tr><td>3</td><td>Trenčín</td><td>23</td><td>13</td><td>30</td><td>43</td><td>17</td><td>33</td><td>6</td><td>10</td></tr>
<tr><td>4</td><td>Nitra</td><td>25</td><td>39</td><td>34</td><td>73</td><td>22</td><td>61</td><td>3</td><td>12</td></tr>
<tr><td>5</td><td>Žilina</td><td>19</td><td>33</td><td>19</td><td>52</td><td>18</td><td>50</td><td>1</td><td>2</td></tr>
<tr><td>6</td><td>Banská Bystrica</td><td>29</td><td>27</td><td>47</td><td>74</td><td>24</td><td>69</td><td>5</td><td>5</td></tr>
<tr><td>7</td><td>Prešov</td><td>18</td><td>15</td><td>32</td><td>47</td><td>15</td><td>37</td><td>3</td><td>10</td></tr>
<tr><td>8</td><td>Košice</td><td>28</td><td>0</td><td>68</td><td>68</td><td>25</td><td>58</td><td>3</td><td>10</td></tr>
<tr><td></td><td>Slovakia total</td><td>178</td><td>160</td><td>292</td><td>452</td><td>152</td><td>395</td><td>26</td><td>57</td></tr>
</table>

Number of samples	Number of examined parameters			
	total	within ŠZD	at provider's expense	exceeding max. values
67	964	583	381	30
195	2 684	609	1 793	230
122	1 832	982	850	105
670	10 510	4 230	2 445	902
239	3 583	1 617	2 088	196
269	3 484	1 045	2 439	241
169	2 683	1 388	627	251
172	2 708	1 290	1 518	201
1 903	**28 448**	**11 744**	**12 141**	**2 156**

Thermal swimming pools have been classified into the three categories with regard to water temperature, yield of well, content of mineral substances and location.

Following the studies of the Geodetic Institute of Ľudovít Štúr and other surveys, further recreational localities with prospect for use of geothermal water are considered: Bratislava, Piešťany-Trenčín, Malá Fatra, Low Tatras, High Tatras, Slovenký Raj, Štiavnica-Kremnica, Orava, Turčiansky Region, Vihorlat, Danube Region, Žilina, Poľana, Košice, Spiš, Prešov, Upper Nitra, Senica, Levice, South Slovakia and other.

List of swimming pools using geothermal waters for recreational purposes based on data from public health institutes is shown in the table 1.

3. Water management - technical problems related to use of geothermal water for recreational purposes.

Exploitation of geothermal water gives rise to the three major water management – technical problems.

- Incrustation in wells and distribution systems of used equipment (profile fouling and decrease in operational thermal capacity). Release of CO_2 disturbs the carbonate balance what results in $CaCO_3$ separation from a solution. The presence of air oxygen gives rise to oxidation of some compounds such as Fe^{2+} to Fe^{3+} and consequential separation from water. Oxidation of other compounds may result from using a disinfection preparation in pool thermal waters.
- Corrosion depends mostly on consistence of geothermal water, presence of O_2, CO_2, H_2S, Cl_2, total content of salt, etc. Corrosion occurs rapidly in an environment of various types of the geothermal water and under the different thermodynamic conditions. It is necessary to monitor corrosion because it is important factor that affects a service life of particular devices required for geothermal water use.

- Impact of geothermal water on environment has important role in its heat utilization. From known methods of geothermal water disposal we can use following:
  – discharge into surface stream (receiving body)
  – dilution and discharge into receiving body
  – partial demineralization and discharge into receiving body or use for irrigation
  – discharge into public sewerage and subsequent treatment
  – reinjection

Discharge of used geothermal water into the receiving body is regulated pursuant to the Act No. 184/2002 Coll. on waters and the Regulation No. 491/2002 Coll. of the Government of the Slovak Republic.

Considering the river system of Slovakia as well as water bearing of streams and asymmetric location of geothermal resources, the mentioned methods are very expensive and cost-ineffective.

The reinjection appears to be the most appropriate method for geothermal water disposal, although it is very expensive procedure. The production well is used for abstraction of geothermal water, which is cooled after heat withdrawal to a temperature of 30–40 °C and then returned through the reinjection well back to the aquifer.

A production well requires sufficient distance from reinjection well in order to eliminate effect of reinjected water on the temperature of pumped water. The distance is usually ranged from 1000 to 1500 m. Temperature in aquifer decreases by 1–2 °C every five years what provides constant temperature production over a 20–30 year period. This method of disposal is very demanding on technical equipment and it requires good knowledge of environment where reinjection is to be carried out, such as evaluation of harmful effects on a geothermal field and its lifetime.

The used wastewater from swimming pools is still discharged into the surface streams, except for two reinjection stations (used for heating of buildings) and such discharge has adverse effect on ecology. More detailed information on conditions and options for disposal of used geothermal water for particular basins of Slovakia as well as technical properties of geothermal waters are listed in the Atlas of Geothermal Waters in Slovakia (1995).

4. Conclusion

Regarding its geothermal energy resources, Slovakia is one of the perspective European regions. Effective use of this renewable energy resource might have economic significance for the Slovak Republic, considering traditional energy sources conservation and opportunity to enhance tourism and recreational capacities in more Slovak regions. Construction of another bathing areas/swimming pools with geothermal water and reconstruction of existing ones may lead to increased attractiveness of Slovakia as well as interest of foreign investors in this field of business. In many cases, the reinjection of abundant geothermal waters into the soil horizons seems to be the most efficient method (ecological point of view). However, it requires perfect knowledge of environment and techniques in order to eliminate all adverse effects of geothermal water use on human environment, which may appear also after several years.

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Analysis of One of the Rare Natural Retention in the Middle Danube

Siniša Maričić

Abstract: In the middle course of the Danube river there is one of the rare, but still large natural areas, where the large waters of this river can pour out into the wider inundation area. As a consequence, along about fifty kilometers of the courseupstream around cca 1370 rkm of the Danube, transformation of large water waves hydrogram is occured.
In the paper the measurements from observation of the Danube and Drava water regime are analyzed in the zone of their connecting. The available data are used to make the hydrogram approximation of some water waves and the retention effect of the existing natural conditions is examined. The example of the possible quantification of retention characteristics is given.
Although for the analyses the uncoordinated time data were used, the results can be still considered as respectable. For the quality observation it is necessary to extend the existing monitoring and to work on the uniting of the measurements and data base collected in this area.
Keywords: Danube, Kopački rit, water regime, retention effect, transformation of hydrogram

In general

The preservation of the biological variety in the Republic of Croatia is pointed out as a government goal. It is founded on The Nature Protection Law, which as a main method has the protection of the particular areas and species. National park and Park of Nature are the highest protection categories and they are proclaimed as the areas of national or world value. [1]

The research of the formation and characteristics of these areas, among which the hydrological-hydraulically elements have important place, enables the foundation of their values protection. This paper is inspired by such motives.

The description and problems of the area

In the middle course of the Danube river there is one of the rare, but still large natural areas, where the large waters of this river can pour out into the wider inundation area. This is the area of co-cource and morphogenesis of the rivers Danube and Drava, in size of cca 50 000 ha (Figure 1).The particularly emphasized is the area of mostly right bank of the Danube from r.km 1383 to r.km 1410 and the left bank of the Drava, which is included by the borders of the Park of Nature Kopački rit (cca 23.000 ha). The values preservation of Kopački rit is of world importance – it was nominated by UNESCO for the World Inheritance List and according to the Ramsarsk convention it is inserted on the list of the world protected areas. It is also inserted on the IBA list (Important Bird Area). [2, 3]

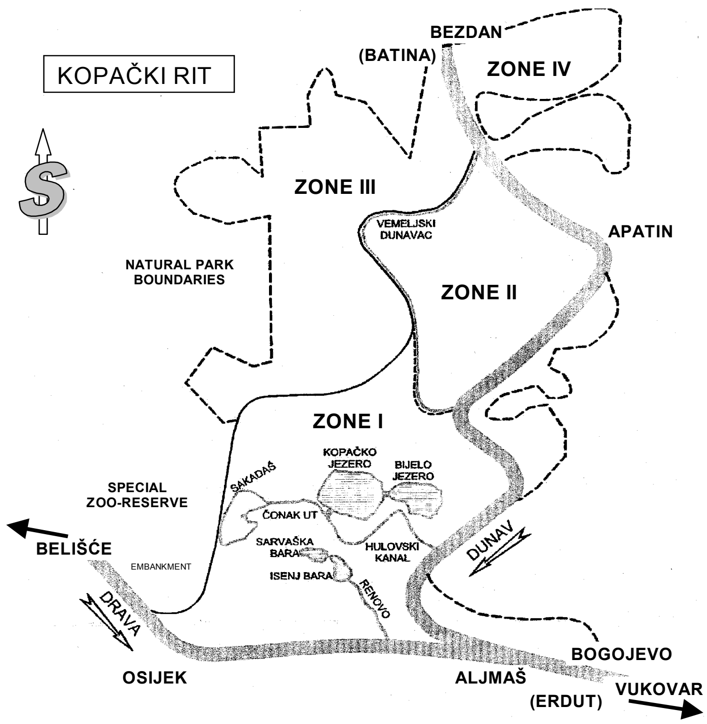


Figure 1 Sketch of the Kopački rit area with an indicator of main guidelines

The characteristic of this area is that on some lower parts, due to impossibility of surface outflow, water stays even after the passing of the water wave. Therefore some swamp charatceristics, lake conditions and the woods of alluvial plains are present here. The presence of water, underground and ground inflows extend preferential treatment to life of numerous plant and animal species. The present biodiversities and ecosystems survive in here described conditions of slow agricultural development, which is, it can be said, adequate to the formulation of so called maintained development. Different interests are here intermixed, some connected to forestry, hunting, agriculture, tourism, roads and water ways in economy sense and some connected to protection of the nature values in ecological sense. [4]

Due to andopogenic influences (dikes against flood, watercource regulation, land-improvement works and so on) the natural regime of inundation has been changing. The reasons are in the wider area – formed in the part of the watercourse through Hungary and inflow areas in Serbia and Croatia. Narrowly seen, the dike against floods Kopačevo-Zmajevac is pointed out, which separates the natural inundation from the nearby agricultural areas of Baranja. [5, 6]

During the Hubsburg management, on the land margin of Kopački rit, the so called Albert's dike was built. It was broken during the flood in 1926, and then even greater depression was formed – todays Sakadaš lake. Between 1966 and 1971 on the same location the new protection dike was built which was reconstructed from 1982-1985 and today it is one of the important references of hydrological processes in Kopački rit. Along the dikes there are peripheral canals and out of inundation zone the net of land-improvement canals were formed. To transport their waters the building of the pumping stations started at the end of the 19th century. Three pumping stations (Podunavlje, Tikveš, Zlatna greda), total capacity 12,75 m^3/s, transport the waters directly to the area of Kopački rit. During the building of the protection dike the dam Kopačevo was also built. It is situated on the dike between canals made of the former parts of the Drava bed (Stara Drava) and Sakadaš lake inside Kopački rit.

In 1973, at the end of the Hulovo canal to the Danube, the sand sill was built, which was broken by the first greater waters. After that the stone sill was built, which is, although constantly repaired, still today in function.There were also some other sill buildings but they were also destroyed by large waters. [7, 8]

At the end of the last century, during the war, objects of water resource management were damaged, the hunting was out of control and the woods were cut down.

Ecological system of Kopački rit has been surviving both dry and wet years. Each of them has been bringing both desirable and undesirable changes of biocenosis and reliefs. If there was not the cyclic exchange of dry and wet periodes, the biogical structure of Kopački rit would be quite different today.

Water regime in Kopački rit, although very important ecological element, is not enough studied. [8]

Hydrology references of the area

Many interests connected to these two large rivers on the mentioned area have as a result the monitoring establishment. (Tbl. I) The available data in this region mostly are from the middle of the 20th century till today, so in times of the conditions changing due to the climate change and significant human activities in gravitating river-basin.

The river Danube is the most impressive element of this area. Its width here is about 250-880 m, the depth of the bed, when the water is in the middle, is 3-6 m and the range of water level oscillation is over 8 m. The average fall of water face for the belonging part is about 5,5 cm/km. [9]

Table I Basic data about hydrological stations of wider Kopački rit area

	STATION (* - out of Croatia)	F (km^2)	"O" (m asl)	POSITION (r. km.)	FOUNDED IN	EQUIPMENT
Drava	D. MOHOLJAC	37 142	88,57	80,6	1890.	limnigraph
	BELIŠĆE	38 500	83,99	53,8	1961.	limnigraph
	OSIJEK	39 982	81,48	19,1	1827.	limnigraph
Danube	MOHACS *(H)	209 064	79,20	1 446,9	1852.	limnigraph
	BEZDAN *(SCG)	210 250	80,64	1 425,5	1856.	limnigraph
	BATINA	210 250	80,45	1 424,8	2000.	limnigraph
	APATIN *(SCG)	211 139	78,84	1 401,4	1876.	lath
	ALJMAŠ	251 513	78,08	1 380,5	1909.	lath
	BOGOJEVO *(SCG)	251 593	77,46	1 367,4	1871.	limnigraph
	VUKOVAR	253 147	76,19	1 333,4	1856.	limnigraph

The river Drava has the shorter course till the junction with the Danube so the water wave made by snow and ice melting in the Alps, where both rivers well, comes earlier to the area of the mouth of the river. The above mentioned, along with one sharp curve (on the Danube near Dalj) slows the water levels by the mouth and influences the arrival of the water in the area of Kopački rit. [9, 10]

Along the main bed of the Danube, especially in the upstream part on the right side, there are the numerous remains of meandar formations.

Among them the most important is small backwater channel Vemeljski Dunavac. It is the abandoned bed, whose entrance is directly connected to the today's bed of the Danube on r.km. 1407, and exit on r.km. 1392.

The heights above sea-level in the area of Kopački rit range from 80-86 m above sea-level. The area is gently inclined toward the south-east.

The heights below 80 m above sea-level are present near the bigger lakes and these are (the biggest) Kopačko lake, Sakadaš and Bijelo lake and near the deepest canals Čonakut, Gorba and Hulovski canal, therefore they have the water all the time.The banks of the Drava and the Danube are higher than the ground in the Rit itself, therefore the change of the surface waters happens first in the bigger canals (Hulovski, Nađhat, Renovski), and after that, but significantly more extensive, in numerous, overgrown, almost invisible canals, old small backwater channels and depressions. It is very complicated process which contains the underwater flowing, which is very difficult to describe. [9, 10, 11]

The researches so far gave the following references:

- according to the hydrographical elements and different water regime there are four zones marked in the figure 1 [9];
- the basic water exchange happens in the following way: in the Danube direction through Vemeljski Dunavac and Hulovski canal, in the direction of the Stara Drava bed over the dam Kopačevo, in the Drava direction through Renovski canal and from the peripheral and land-improvement canals by means of the pumping stations;
- the surface flowing in the Hulovski canal is limited by the sill with the crown elevation in the height of 79,5 m above sea-level;

- the greater outflow from the depression areas is characterized by the height of 81,5 m above sea-level, and inflow in the wider zone in the height of 82,0 and 82,5 m above sea-level;
- when the water level is 83,0 m above sea-level the larger part of Kopački rit is overflown and outflow, starting in Renovski canal, extends on the whole contact area with the Drava.

Analysis of incoming-outgoing discharges in the Kopački rit area

The observed area is very well covered by the nearby measuring stations but available data ranges have some limitations. Basic data about measuring stations are given in table I, and table II shows the values of characteristic water levels and discharges measured there.

Great effort is put into the organisation of water level monitoring in Kopački rit itself.

According to the data gathered till nineties the decrease of all water levels was observed. In general, the greatest decrease was observed by medium water levels (for the Danube about 1 cm a year, and for the Drava even more) and the least by minimum annual water levels. The decrease of discharges of these rivers was also observed. The decrease trends of medium and minimum annual discharges were established, for the Danube about 5 m^3/s and for the Drava about 0,5 m^3/s a year. The cause for water level decrease was, apart from the discharges decrease, the riverbeds deepening. [8]

The recent analysis of the water levels, entrance locations, locations in Kopački rit and downstream locations shows some characteristics of this area. [12]

Table II Characteristic water levels of observed hydrological measurements

	HYDROLOGICAL STATION	* WATER LEVELS (m asl)		RANGE (m)	MEAN FLOW ** (m^3/s)
		MIN	MAX		
DRAVA	BELIŠĆE	84,23	90,26	6,03	556
		84,33	88,18	3,85	-
	OSIJEK	79,82	86,93	7,11	-
		80,03	85,88	5,85	-
DANUBE	BEZDAN (BATINA)	80,14	88,40	8,26	2303
		80,80	87,76	6,96	-
	APATIN	78,70	87,08	8,38	-
	KOPAČKO LAKE	79,83	86,18	6,35	-
	ALJMAŠ	77,40	86,44	9,04	-
	BOGOJEVO (ERDUT)	77,21	85,61	8,40	2859
	VUKOVAR	75,27	83,87	8,60	-
		76,69	82,63	5,95	-

** - last 50 years. and two-year monitoring ** - thirty-year course (1961.-1990.)*

The logical continuation of the water regime analysis in this area is connected to the discharges. The attached tables show that the flow through (the flow speed per profile and the bed shape) is measured upstream at stations Bezdan (Batina) and Mohach on the Danube and

Donji Miholjac and Belišće on the Drava. At the exit, there are some measurements for the location Bogojevo (Erdut). The data are gathered by three countries which makes their uniting difficult. War conditions and poorer economic circumstances affected the measurement amounts and work interruptions. Nevertheless, it was possible for the specialized analysis to collect and use some new and coherent data. [13, 14] They are the following:

- Daily water levels (situation at 7 a. m.) for the locations Bezdan, Belišće and Bogojevo in the period from 2001 till 2003;
- Daily water levels and discharges for the locations Bezdan and Bogojevo for the years 1999, 2000 and 2001;
- The discharge curve for Belišće for the years 1993 and 2003.

According to the above mentioned the following was reconstructed and adopted:

- The representative discharge curves (done according to the available three years data), with water levels in meters,
  - for Bezdan:

$$H < 5{,}0: \qquad Q = 37{,}506 \cdot H^2 + 441{,}09 \cdot H + 1116{,}1$$

$$H > 5{,}0: \qquad Q = 251 \cdot H^2 - 1697{,}4 \cdot H + 6503{,}2 \quad ;$$

  - for Bogojevo:

$$H < 5{,}0: \qquad Q = 53{,}914 \cdot H^2 + 451{,}07 \cdot H + 1070{,}6$$

$$H > 5{,}0: \qquad Q = 103{,}27 \cdot H^2 - 6{,}791 \cdot H + 2129 \quad ;$$

- The representative discharge curve for Belišće (reconstructed according to the existing data (1993, 2003) for about year 2001:

$$H \le 2{,}7: \qquad Q = 27{,}77 \cdot H^2 + 134 \cdot H + 153{,}23$$

$$H \ge 2{,}7: \qquad Q = 60{,}06 \cdot H^2 - 51{,}44 \cdot H + 416{,}69 \; .$$

The received discharges for the observed period (the measuring was done in Kopački rit and the water level analysis was carried out, lit.) are shown in the figure 2. The incoming discharges (from the location Belišće / the Drava and Bezdan / the Danube) are added in one hydrogram so that they can be compared to the outgoing discharges (Bogojevo / the Danube). It is a question of the roughly equal distances and flowing conditions, which justifies the procedure.

The discharge run as well as water level run shows the appearance of floodwaters and dry periods. During the observed period the water overflowed seven times in greater amounts in the area of Rit, which is shown by the water levels in Kopački Rit higher than elevation 82,5 m above sea-level. The whole area with the water level elevations above 84,0 m above sea-level experienced flood three times. Small water period was recorded three times when the levels were below 81,00 m above sea-level. [12]

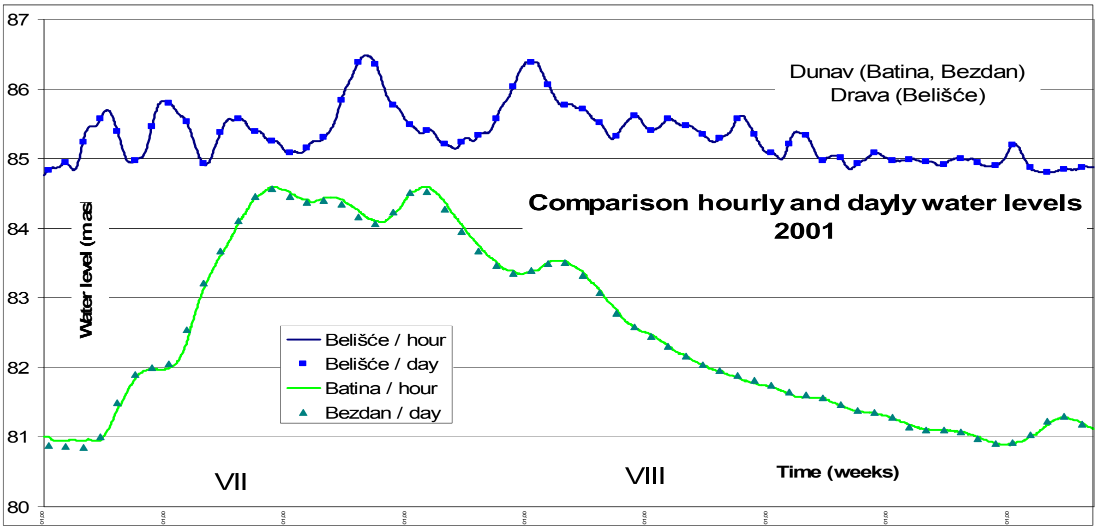
Since the water levels were registrated every full hour and discharges analysis was based on daily water level data (at 7 a. m., because there are data on some for them interesting locations), the interacting comparison was done. It was shown that the daily data completely concretely describe the observed regime, which is illustrated by the figure 2.

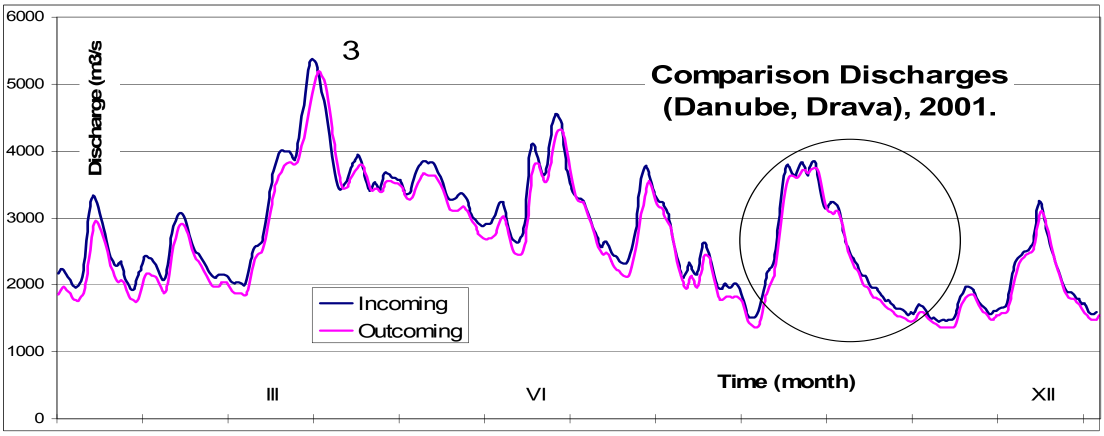
The constantly larger amount of incoming water is noticed, larger than outgoing according to the observed model. It is a question of lower row of quality, and the reasons can be different. Firstly, the quality of discharge curve adopted for the location Belišće / the Drava is debatable. The measuring location was without measuring for the longer period of time due to war circumstances. Therefore the adopted Q-H relation was based on a few new measurings. Besides, the location is under the influence of the Danube slow down, and one can not be sure how it influences the data (discharges). In addition, due to the ignorance, the incoming /outgoing influence of the hydrotechnical system Danube-Tisa-Danube was not examined. Also, underground flowing and evapotranspiration for the observed areas are mostly unknown. The constant deficit is surely interesting, it raises some questions and suggests the need for additional monitoring.

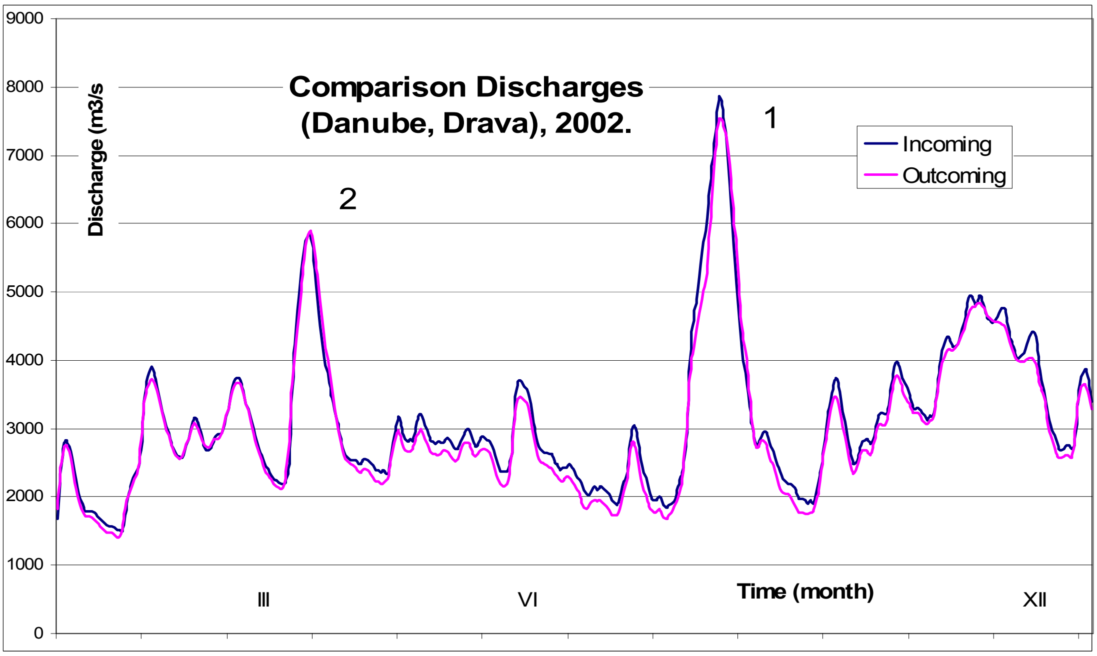
The biggest water waves with discharges above 5000 m^3/s were particularly observed. Because of the comparison and the graduation of retention effects the joint incoming hydrogram was moved in time (for the whole day, which suits the water travelling) in relation to outgoing hydrogram. In that way it was possible to notice the transformation of the water wave, considering the filling and empting of the Kopački rit area. The observed waves are given in the figure 4. They appear at the beginning of the spring and at the end of summer. They last twenty and more days.

On the included greater water waves (which were not serious flood problem) some differences were noticed in dynamics and exchange intensity of upgoing and downgoing part of the hydrogram, which means in filling and empting of inundation area. The possible explanation is that the differences are the result of the present circumstances, which precede the water wave arrival. It is a question of ground saturation by water of inner and outer origin, development phase of vegetation caused by general weather conditions, some new morphological changes in the ground and so on.

According to the coordinated hydrograms the amount of filling and empting of the inundation area was estimated. It is a question of hundreds of millions m^3 and the biggest reference was estimated on 685 mil m^3. Noticeable retention effects are delaying of the hydrogram top for about a day or more and discharge decrease for about 5% (it is possible there isn't any).







Figures 2 and 3 Relevant comparisons of water levels and discharges

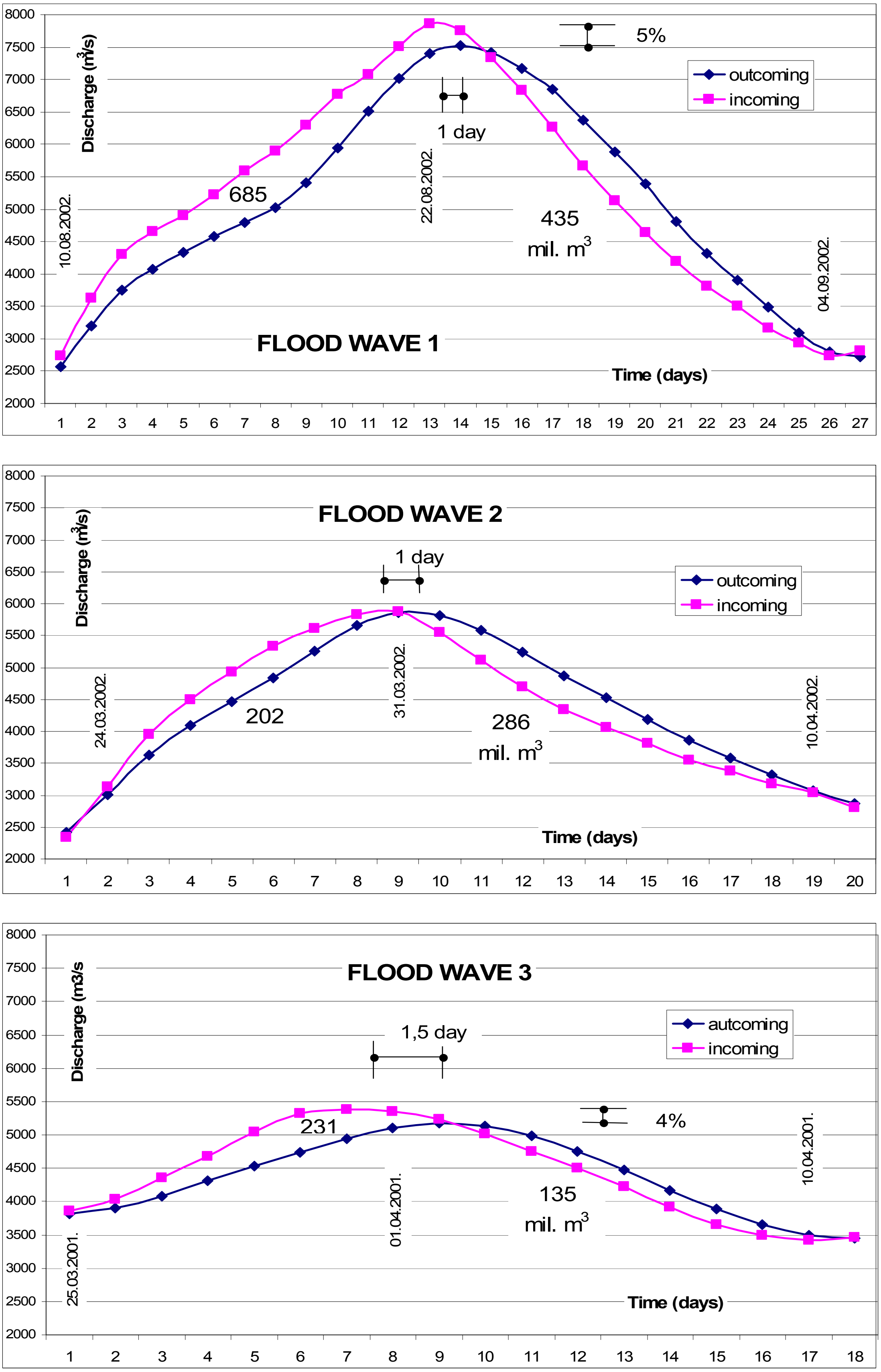


Figure 4 The biggest water waves (> 5 000 m^3/s) in period 2001 - 2003

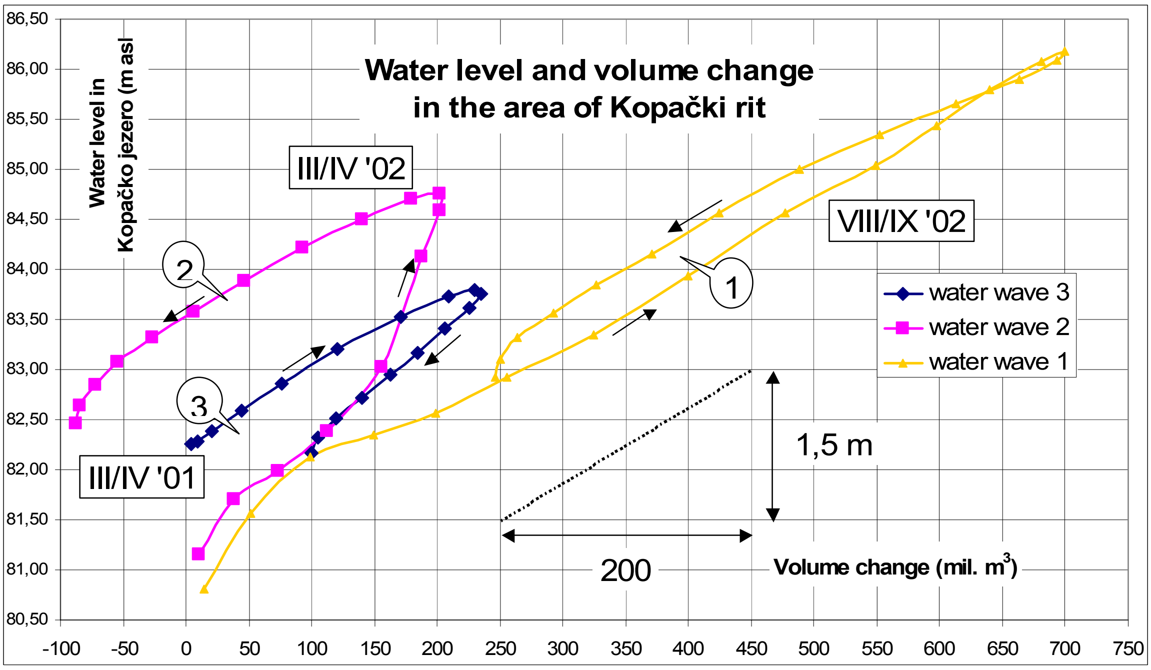


Figure 5 Analyses of inflows and outflows in observed area

Further, the analysis of the volume and water level change in the area of Kopački rit was done. The relation is given on figure 5 and shows some filling and empting regularity. Used water level measuring data on the location in Kopački rit were adequate to the time (spreading out) of the water wave appearance and the volume change was received from the difference between incoming and outgoing daily discharges hydrograms coordinated in time. The dependence, which comes out of upgoing and downgoing part of the wave 1, downgoing part of the wave 2 and upgoing part of the wave 3 can be seen in the figure 5. According to it the water level change of 1,5 m changes the volume in Kopački rit for about 200 mil. m^3. That change can also be different which is shown by the others analysed part: upgoing part of the wave 2 suggests that another inflows (underground, gravitating waters, falls) can be even about 3/4 of the total water income contribution to this area; and downgoing part of the wave 3 suggests that about 1/3 of the water volume can be drained by underground courses.

Conclusion

The correlation of the data from different sources and with some specific imperfections into one complete model of inflows and outflows is the contribution to the understanding of the water regime of Kopački rit area and its problems. From the gathered data some approximations of incoming-outgoing hydrograms were made and the general surplus of the incoming compared to outgoing values of the flowing through the main riverbeds was noticed. Everything that has been done can and should stimulate the further efforts to meet higher standards in comprehension of the present conditions. The established monitoring in this area should be improved and precisely carried out. At the same time it is very important to insure both better promptness and accessibility of the gathered data.

On the greater water waves the observed retention effect of the existing, mostly natural, conditions showed some variations. Quantification efforts have their limits because it is a question of few and different water waves. On three water waves with discharges bigger than 5 000 m^3/s there is constant decrease of the discharges (maximum about 5 %) and additional withholding of the maximum wave for about 1 day and more. So, it is not a question of significant retention effects.

The established dependence between water levels in Kopački rit and volume changes in the whole inundation area shows important moments of the water regime of this area. In most cases, the water level change of 1,5 m suits the volume change of 200 mil. m3. The significant influence of the underground flowing was established, whose part in total changes of water regime was estimated on about 1/4 in outflowing and 3/4 in inflowing.

Although for the analysis some time-uncoordinated data and some not enough measured data were used, the results can be still considered respectable and welcomed for the condition examination of this area.

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Problems of High Water Appearances in Urban Areas

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Abstract: Flood appearance in urban areas is commonly caused by the urban areas precipitation's themselves. The reason for their appearance is often inevitable e.g. extreme rainfalls, hydrological runoff conditions or the degree of drainage systems development, but sometimes the damages caused by flood appearance are the result of methodological mistakes in hydrological calculations.
Present paper analyses high water appearances in urban areas from the aspect of hydrological practice. The most frequent causes of high water appearances are presented and much more detailed evaluation of problems of high waters in urban catchment areas as the consequence of precipitation is performed. The main principles of drainage systems planning for negative effect elimination of high waters is described. The emphasis is given on the hydrological component of the mentioned problem. The actual problem of urban precipitation water drainage system planning is given on the example of Škurinjski potok catchment area in Rijeka.
Keywords: flood waters, urban drainage.

1. Introduction

High waters are integral parts of water regime of every water stream or drainage zone and of urban areas also. Here, because of their changed natural hydrological cycle and because of their consequences, high water appearances are much more pronounced. Most frequently, they are the consequence of intensive precipitation on immediate urban area or in close vicinity and that is what is analyzed in the present paper. The causes of high water appearances can be even different and can be connected with more distant parts of greater water catchments.

According to Niemczynowicz (1999) problems of high waters in urban areas are caused mostly by a great concentration of population on relatively small area. Lowered infiltration capability and underground runoff cause the change in surface runoff characterized by increase of water wave's peaks and volumes of out flowed water. Such precipitations, although not necessarily characterized as catastrophic in the sense of human lives endangering, could frequently have devastating character. Urban precipitation waters are influencing the velocity of sediment transport and the pollution and so their environmental influence is increasing. That is the case in the down-stream part of a water streams and for coastal sea zones if the sea is the immediate recipient of precipitation waters from an urban area. Considering these facts, during the 1970s the traditional approach to precipitation waters drainage was changed with a new approach focused on withholding of precipitation water, their retention and reuse. Later on, during 1980s, urban precipitation waters were considered as important cause of pollution and the main aims of the management were oriented toward protection of natural water cycle and ecosystems introducing local control measures and different biological systems for their treatment. Afterwards, the diversity of different methods for treatment and use of precipitation waters was increased and further developed. It is generally accepted that urban precipitation waters are primarily resolved locally with a combination of different procedures and treatments. Primarily these measures are focused on directing the water flow through natural or constructed ecosystems for preventing their pollution, stimulating their infiltration and their use for possible urban purposes.

This paper discusses approaches to problems of urban water drainage in the context of protection from high water appearances on the example of coastal town Rijeka (approx. 200.000 inhabitants). The paper analyzes the project solution for the drainage of Škurinjski potok, one of the rare naturally conditioned torrent streams in Rijeka. It is small karst catchment (approx. 3 km^2 of area), that enters into urban tissue of the center of the town. The downstream part is covered and has a maximal capacity of about 2 m^3s^{-1}. Examples shown in present paper point on causes of floods in urban areas and solutions, especially in the domain of ensuring the adequate managemental foundations.

2. Strategy for Implementation of Flood Control from Local Character Catchments in Urban Areas in Croatia

Strategy and measures of flood control in urban areas can be divided into two groups – constructive and nonconstructive measures. The first group includes construction of new drainage systems and regulation structures (different types of open and covered over channels), reconstruction and modification of existing channels and pipelines for the increase of their permeability, formation of inundation or retention areas (e.g. accumulations or retentions construction) and formation of systems with controllable temporary flooding of urban zones. Previously mentioned measures, especially those with channel and pipeline systems for the drainage are mostly present in Croatian practice of flood control in urban areas, especially in bigger towns. Former concept for the management of drainage systems of precipitation water was based on constructive measures. One of them is the increasing of permeability of existing covered channel collectors (once natural water streams that were for reasons of urbanization regulated or piped). Another one is the construction of new regulations or underground pipelines for the drainage of continuously increasing surfaces and consequently increasing amounts of water. The constructions of retention accumulations in higher parts of urban catchments are so far rarely present with the exception of retention objects constructed on slopes of Medvednica mountain that serve for protection of Croatian capitol Zagreb. Zagreb was in history several times flooded by waters arrived from the mountain Medvednica. Because of their efficacy the construction of such objects are planned to be continued even in those parts of country presently without them. Škurinjski potok described in present paper is one of such examples.

Nonconstructive measures so far are less frequently used in Croatia and their implementation in practice is still at the beginning. Still, the most frequently used among them are limitation of further urbanization and building in susceptible areas (through obligatory professional expertise on spatial plan documentations), determination of necessary proportions of infiltration into underground at the site of building, flood control plans designing and evacuation plans for local areas designing.

For most of these measurements (strategies) it is necessary to standardize input parameters regarding analysis solutions e.g. the requested degree of protection from flood water appearances within certain protected area. In Croatian practice so far, variable degrees of protection are used (return periods) for solving and planning of flood protection of urban areas hence in the near future will be necessary to create certain recommendations or regulations based on current applications of solutions for flood protection in urban areas.

3. Causes of Flood Appearances in Urban Areas and Possible Solutions

The roughest division of flood causes would divide them on three major factors – precipitations as natural phenomena, built urban substance that causes flow of greater water amounts in shorter time toward lower parts of towns and finally, the ignorance or negligence that could be attributed to managemental structures.

Short termed precipitations are the only objective reason of flood that can not be influenced on. Therefore it is necessary to adjust urban drainage systems as well as other urban contents to them and to consequential flows in a manner to minimalize undesirable consequences of such events. Frequently, urban drainage systems are not capable to accept greater water amounts following intensive local precipitations. The dimensioning of covered channel drainage systems is usually performed for precipitations of relatively more frequent return period (0,5 – 5 years – dependent on significance of the zone and on desired level of protection). Considering the mode of surface evacuation of part of precipitation waters –

flows of longer return period are usually not considered. Even greater problem that is often present is oversimplified approach to calculations of the drainage of urban precipitation waters is that characteristics of precipitations are often generalized mostly through only one parameter – precipitation intensity, without entering into the structure of data e.g. their interrelation with the duration and the return period (Ožanić, Rubinić, 1998.).

Therefore, in conditions of intensive precipitations streets are taking the role of uncontrolled surface collectors so in urban areas flood appearances are possible also. That is especially considering on coastal urban areas in Croatia where, in contrast to the continental part, karst structures are dominant and short termed precipitation intensities are more pronounced. Just for illustration Figure 1. represents the comparison of 2- and 100-years HTP curve (Precipitation height-Duration-Return period) for the coastal town Rijeka and the city of Zagreb located at continental part of Croatia. It can be noticed that in spite of the distance of only 180 km and practically almost the same altitude above sea level 2-years HTP curve of Rijeka and 100-years HTP curve of Zagreb are almost the same.

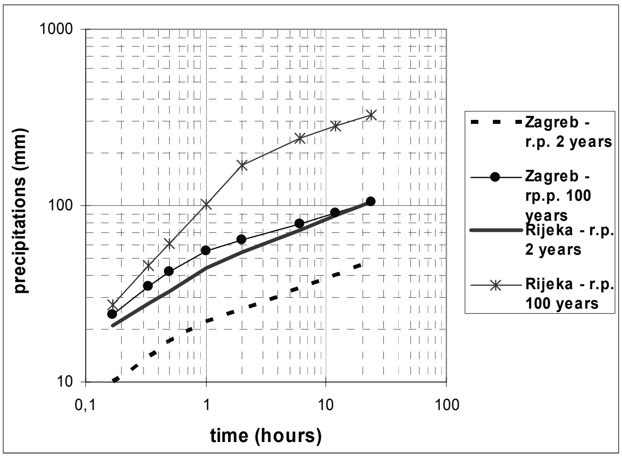

Figure 1 The comparison between HTP curves of 2-years and 100-years return period for stations Zagreb-Grič and Rijeka

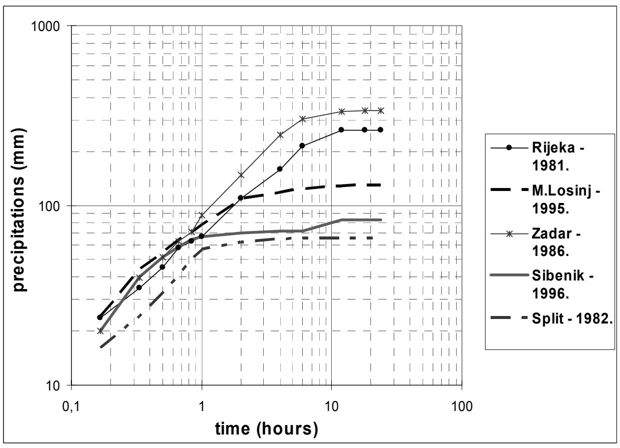

Figure 2 Comparison between maximal precipitations of analyzed storms in the coastal region of Croatia

High waters in urban areas are not hypothetical e.g. by hydrological calculations presumed values. They are real and appear from time to time. Figure 2. shows comparatively cumulative short termed precipitations for several documented cases when extremely intensive precipitations caused extreme precipitation waters in urban areas of Croatian coastal region. Namely these storms happened on August 21st , 1981 in Rijeka; October 13th , 1982 in Split; September 10th , 1986 in Zadar; October 30th, 1995 in Mali Lošinj and August 30th, 1997 in Šibenik.

Such appearances of extreme precipitation intensities in urban areas cause high waters even in localities where they do not appear usually, and are not even expected. For example, during previously mentioned extreme precipitation appearance in Zadar flood of some urban quarters happened. Unfortunately, the catastrophe was even greater since a child has drowned in floodwater flowing down the road. Another illustrative example happened on October 30th, 1995 at Veli Lošinj area on the island of Lošinj. During 2 hours, approximately 200 mm of rain precipitated. The mentioned precipitation caused a significant surface flow along a usually inactive and unexpressed gully that gravitates toward the centre of the town. It caused intensive flow along streets of the town also (Figure 3). According to analyzes performed, a catchment with an area of only 0,334 km^2 produced a water wave with maximal flow of 7,5 m^3s^{-1} (Ožanić et al., 1996.).

It is obvious that solutions of drainage of urban areas should not be partial e.g. conducted in a manner that drainage of one part should not endanger downstream parts.

Figure 3 Flooded street in Veli Lošinj (coastal part of Croatia), October 30th, 1995 photographed ½ hour after the rain

The solutions are withdrawing of water in the catchment and prolongation of flows through the urban drainage system during the storm duration. Therefore, regardless of other advantages, the construction of multipurpose objects for withdrawing or accumulation of precipitation waters in urban water systems is practically the sole solution for the insurance of appropriate drainage.

4. The Example of Flood Control System at Locality Škurinjski Potok in Rijeka

Škurinjski potok is one of the rare naturally conditioned torrents in the area of Rijeka. The catchment area of its open part (2,9 km^2) is localized at the skirt of urbanized city area (Figure 4.). The geological structure is composed of limestone that at the surface has expressed geomorphologic phenomena – funnel shaped holes, abysses and gaps in its natural waterbed and with mostly shallow soil coverage across catchment's area. Therefore the surface water flow along its natural waterbed is very rare – mostly as a consequence of extreme precipitations or intensive precipitations on previously saturated ground. The land's assignation has changed so the built urban contents and infrastructure objects mostly disturb natural flow conditions. The expansion of urban contents inside the catchment area together with coverage of downstream waterbed part disabled the increase of evacuation capacities of precipitation water that started to appear more frequently particularly on public surfaces that are for most inappropriate – the main road passing through the settlement. For further urbanization planning of the valley part of Škurinjski potok area and for appropriate solution of urban precipitation water drainage system from the entire catchment area the project was made (Faculty of Civil Engineering Rijeka, 2002.) for the open part of Škurinjski potok stream consisting of three retentions with total volumes of 26600, 22800 and 30100 m^3.

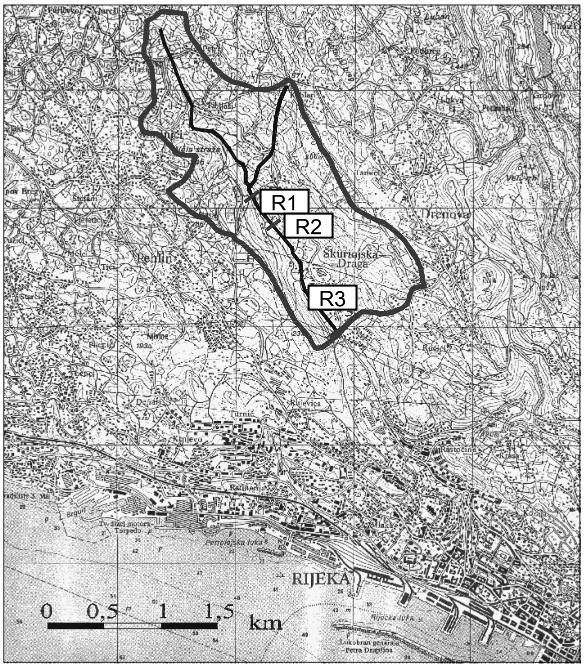

Figure 4 Catchment area of analyzed Škurinjski potok with planned retention compartments plotted

These retentions should withdraw precipitation waters produced on not built catchment surfaces and consequently induce their infiltration into underground. They would collect even the excess of surface waters that could not be evacuated through the existing channel network system for precipitation drainage. The plan proposes the use of a recreational area, park and an orchard for retention purposes. The plan considers the possibility for accumulation of a part of precipitation water in order to provide irrigation needs for the mentioned orchard. It was judged that withholding of a part of high waters in the catchment could provide proper functioning of the downstream covered part of waterbed. The proposed solution predicts also the drainage of the part of system that gravitates toward it during the critical duration of high waters. After the peak water wave a part of retained water would be directed under control.

The mentioned project analyzed several possible locations and magnitudes for retentions as well as possibilities of precipitation waters conduction from different catchment parts. Calculated maximal runoffs at the final profile are reaching 15 m^3s^{-1} and the respective water wave's volumes are reaching 150.000 m^3 (for 100-years return period). For analyzed solutions calculations of transformation of characteristic hydrograms of water waves were performed for their passage through the system of planned retentions (Figure 5.). The resulting peak runoff at the end of retention's system e.g. at the entrance to the covered part of Škurinjski potok could be limited to 4,20 m^3s^{-1}. A part of precipitation waters from the most built part of catchment would continue to evacuate through the existing collector. That would diminish the possibility of infiltration of most polluted waters to underground considering the fact that the analyzed part of Škurinjski potok catchment belongs to the II. zone of sanitary protection of water spring Zvir in Rijeka.

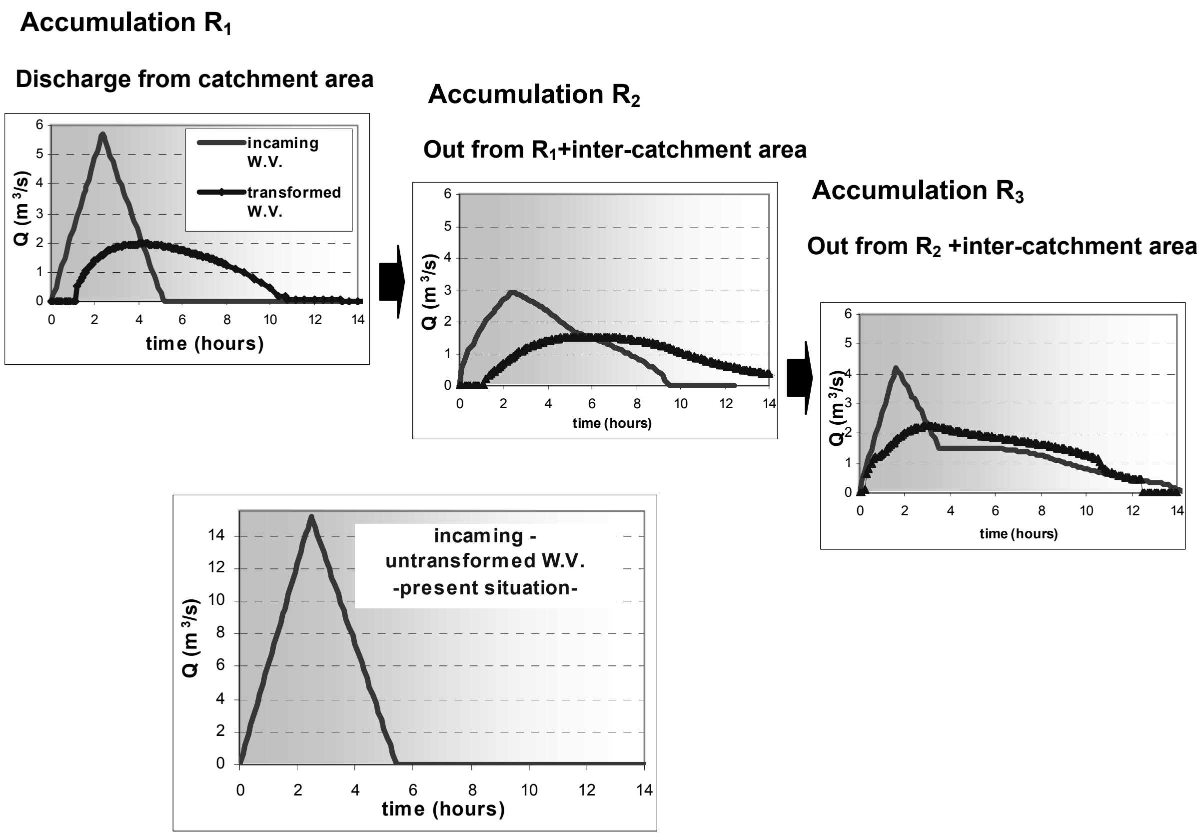

Figure 5 Comparison of maximal 100-years water wave for the existing situation and after the construction of planned retentions

5. Summary

Urban areas water drainage solutions are made according to hydrological calculations. Their results are the immediate consequence of precipitation regime of short termed intensive precipitations. The circumstance that on some area there is an existing system for channel drainage of precipitation water without proper solution of surface evacuation of precipitations of rarer appearance order than that according to which the drainage system is dimensioned on often gives an illusion of the existence of flood protection of that particular area. Such illusion is even greater if calculations of solutions for the drainage are based on inappropriate generalized matrices regarding the characteristics of short termed intensive precipitations and that do not consider spatial differentiation of these characteristics, which can, like in analyzed example in Croatia, be very pronounced.

Paper stresses an important difference between flood control of urban areas from that on open water streams. .River floods are usually of longer duration and are manifesting on wider area. It is usually easy to prove that benefits of flood control measures are greater then costs. With precipitations in urban areas the situation is the opposite – floods appear locally, they are short termed and the visible damages are consequentially relatively small. However, damage costs in crowded urban areas can be considerable. Therefore, needs for adequate precipitation waters drainage systems are much more complex to show compared to river floods (Margeta, 1998.).

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The Groundwater Flow Problems in Urban Areas

Jaromír Říha

Abstract: The groundwater regime in urbanized areas is influenced both by natural conditions and by intensive human activities. The natural conditions are formed by aquifer morphology often combined with the network of "buried" water courses draining the surrounding strata. The human impacts concern a change of geological structure (anthropogenic geological layers - fills), leakage from water supply mains, pumping from deeper cellars and subsurface garages and finally construction of deeply placed (founded) engineering network collectors in city centres. Construction of subsurface parts of civil structures significantly influences the groundwater flow regime, on the other hand, groundwater level conditions affect a technical solution to those structures and foundation methods at the site. In the paper, the groundwater flow problems in urban areas are summarised and demonstrated on two examples from cities of Brno and Prague.
Keywords: groundwater flow, subsurface structures, flood protection, urban areas

Introduction

The groundwater occurrence and flow are rather complicated in urbanized areas. It is due to several natural and above all anthropogenic factors influencing the groundwater flow regime. In this paper, examples of solutions to selected problems are mentioned which are connected with groundwater regimes on territories of Brno and Prague.

On territories of both cities, influential factors are represented by a morphologically rugged topography, a network of surface and subterrenean (buried) water courses, a complicated geological structure influenced by historical constructions (anthropogenic geological layers - fills), water leakage from engineering networks, water pumping from cellars of more deeply placed structures, construction of new subsurface and deeply placed structures, an extensive construction of primary collectors in city centers, construction of the subway on the Prague territory and built-up elements of flood protection.

This paper mentions three examples of influencing the groundwater flow regime by anthropogenic activities. Out of these, two examples are related to the city of Brno and reveal technical possibilities of eliminating a negative influence of construction activities on the adjacent housing development. Problems are connected with construction of primary collectors in the city centre and construction preparation of the PALACE CD edifice. The last- third – example concerns consequences of the proposed flood protection of Prague.

Both urbanized areas have a complicated geological development. With regard to a low permeability, a pre-quartenary underbed can be considered as an impervious stratum. The surface of the massive is irregular with depressions and talwegs. 1-9 m thick sandy gravel strata sit on neogeneous bedrocks and loess layers form an upper confining layer in case of higher groundwater levels. In the area concerned, an influence of the anthropogenic activity is evident and is manifested in an irregular distribution of fills of a various character and hydraulic properties like debris, old masonry relics and even existence of old cellars which can act hydraulically as privileged groundwater flow paths.

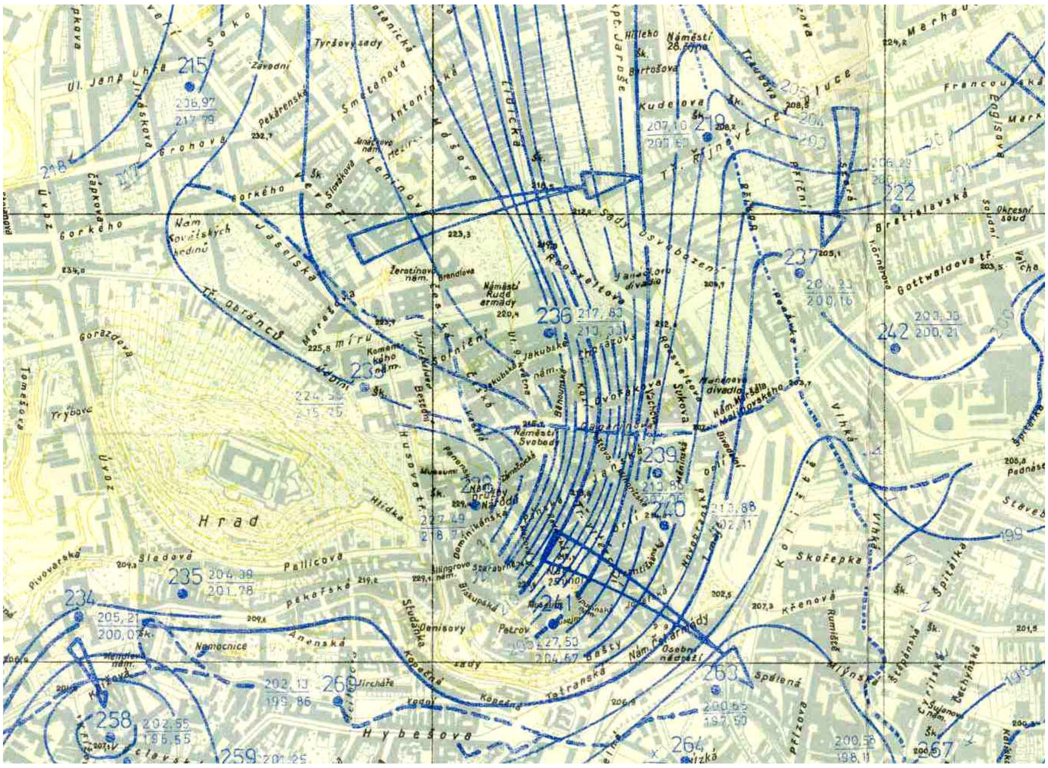

Figure 1 Groundwater flow direction in the Brno centre [Kouřil, Prokop 1977]

Primary collectors in the Brno city centre

In the Brno city centre, the groundwater level varies with maximums in spring months and minimums in October and November. The hydraulic conductivity coefficient of gravel sand sediments is estimated k = n.10^{-4} m/s. A prevailing groundwater flow is directed from lower locations in the area of the Dominikanske square across the Square of Liberty eastwards towards the Malinovske square and Cejl where the aquifer is drained by the Ponavka stream (Fig. 1). A natural groundwater flow related to climatic conditions and rainwater infiltration can be locally influenced by engineering network defects (water supply mains and sewer system). On the Brno territory, approximately 250 l/s water is estimated to leak to the underground and thus to extend the natural groundwater.

In the Brno city centre, the layout of secondary collectors was proposed. They intersect the groundwater flow and thus form an obstacle to its run-off. Hence, in some places “subsurface barrages” would come into existence causing the groundwater level increase by up to 3 m (Fig. 2). This condition would result in wetting the adjacent housing development and in flooding cellars in the city centre. It leads to the proposal calling for measures limiting a negative influence of transversal subsurface structures on the groundwater flow regime. It was clear that it was necessary to guarantee a hydraulic connection between both sides of the collector in a convenient way so as to ensure a mutual groundwater level communication.

Observing the groundwater level reveals that the groundwater flows in “natural” conditions through the area with the hydraulic gradient J = 0.05. On its basis, the marginal difference 0.17 m in the groundwater level on both sides of the collector was derived corresponding to the collector width 3.4 m.

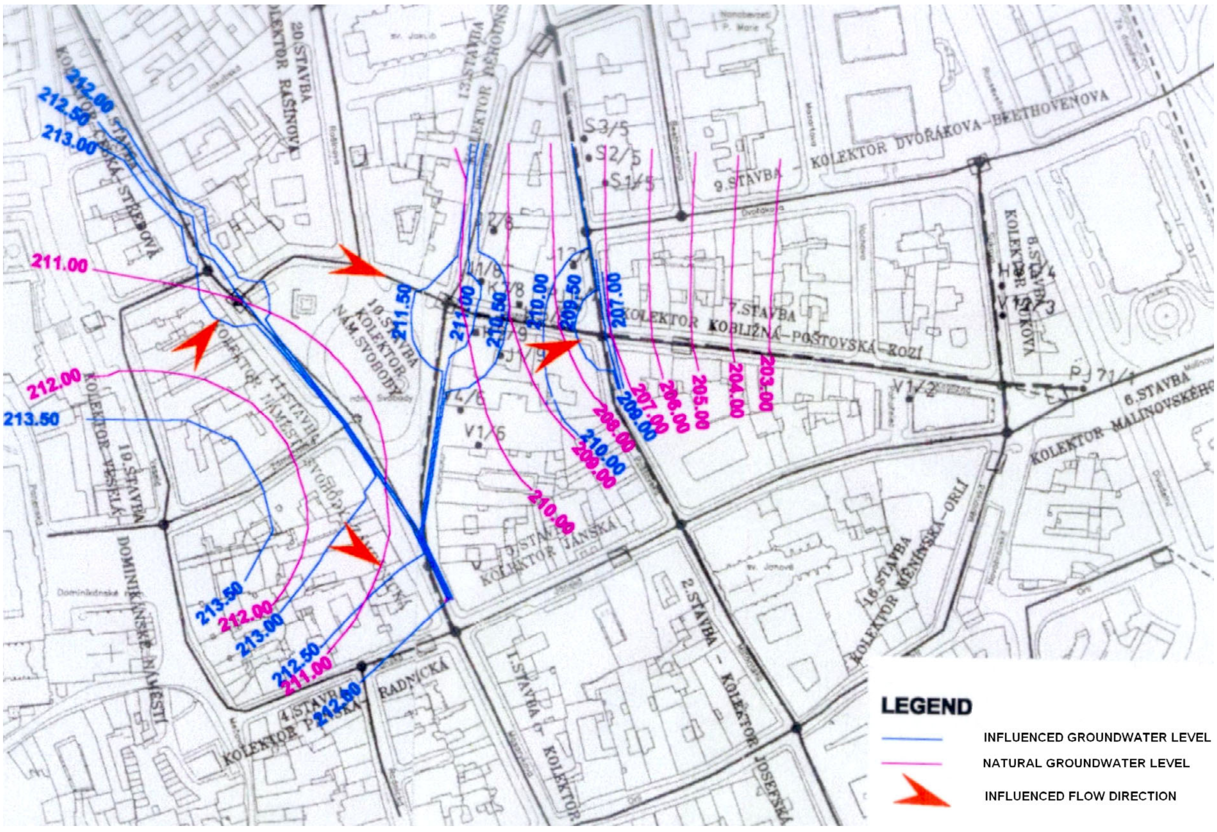

Figure 2 Groundwater level course influenced by the collector network

Results of hydraulic calculations proved that in case of collectors obstructing the natural groundwater flow dead corners would occur on their upstream face while the groundwater flow is redirected into the collector line and the groundwater level upheaves. In the lowest places of collector connecting chambers, water "overflows" the collector head. That is why, groundwater levels were calculated for single collectors in overflow places. For the assessment of hydraulic conditions and remedial elements, the groundwater flow model was employed.

The final proposal is the connection of both sides of the collector by a drainage flexible pipe with the inner diameter 30 to 60 mm complemented by geotextile fabric. The protection against silting by fine-grained particles at pipeline entry and exit points is guaranteed by geotextile bandaging.

Current construction experience points at difficulties with drainage elements on the outer tunnel lining where a collision takes place with elements of a provisional collector profile curb before its concrete casting.

PALACE CD edifice

The second locality under examination is the multifunctional centre "PALACE CD" construction with the built-up area equalling to about 9000 m^2 . The structure located in the Brno city centre will have 7 to 9 above-ground storeys and 5 underground storeys with the foundation depth up to 16 m in the pre-quartenary impermeable bedrock. The new civil unit forms an obstacle to the groundwater flow. The question is what impact a groundwater flow change might have on hydrogeological conditions in the nearest surroundings.

The groundwater occurs above all at quartenary fluvial gravel sediments in the Ponavka stream and its tributaries. A continuous horizon of groundwater with a phreatic or slightly confined flow regime appears in sediments. The groundwater is replenished by rainwater infiltration through higher layers of fills, then in the form of engineering network leakage (water supply mains, sewer system, heat ducts, etc.) and of water from "buried" water courses – the Ponavka right bank affluents. A relatively wide area is drained off by pumping 1 to 2 l/s which affects the groundwater level in cellars of the department store Centrum and Mahen theatre. It is evident from results of geological surveys that the territory in the PALACE CD construction site is partly drained by pumping from a well in the Morava palace.

Under both higher and lower groundwater level conditions, a drainage effect of the Ponavka is immediately demonstrated. The Ponavka drains groundwater from right-bank and left-bank aquifers (Figs. 1 and 3). Permeability of quartenary fluvial sediments varies considerably. In preceding research, it is evaluated by the hydraulic conductivity coefficient equalling to 10^{-7} to 10^{-4} m/s with an average value $2.5.10^{-4}$ m/s. The hydraulic solution to the impact of the „PALACE CD" edifice construction on the groundwater regime in its vicinity was performed for conditions during constructing subsurface walls and for conditions after the construction completion. The last alternative was to assess conditions after having implemented measures proposed.

The solution results indicated the increase of a piezometric level, as a result of constructing the subsurface part of the „PALACE CD“, to be, after its completion, in approximately 0.50 m on its northern „upstream“ face in the direction of the Morava palace (Fig. 3). Results confirm a negligible raise of the piezometric level in case of constructing subsurface wall parts which roughly follow the groundwater flow direction.

In the conceptual solution, it was looked into possibilities of various remedial measures resulting in the groundwater level maintenance at its present level. The final alternative was a interconnection of an upstream and downstream sides of the structure by drainage piping linked with positions of water-bearing quartenary gravel sands by the system of wells. On the upstream face, the drainage pipe complemented by perforated vertical release wells will perform its draining function. On the downstream face in the Koliste area, a “released” amount will infiltrate into a relatively permeable aquifer. An advantage of this arrangement rests in limiting the ballast water discharge into the sewer system, in leaving the groundwater in the aquifer natural system and finally in reducing costs of the water discharged into the sewer system. A chamber with an overflow in the sewer system will serve as a safety device.

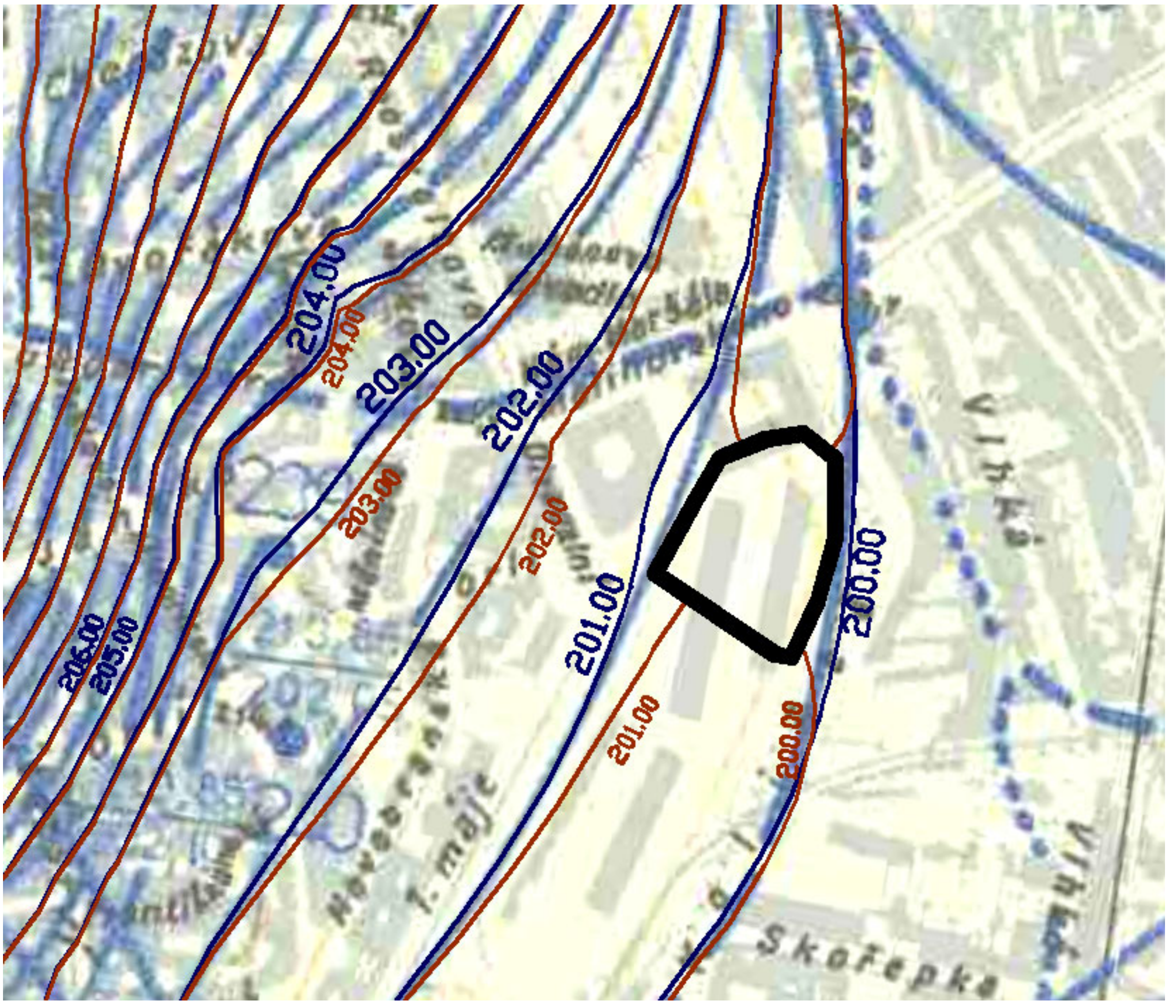

Fig. 3 The groundwater level affected by the PALACE CD edifice
(present state and state without remedial measures)

Flood protection of Prague

At the moment, flood protection of Prague is related to the Vltava water level corresponding to the peak discharge during the 2002 flood. Flood protection of the capital city area is guaranteed by technical measures including a terrain elevation, ground embankments, quay walls and mobile flood protection barriers. In many places, the elements mentioned create a considerable difference in elevation between the proposed Vltava flood water level and the area surface behind the protective elements.

When assessing hydrogeological conditions at the site, the existence of a relatively low permeable pre-quartenary bedrock is essential. In the area concerned, it is formed by slightly cracked slates. They are located of 10 - 15 m below the terrain level. Slates can be regarded as relatively impermeable, only the distorted near-surface layer has a capacity to conduct water. The slate overlaying formation is formed mainly by fluvial gravels, sandy gravels and sands with a mixture of 5 to 8 m thick gravels and with the hydraulic conductivity $k = 3.9.10^{-4}$ m/s to $1.5.10^{-3}$ m/s. Up to 6m thick holocene sandy clays are placed locally on the gravel surface. In the near-surface zone these sediments are replaced by non-homogenous consolidated 6 to 9 m thick made-up grounds. Their hydraulic conductivity ranges between $k = 5.10^{-5}$ to 1.10^{-6} m/s. Moreover, they are subject to subsurface erosion with the possibility of occurrence of privileged routes and local caves. The simplified scheme of the geological composition is shown in Fig. 4.

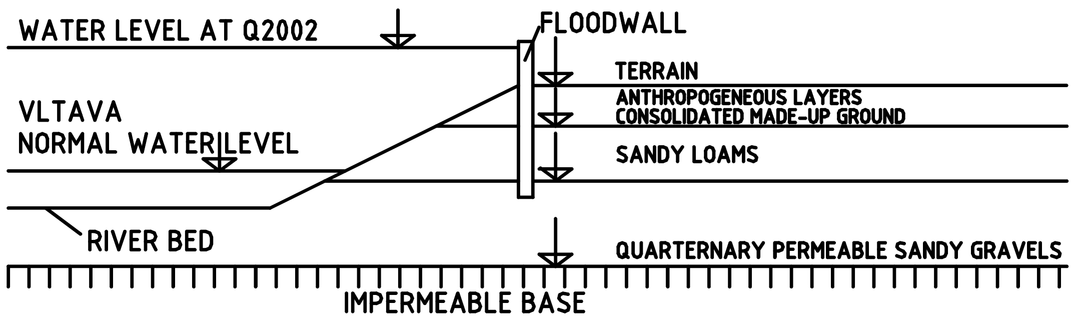

Figure 4 Example of a typical layer composition and of a flood protection measure

The groundwater level in an aquifer reacts sensitively to the Vltava water level changes. Habitually, the groundwater level occurs deeply below the terrain, i.e. 4.5 to 7.0 m. The groundwater flow direction roughly follows the Vltava flow and the river drains the adjacent aquifer according to the immediate water condition in the Vltava and in relation to the weir arrangement in the Vltava. During the flood, the Vltava water level increase results in a rapid change of the groundwater flow direction in the offshore zone. Infiltrated water leaks through permeable gravel sands in the direction from the river bed and consequently even through fills up to the surface of protected area. Groundwater unfavourable effects behind protective elements can be manifested by:

- uplift pressure on slightly permeable flood clays and their rupture;
- considerable leakage in the protected area;
- internal erosion with a possible loss of flood protection elements stability;
- wetting cellars and uplift pressures on cellar floors and walls.

In order to reduce these impacts, anti-leakage and drainage elements were proposed.

- cutoff walls carried out up to bedrock gravel sands cause a slight delay and a decrease in the leakage "wave" culmination in relation to an increasing depth. Not to distort natural connection between the water level in the Vltava and in the aquifer, it is not possible to carry out a continuous subsurface wall keyed into the impermeable bedrock. It could, as a matter of fact, bring about a groundwater surge and wetting buildings in the protected area;
- relief wells cause a multiple increase in the amount leaked with the necessity of its pumping back to the Vltava across the protective wall;
- drainage pipes running along the protective line prevents and checks water leakage to the terrain in the vicinity of the flood wall. Unfortunately, the use of a continuous drain often causes the problem of intersecting engineering networks;
- Earth loading fill at the bank side of the floodwall. The use of this measure is hardly possible in the built-up and intensively used area and faces problems related to rights of property in undeveloped areas.

With regard to local conditions, final measures make use of a suitable combination of single measures. Their effect was assessed in variants with the help of groundwater flow models. They proved it would be difficult and costly to guarantee safety of all the structures in the protected area normally required at hydrotechnical structures (e.g. dams). Thus, the following starting points were adopted for the final proposal for measures:

- safety of the flood protection elements is primary;
- in the protected area further from the line of flood protection elements, a local hydraulic cracking and non-controlled water leakage are accepted including a local terrain distortion.

During the construction preparation, combination of individual remedial measures was evaluated by mathematical models with the use of probabilistic and statistical methods. The project aim was to decrease investment costs to the minimum, i.e. to the line of flood protection measures while maintaining their reliability at an acceptable level.

Conclusions

In the paper, several problems are mentioned which are connected with the groundwater flow in the urbanised area. The issue has several aspects, out of which the most important ones are chosen:

- In the framework of the city development, these days buildings are constructed with several subterranean floors. The purpose is to use relatively expensive plots as efficiently as possible by placing e.g. subsurface garages in building basements. Thus, subsurface parts of buildings frequently reach the groundwater level. When proposing structures arranged in this way, it is necessary to take into account groundwater effects (uplift pressures on the foundation base, aggressive impact) on the newly proposed structure and a reverse influence on the new structure on the groundwater flow regime and an following impact on existing structures.
- In case of constructing a continuous line of flood protection, an “artificial” excess pressure occurs in the hydrogeological collector from the river side and the flood level gets propagated into the aquifer. Though the flood wave does not overflood the protected inundation area, it expands through permeable alluvial gravels towards the protected area. Then structures are strained by water pressures invoked by the groundwater advance in the offshore zone. In relation to permeability of alluvial fluvial loose sediments, the groundwater level advance can reach the tens to hundreds meter wide strap along the water course.

Experience shows that these aspects cannot be overlooked during the preparation of extensive investment and that it is always worth checking local conditions in the framework of the project elaboration through geological and hydrogeological surveys and through the use of interrelated model hydraulic calculations to carry out conceptual proposal for possible measures. They have to guarantee a sufficient protection of structures themselves against unfavourable groundwater effects and to eliminate, in a suitable way, groundwater flow regime changes and a possible impact on the adjacent civil structures and environment.

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Probability Estimation of River Channel Capacity

Jaromir Riha
Pavel Golik

Abstract: Recently, the risk analysis of floodplain areas has been one of the most frequently used tools for assessment and design of flood protection measures. One of sub-tasks of risk analysis is to estimate probability of river channel overbanking. The basis of a probabilistic approach to evaluation of river channel capacity is shown in this paper. Probability of river channel overbanking is directly connected to channel capacity, which is affected by a number of separate uncertainties, namely by input data uncertainty (e.g. hydraulic parameters, channel geometry and design discharge), mathematic model inaccuracy or statistical uncertainty. In the study presented, the Monte Carlo simulation was used for statistical evaluation of influence of the uncertainties mentioned above. The method proposed was applied and verified within the parametric study at the Svitava river channel.

Present state

Water structures are usually designed using deterministic calculation schemes. The designing of water structures in the Czech Republic is governed by Czech national standards (ČSN). When assessing the capacity of the river channel, the randomness included in the variability of the individual parameters entering the calculations (e.g. the N –year flood discharge, channel geometry, hydraulic parameters etc.) is reflected using the recommended freeboard, i.e. vertical distance between the crest of the levee (the bank line) and design water level. As an example, we can mention the recommended freeboard for the design discharge of Q_{100}, which is set in ČSN 73 68 20 at a value of 0.30 m above the calculated water level.

The result of the proposal for a corresponding deterministic conception need not always represent an optimum solution and it can result in overestimating or underestimating of the design parameters of the river channel.

Uncertainties

There exist a number of uncertainties affecting the calculations of channel capacity. Besides the aforementioned, this includes the uncertainty of the validity of assumptions used when developing the mathematical model, uncertainty of the numerical solution, uncertainty in hydraulic parametres, such as the roughness coefficient, Coriolis number etc. In the text the uncertainties related to three most important parameters are briefly described.

Uncertainties in hydrological data

In the Czech Republic, the hydrological data is provided by the Czech Hydrometeorological Institute (ČHMÚ) as a standard. The basic hydrological data include mainly the value of *N* – year and *M* – day discharges, which are usually used when designing the river channel capacity. Each report showing the basic hydrological data provided by the ČHMÚ includes a notice: "the data of flood level is not unchangeable, but may be changed according to new findings" and "the data is valid for 5 years from the date of issue". These facts and the data stated in ČSN 75 14 00 imply that the hydrological data provided by the ČHMÚ have a probabilistic character and that there are a number of uncertainties that may affect the magnitude of *N* –year (design) discharges. According to [Riha et al. 2005] these uncertainties are caused mainly by:

- errors in stream gauging and its averaging;
- errors in the measurement of the cross section (e.g. width, depth);
- errors in velocity measurement;
- errors in the calculations of flow rates and deriving of rating curves in the gauging profiles;
- errors in the conversion or extrapolation of the discharge series, characteristics and parameters into profiles with no monitoring.

ČSN 75 14 00 reflects the uncertainties of the *N* – year and *M* – day discharges by so called "standard errors". Depending on the character of the hydrological data and the reliability class, the standard error is estimated in the range of 5 – 80 %. The aforementioned standard does not specify to what value the percentage of the standard error is related. If the percentual determination of the standard error was related to the relevant flow rate value, the flow rate intervals for the individual *N* – year would overlap in certain cases. [Juranek, 2003]. The same study presents a procedure of quantifying the uncertainties of determining *N* – year flow rates in the Brno branch office of ČHMÚ. It applies the method of estimating the tolerance limits reflecting the fact that the Q_N values must be distinguished in practice (thus, the tolerance interval Q_{20} must not overlap with the tolerance interval Q_{50}. The usually presented values of tolerance limits relatively related due to the Q_N values, are, as an example, in a range of ± 5 % for streams in the first class of hydrological data reliability.

Uncertainties in the channel geometry

The reliability of data, which describe the channel geometry for hydraulic calculation, is often affected by a high number of uncertainties and inaccuracies. The following list shows the sources of uncertainties in the channel geometry, and identifies the possibilities of assessing and reducing the effects of these uncertainties as well:

- Surveying of the river channel is performed in dependence on the financial possibilities of the investor and on the basis of the requirements raised by the hydraulic engineer conducting the calculation. The distances between the cross sections are usually selected so that the profiles can copy the geometry of the river channel as closely as possible and so that it is possible to interpolate potential intermediate profiles. However, the calculation of the capacity does not reflect the unevenness of the river bed and banks, which is not detected by the surveying (e.g. they are located between the surveyed profiles).
- When surveying the cross sections, the individual surveyed points are located so that the segments between them can be approximated by lines. For various reasons (e.g. adverse conditions during measuring, insufficient qualification of the lineman, low budget and, consequently, thin network of surveyed points etc.) it may be expected that the surveyed geometry does not enough describe the real channel.
- In both natural and regulated channels, the geometry changes in time and space. During higher discharges, erosion and alluvial fans occur and these changes are often neglected when calculating the channel capacity.
- As stated above, geodetic surveying of the channel is considered to be “error-free” and “accurate” and potential measurement errors are not generally taken into account in the calculations.
- When regulating or repairing the river channels, the inaccuracies may occur (deviations from the design) already during the construction; the effect of these inaccuracies is not usually reflected in the calculation, either.
- The position of the levee crest may experience time-dependent changes for many reasons. For example, imperfect compacting of embankment, subsoil with low load-bearing capacity, internal erosion of the levee, its frequent crossing by agricultural vehicles, etc. In exceptional cases undermining or dismantling of parts of the levee by local inhabitants may be experienced.

Uncertainties in the roughness coefficient

The roughness coefficient is the important hydraulic parameter, which directly affects the calculated channel capacity. A frequent problem consists in determining of the appropriate value of the roughness coefficient, which is normally set by an expert estimate. Determination of roughness coefficient includes subjective elements affected by the hydraulic expert’s experience. Its value is estimated on the basis of site investigation, photographic documentation of the stream, grain size distribution curve of the river bed material etc. This can be specified using the calibration of the mathematical model. Thus, it is obvious that determination of the roughness coefficient may be to a certain extent affected by the experience and opinion of the hydraulic engineer and it contains a considerable degree of uncertainty.

The study [Koutkova, 2003] shows the results of an experiment. To evaluate the uncertainty, 30 hydraulic experts were asked to evaluate roughness coefficient based on photographic documentation for twelve various channels.

The conclusion of the study shows, that almost 80 % of experts have underestimated the value of roughness coefficient determined by hydraulic model calibration. In practice it may cause underestimating of the river channel capacity.

Probabilistic approach

Statistical modelling of stochastic phenomena is described in e.g. [Rao, 1992] or [Teply, Novak, 2004]. The main principles of these methods are shown in following text. The basic requirement for structure reliability is:

$$S > L\,, \tag{1}$$

where S stands for structure resistance (channel capacity) and L for the load (flood discharge). Using the probability approach, this can be replaced by the following relation:

$$R = P(S > L) = P((S - L) > 0), \tag{2}$$

evaluating the probability that the structure resistance will be higher than the load, in other words, reliability R is evaluated. The probability P_F of the occurrence of failure can be set as follows:

$$P_F = P(L \geq S) = P((L - S) \geq 0)\text{, i.e.:} \tag{3}$$

$$P_F = 1 - R\,. \tag{4}$$

When evaluating the structure reliability using the probabilistic approach, higher knowledge of statistical and probabilistic characteristics of the parameters entering the calculations is necessary (above all, the probability density and distribution functions). Based on these data, the probability density functions *f(s)* and *f(l)* resistance S and load L are derived, as shown in Fig. 2.

If parameters S and L are statistically independent, the failure probability can be defined as an area limited by curves *f(l)*, *f(s)* and axis Q (l, s) (see Fig. 2). The final failure probability corresponding to the size of mentioned area, can be determined through integration of the probability density curves within relevant limits, see equation (5). Those limits generally shown in the equation (5) are specified for the practical application in the equation (6). The integration can be made used by the numerical integration.

$$P(l > s) = \int_{-\infty}^{X} f(s)ds + \int_{X}^{\infty} f(l)dl \tag{5}$$

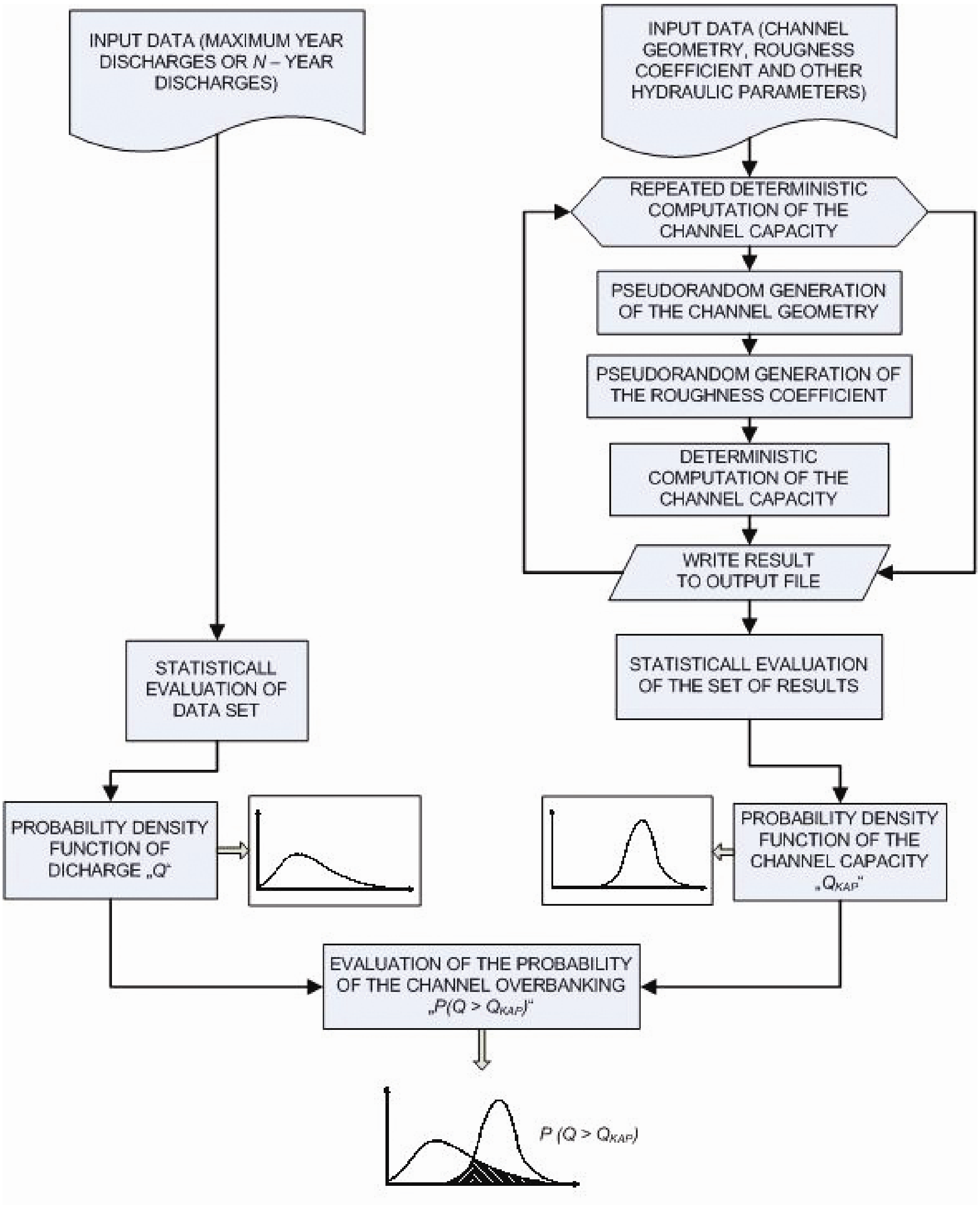

Fig. 1 Procedure scheme of the calculation of the probability of the river overbanking.

Probabilistic assessment of the river channel capacity

The probabilistic analysis of the channel capacity performed by the statistical modelling usually provides results with a higher information value than the commonly used deterministic computations. This analysis does not result in a single specific value of the channel capacity, but in data providing probability density function of the channel capacity, respectively probability of river channel overbanking.

When evaluating the probability of river channel overbanking, another parameter enters the calculation namely the natural discharge "loading" the channel. The analysis of relations between the discharge and the channel capacity may be conducted in several manners; however, the result must always be the probability of river channel overbanking.

The practical use of the statistical modelling methods is pre-conditioned by a sufficient knowledge about the variable parameters affecting the calculation. These parameters can be described using the probability density, or using the basic statistical moments – mean, standard deviation, etc. When the sufficient data of some of the parameters are not available, it is possible to use "deterministic" value for the calculation. It is also possible to choose an appropriate probability distribution function (uniform, triangular, etc.), based on an expert opinion. When estimating the probability of channel overbanking using the statistical modelling, several methods can be employed. In Fig. 1 the methodology which was applied in this paper is shown.

Practical application

The Svitava river reach in the urban area of Brno, river km 4,885 - 5,175 was selected for the case study. The main reason for choosing this locality has been the sufficiency of basis documents [Gimun et al., 2003], [Koutkova, 2003] and [Riha, Golik, 2004], easy accessibility of the locality for further potential measurement and regular shape of the channel suitable for simplified hydraulic calculations.

Probability density function of the discharge Q

When evaluating the probability density function of discharge Q, data from a set of peak flows in the Svitava recorded by the stream gauge station in Bilovice between years 1918 - 2003 [Juranek, 2005] were used. The set contains 134 peak discharges in total with minimum and maximum values of 13 m^3/s (1918) and 170 m^3/s (1938), respectively. The empirical exceedance curve was approximated by a two-parametric logarithmic-normal distribution (Fig. 2, the *f(l)* line).

Probability density function of the channel capacity Q_{KAP}

When evaluating the probability of channel capacity in the practical application, the uncertainty in channel geometric data and in channel roughness coefficient were considered.

The results of previous studies [Koutkova, 2003 and Riha, Golik, 2004] were used for determination of the channel capacity probability density function. This was performed by means of statistical simulations using the Monte Carlo method.

The channel geometry and roughness coefficient were pseudo-randomly generated and for every set of input data the deterministic computation of the channel capacity was performed. The dashed line in Fig. 2. shows the empirical probability density function of channel capacity.

Results of the study

The final evaluation of the results was conducted in compliance with the methodology described at the beginning of this article. The probability of river channel overbanking can be shown in Fig. 2 as an area limited by the probability density curves of the channel capacity and discharge and by the horizontal axis.

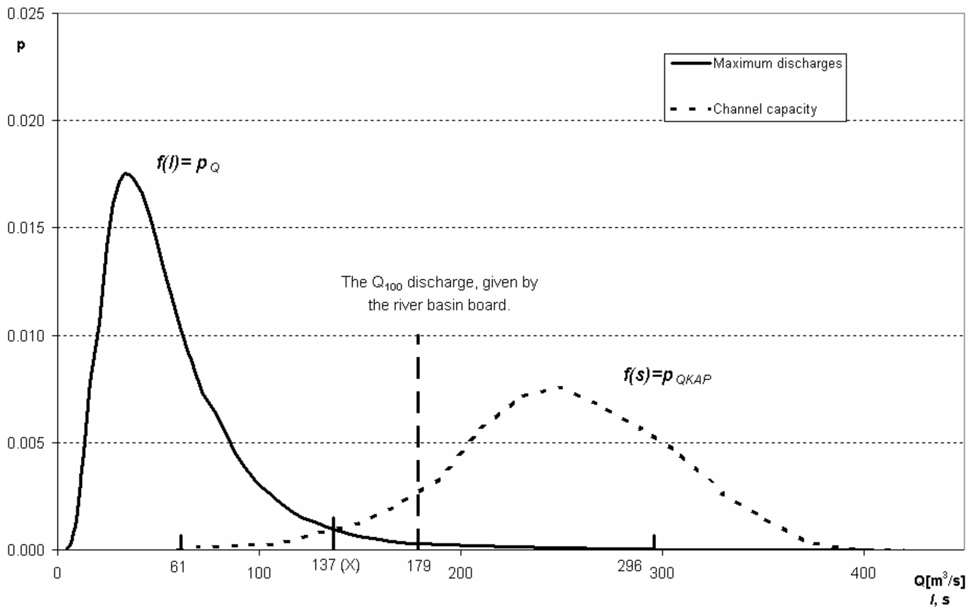

Fig. 2 Density of Q flow rate and QKAP channel capacity probability

The probability of river channel overbanking estimated on the basis of the river channel and the discharge probability density functions derived from the maximum year discharges is:

$$P_{MAX} = \int_{61}^{137} p_{QKAP} dQ + \int_{137}^{296} p_Q dQ = 0.066 \,. \qquad (6)$$

Gimun [2003] indicates, that the capacity of the Svitava in the evaluated river reach is about 179 m^3/s, what equals the Q_{100} discharge. Then the probability of the Svitava river overbanking estimated based on the data provided by the river basin is approximately:

$$P_{RBB} \leq \frac{1}{N} \leq 0.01, \quad (7)$$

where $N = 100$ is the return period (exceedance interval).

Conclusion

Our study shows that the probability of Svitava river overbanking estimated by the river board agency Povodi Moravy, state enterprise, is substantially lower (more then six times) than the probability of overbanking estimated based on the results of statistical modelling. The aforementioned facts imply that the risks encountered in floodplains as a result of floods are actually higher than the risks determined using standard methods.

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Determination of CSO Spill Frequency Based on Rainfall Data

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Abstract: In the paper a simple method of CSO (combined sewer overflow) spill frequency determination, which uses historical rainfall records and linear reservoir model, was described. The method is based on evaluation of characteristic parameters, such as the time of duration and the intensity of the rainfall which initiates CSO, further referred to as the initiating rainfall. An initiating rainfall is determined individually for particular CSO as a function of the catchment parameters and the parameters of CSO spillway. The initiating rainfall intensity is calculated on basis the flow with depth equal to weir crest level. The duration time of initiating rainfall is a function of catchment parameters and travel time in sewers. Classification of historical rainfalls according to parameters of initiating rainfall makes possible to determine the number of rainfalls generating CSO spills in a specified period, hence the frequency of spills can be easily computed. In the paper a case study results were presented. Results obtained from described method are almost the same as results of SWMM5 package simulation of the rainfall-runoff process in urban catchment.
Keywords: CSO, spill frequency, rainfall-runoff process

1. Introduction

In Poland, like in many other countries, the evaluation of an impact of CSO spills on receiving waters is based on the criterion of outflow frequency in a year. The allowed frequency, determined by Polish Regulations is equal to 10 events/year. A practical application of this criterion encounters difficulties. For existing urban drainage systems, it could be based, theoretically, on data from measurements performed for a sufficiently long period of time. Such measurements are usually not available. Hence, in this case, as well as for new designed systems, a simulation of rainfall-runoff processes based on computer modeling seems to be an attractive alternative. It is an effective tool, but its application requires, apart from access to a relevant computer model, a full set of data including meteorological (rainfall), hydrologic (catchment) and hydraulic (sewers) subsets together with considerable computational effort due to the large number of simulations required in order to identify rainfalls causing CSO spills (Skotnicki and Sowinski, 2003). An application of methods based on simplified assumptions – less accurate, but easier and cheaper – seems reasonable in many cases (Dąbrowski, 2004).

The method presented in the paper, which belongs to the latter category, is based on an application of the linear reservoir model to transformation of rainfall into outflow. The implementation of the method is based on the two characteristic parameters: the time of duration and the intensity of rainfall initiating CSO spill.

2. Determination of rainfall initiating the spill

2.1 Assumptions of the method

The following assumptions were made when deriving the basic relationships of the method:

- The runoff is formed only from impervious surfaces.
- The surface runoff hydrograph is equivalent to the outflow hydrograph from a sewer network; transformation of hydrograph shape in sewers is not considered; only a delay as a result of travel in sewers is taken into account.
- The rainfall intensity is constant in time and uniformly distributed over a whole catchment area; for rainfalls with an insignificant variation of intensity (in time and space) a mean value can be used.

2.2 Determination of a spill initiating flow

Spill initiating flow Q^* is defined as the flow at which CSO spill commences (its depth is equal to weir crest height). It can be computed as the sum of dry weather flow Q_{DW} and the spill initiating storm flow Q_S^*:

$$Q^* = Q_{DW} + Q_S^* \ [dm^3/s] \qquad (1)$$

In storm sewer systems the dry weather flow Q_{DW} is a result of groundwater infiltration. In case of combined sewer systems, this flow can be identified with foul flow, which is usually much larger than infiltration water flow. In some specific cases, a basic flow can be assumed to be equal to zero. The spill initiating flow Q^* can be evaluated by measurements or using hydraulic calculations for known sewer parameters (shape, dimensions, bottom slope and roughness) and a weir crest elevation above the bottom of the sewer.

2.3 Determination of a spill initiating rainfall intensity

The spill initiating rainfall intensity q^* is the mean rainfall intensity, which generates a spill initiating storm flow Q_S^*. According to the first assumption in the previous section, spill initiating storm flow Q_S^* is equal to quotient of spill initiating rainfall intensity q^* and impervious catchment area:

$$Q_S^* = q^* \cdot \varepsilon \cdot A = q^* \cdot A_Z \ [\mathrm{dm^3/s}] \quad (2)$$

Rearranging equation (2) and using equation (1), one can compute the spill initiating rainfall intensity.

2.4 Determination of spill initiating rainfall duration time

The spill initiating rainfall duration time T^* is defined as the time from the initiation of rainfall to the time of equilibrium between the maximum runoff and the outflow in the CSO cross-section. This time is the sum of three components:

- T_{LOSS} – the time from the beginning of rainfall to the beginning of runoff from a catchment. i.e. the hydrological losses time (Fig. 1),
- T_O – the time from the beginning of runoff to the time when it reaches its maximum value, i.e. the time of outflow; this time is understood in literature as the time of concentration,
- T_T – the time of surface runoff wave travel to the CSO cross-section through the sewer network, i.e. the travel time.

$$T^* = T_{LOSS} + T_O + T_T \ [\mathrm{min}] \quad (3)$$

The hydrological losses time T_{LOSS}, i.e. the time needed for wetting a catchment surface and filling its retention storage H_{LOSS}, can be calculated from the formula:

$$T_{LOSS} = \frac{166.67 \cdot H_{LOSS}}{q*} \ [\mathrm{min}] \quad (4)$$

If data from measurements are not available, hydrological losses can be taken in the range 1.5 to 3.0 [mm] (Huber and Dickinson, 1988).

For the determination of the outflow time T_O, the linear reservoir model, widely used in hydrology, can be applied (Chow, 1964; Maidment 1997). Total outflow hydrograph in this model is the sum of N hydrographs, each being generated by the single rainfall event of duration Δt. In the case of rainfall with the constant intensity, the sum of hydrographs can be replaced by one hydrograph generated by a rainfall event with duration equal to $\Delta t \cdot N$.

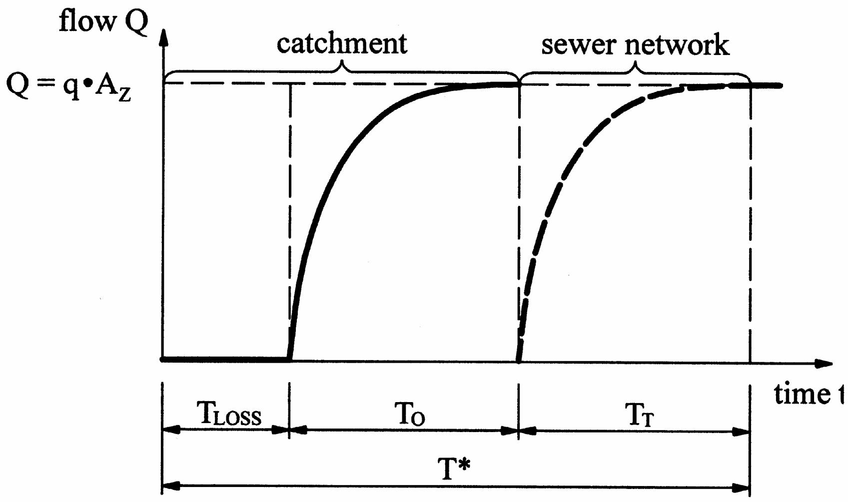

Figure 1 Graphical interpretation of initiating rainfall duration time

The hydrograph is composed of two segments – a rising and a falling one. Because only the time of reaching a given outflow value is of interest in the presented analysis, the equation for the rising segment will be used (Sieker et al., 1988):

$$Q = q \cdot A_Z \cdot \left(1 - e^{\frac{-t}{K}}\right) \ [\mathrm{dm}^3/\mathrm{s}] \qquad (5)$$

Equation (5) indicates that the value $q \cdot A_Z$, equivalent to Q_S^* , will never be reached (Fig. 1). This means that the outflow time T_O is equal to infinity. Taking into account that the curve described by the expression in brackets approaches asymptotically $Q_S^* = q \cdot A_Z$, the time T_O can be approximated by the time required to reach outflow value Q_S^* reduced by some percentage (e.g. 5%). Taking this into account, assuming in equation (5) $Q = 0.95 \cdot Q_S^*$ and $t = T_O$, one obtains the relationship between the reservoir constant K and time T_O:

$$T_O \cong 3 \cdot K \ [\mathrm{min}] \qquad (6)$$

The reservoir constant is a function of catchment characteristics (Sieker et al., 1988):

$$K = \frac{0.67 \cdot L^{0.6} \cdot n^{0.6}}{s^{0.4} \cdot I^{0.4}} \ [\mathrm{min}] \qquad (7)$$

It was found that the catchment characteristics appearing in equation (7) should be evaluated as weighted averages with the impervious subcatchment area as a weight. The coefficient of imperviousness should be determinated as a weighted average too, but with the total subcatchment area as the weight (Huber and Dickinson, 1988).

The travel time in sewers T_T can be calculated from the relation:

$$T_T = \frac{L_{TOT}}{60 \cdot v_M} \text{ [min]} \tag{8}$$

A mean velocity v_M in equation (8) should be evaluated as a weighted average of velocities in fulfilled sewer links with a link length as a weight.

3. Classification of historical rainfall records

A spill initiating rainfall can be described by two parameters:

- the duration time T^*
- the spill initiating intensity q^*

Identification of spill generating rainfalls in a set of historical rain events is based on these parameters and performed according to the following criteria:

- Rainfalls with intensities smaller than the spill initiating intensity q^* do not generate spill regardless of their duration,
- Rainfalls with intensities equal to or larger than the initiating intensity q^* and with time of duration longer than T^* always generate spills .
- Rainfalls with intensities equal to or larger than the initiating intensity q^*, but with time of duration shorter than T^*, generate spills depending on their duration T

3.1 Determination of the spill initiating time for rainfall intensity larger than q^*

Evaluation of the spill initiating time T can be performed by applying a procedure similar to that described earlier for the rainfall intensity q^*. First, a new value of the reservoir constant K_1 (eq. 7) should be calculated as a function of the rainfall intensity q (larger than q^*). The time required to reach Q_S^* in CSO cross-section is substituted by the time to reach this value reduced by 5% and described as T_{O1}. Using equation (5), the outflow time T_{O1} can be expressed as:

$$T_{O1} = \ln\left(1 - \frac{0.95 \cdot q^*}{q}\right)^{-1} \cdot K_1 \text{ [min]} \tag{9}$$

For rainfalls with intensity higher than q^*, CSO spills are generated for rainfalls with duration equal to or longer than T, calculated as:

$$T = \frac{H_{LOSS}}{166.67 \cdot q} + T_{O1} + T_T \text{ [min]} \tag{10}$$

4. Verification of proposed approach

The verification was aimed to check the following hypotheses:

- The sum of runoff hydrographs from partial catchments can be replaced by a runoff hydrograph evaluated for the whole catchment, which is based on average parameters of a catchment,
- The deformation of surface runoff hydrograph during travel through sewer system is negligible in comparison with the travel time,
- The difference between the number of CSO spills obtained from the proposed method and that determined by the computer package SWMM5 is acceptable for simplified approach.

In order to verify these hypotheses, a case study was undertaken. A catchment of a combined sewer network in Glogow was chosen (Skotnicki, 2001). The total area of this catchment amounts to 236 [ha], the mean coefficient of imperviousness is equal to 24.7 [%].

The catchment is split into 28 subcatchments. The analysed part of the sewer network includes 31 links with circular cross-section. Following the assumption specified in the first section of the paper, a constant rainfall intensity was assumed. Surface runoff hydrographs were calculated by applying the linear reservoir model in an Excel 2000 spreadsheet. The flow in channels was computed by means of the EPA SWMM5 computer program.

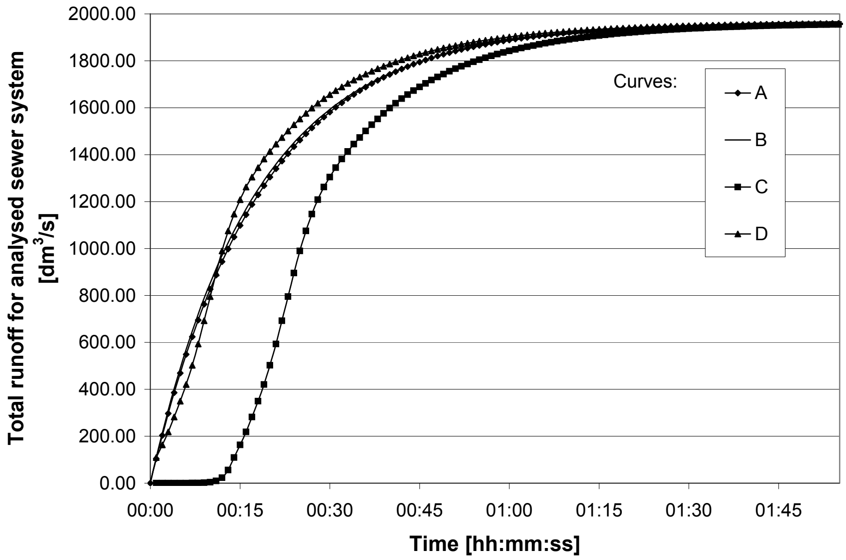

Figure 2 Comparison of outflow hydrographs evaluated by different approaches

In the first stage of verification, the surface runoff hydrograph computed as the sum of 28 runoff hydrographs from individual subcatchments (curve B on Fig. 2) was compared with the surface runoff hydrograph computed for the whole catchment. The parameters of the whole catchment hydrograph were derived as weighted averages, according to the proposed approach (curve A, Fig. 2). The results presented in Figure 3 show that the differences in shape of both hydrographs are negligible, which confirms the first hypothesis.

In the second stage of verification, the surface runoff hydrograph (curve C, Fig. 2) generated by the SWMM5 package was used. It is shifted by ~14 [min] in comparison with hydrographs (A) and (B), due to the travel time of the surface run-off wave through the sewer network. In order to compare its shape with the shape of hydrographs (A) and (B) curve C was shifted back to the origin of coordinates (curve D, Fig. 2). It can be seen that this curve is similar to curves A and B. The difference being in the range ± 2 [min] can be neglected in comparison with the shift time of the hydrograph described earlier.

The aim of the last stage of the verification was to check validity of outcome of the method, i.e. the frequency of the CSO spills by its comparison with the outcome of the SWMM5 package simulation. It required an implementation of a fictitious CSO (weir crest height 0.4 [m]) upstream of the outlet of the main collector (diameter 1.4 [m], invert slope 1.08 [%]) of the Glogow sewer network analysed earlier.

Table 1 Results of CSO frequency determination

Rain event	Rainfall duration t [min]	Mean rainfall intensity q [dm^3/s·ha]	Spill initiating time T [min] eg. (10)	CSO Spill		Depth in CSO weir cross-section H [m]
				according proposed method	according SWMM5 simulations	
RE1	105	21.4	70	yes	**yes**	0.41
RE2	145	20.7	75	yes	**yes**	0.44
RE3	70	27.4	48	yes		0.39
RE4	35	36.9	36			0.32
RE5	130	20.2	80	yes	**yes**	0.46
RE6	45	25.9	51			0.27
RE7	30	83.3	22	yes	**yes**	0.63
RE8	30	22.2	65			0.15

Based on these parameters and catchment parameters, assuming Q_{DW} = 0 and H_{LOSS} = 1.7 [mm], the spill initiating rainfall was characterised:

- spill initiating rainfall intensity q^* - 18.6 [dm^3/s·ha]
- spill initiating rainfall duration time T^* - 105 [min]

Eight historical rainfall were selected for analysis. Their parameters are specified in second and third columns of Table 1, together with values of the spill initiating time T computed from equation (10), number of the CSO spills computed by the proposed method and applying the SWMM5 package. The table shows that the number of rainfalls generating spills from the CSO weir is almost identical in both approaches. Selection of rainfall RE3 as generating spill only by the proposed method can be explained by distribution of this rainfall intensity in time. Maximum intensity of this event is almost four times greater than mean intensity hence the assumption of constant rainfall intensity is not satisfied. In consequences this rainfall was false classified as spill generating.

5. Conclusions

The proposed method allows for the evaluation of the frequency of CSO spills based on data including historical rainfalls, catchment parameters, basic information on the structure of the sewer network and the CSO under consideration. The method is relatively simple and the required data are easily available. The method allows for an easy correction of results due to changes of data concerning characteristics of the CSO or catchment parameters. Because of simplifications applied in the method, the obtained results should be treated as approximations of values evaluated by more accurate methods, e.g. based on simulation of rainfall-runoff process.

Notations

A - the total catchment area [ha],
A_Z - the impervious catchment area [ha],
H_{LOSS} - the hydrological losses; the rainfall depth needed for wetting surface and filling retention storage [mm],
I - the rainfall intensity [mm/min],
K - the reservoir constant (catchment specific) [min],
L - the catchment length [m],
L_{TOT} - the longest flow path to CSO weir cross-section [m],
n - the Manning's surface roughness [-],
q - the rainfall intensity [$dm^3/s \cdot ha$],
q^* - the spill initiating rainfall intensity [$dm^3/s \cdot ha$],
Q - the runoff [dm^3/s],
Q^* - the spill initiating flow [dm^3/s],
Q_{DW} - the dry weather flow[dm^3/s],
Q_S^* - the spill initiating storm flow [dm^3/s],
s - the surface slope [-],
t - the time [min],
v_M - the mean flow velocity [m/s],
ε - the percentage of imperviousness [-]

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Hydrologic-Hydraulic Assessment of Internal Water Drainage in Lowland Regions of Slovakia

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Radomil Květon
Dana Baroková, Estera Vércseová

Abstract: The contribution is a continuation of author research activities in the past and present time and shows the results of hydrological as well as hydraulic assessment of internal water drainage in one of the most critical region in Slovakia from hydrological, pedological and orographical point of view (Bella, V., 1971). The main reason for water surplus in lowland regions of Slovakia are precipitation and region conditions, flood regimes of rivers, efficiency of drainage systems. Results show the necessity of new pumping equipment at pumping stations and what kind of response does it have on surface water regime in the main channels of the drainage systems in the lowland region. The described region is one of the greatest drainage areas of the East Slovakian Lowland (ESL), so called ESL-4 where the main pumping station is Streda over the Bodrog River with capacity of more than 20.0 $m^3.s^{-1}$ (Šoltész, A.-Šoltész, J., 1999). The interaction of surface and subsurface water has been considered in the hydraulic assessment, as well.
Keywords: internal water excess, hydrological and hydraulic assessment, numerical modelling in drainage channels, pumping rate

1. Introduction

The investigated Medzibodrožie region (i.e. region along the Bodrog River) is a very specific region. The first specification is that it is the lowest region of the Slovakian Republic. The second specification is that this region is a third part of a region which is a joint Hungarian – Slovak region between the Tisza and Bodrog rivers. The area of the Slovak part which has been considered is more than 300 km^2. Although the East Slovakian Lowland is typical as a region with very heavy soils (Šútor, J. et al., 1995), the Medzibodrožie region is an example for another access to surface and ground water interaction.

Internal waters of the Medzibodrožie region are drained by a system of channels with a main drainage Somotorsky channel which is more than 25.5 km long. The longitudinal slope of this channel is 0.2 ‰. The main pumping station Streda over Bodrog is working since 1961 with a pumping rate 20.0 $m^3.s^{-1}$ (Rehák, Š. et al., 1997). There are two additional pumping stations in this region with more than 12.0 $m^3.s^{-1}$ pumping rate.

The most complicated period for creation of internal water surplus in East Slovakian Lowland are precipitation in two temporally distinguished cases:

- in the spring time after snow melting and
- in the vegetation period after increased rainfall activity.

2. Methodological procedure

Concerning to the methodological procedure elaborated in (Šoltész et al., 1997) we have analysed hydrological data from the 20-year series from the hydro-meteorological station Somotor. As the most critical period the spring time of the year 1999 was chosen. According to this data complete snowmelt was performed in 13 days (21.2.-5.3.1999), where the most critical period was a 6-day period (28.2.-5.3.1999), Fig.1. On the base of precipitation data, height of the snow cover and snow density value a water value of snow cover has been determined and its daily change can be expressed from the equation:

$$h_{0i} = (H_i + Z_i) - (H_{i+1} + E_i) \qquad (mm), \tag{1}$$

where h_{0i} - water value of the of the snowmelt in mm in the evaluated day,
H_i - water value of the snow cover in mm in the beginning of the snowmelt,
Z_i – precipitation in mm in the evaluated day,
H_{i+1}- water value of the snow cover in mm in the next day,
E_i – evapotranspiration from the snow cover in the evaluated day.

When evaluating the water value of the snow cover in conditions of the East Slovakian Lowland the density value of 0,27–0,28 (Šoltész, J. et al., 1975) was taken into consideration.

Tab.1 shows the evaluated loading of the drainage basin in Medzibodrožie region with water surplus for the hydro-meteorological station Somotor for a spring period 1999 as well as the data about potential pumping at the station. Tab.2a and Tab.2b show the melting snow load in the catchment at several values of the specific runoff coefficient φ. From the data comes out that the pumping rates should almost satisfy the necessity of draining internal waters in the Medzibodrožie region. The daily snow cover height and the course of daily temperatures (mean and maximum) are shown in Fig.1.

Tab. 1

Pumping plant: *Streda over Bodrog*
Drained area: $153\ km^2$
Maximum capacity of pumping plant: $20\ m^3.s^{-1}$
Daily maximum pumped water volume: $1{,}728\ 000\ m^3.d^{-1}$
Specific runoff ensured by pumping plants q: $1.307\ l.s^{-1}.ha^{-1}$
Assessed gauge station: *Somotor*
Assessed snow cover thawing period: *21.2. - 5.3.1999 (13 days)*
1.a) selected most adverse partial period: *28.2.- 5.3.1999 (6 days)*
b) melted snow water content: *88.6 mm*
c) melting snow load in the catchment: $13{,}555\ 800\ m^3$
d) assumed water runoff *O (cum)* at the runoff coefficient φ, pumping time t_1 (d) necessary for pumping the assumed water runoff, whereby the pumping plant maximum capacity is fully utilized and
e) provided that the runoff is *O(cum)* pursuant to d), *V* (cum) of water are retained in the given area, whereby t_{11} (days) are necessary to drain the water:

Tab. 2a

φ	O		q	t_1	V		t_{11}
~	(m^3)	%	$(l.s^{-1}.ha^{-1})$	(d)	(m^3)	%	(d)
1	13,555 800	100	1.709	7.85	0	0	0.00
0.6	8,133 480	60	1.025	4.71	5,422 320	40	3.14
0.5	6,777 900	50	0.855	3.92	6,777 900	50	3.93

2.a) total snow thawing period: *21.2.- 5.3.1999 (13 days)*
b) melted snow water content: *152.8 mm*
c) melting snow load in the catchment: $23{,}378\ 400\ m^3$
d) assumed water runoff *O (cum)* at the runoff coefficient φ, pumping time t_2 (d) necessary for pumping the assumed water runoff, whereby the pumping plant maximum capacity is fully utilized and
e) provided that the runoff is *O (cum)* pursuant to d), *V* (cum) of water are retained in the given area, whereby t_{22} (days) are necessary to drain the water:

Tab. 2b

$\tilde{\varphi}$	O		q	t_2	V		t_{22}
	(m^3)	%	$(l.s^{-1}.ha^{-1})$	(d)	(m^3)	%	(d)
1	23,378 400	100	1.360	13.53	0	0	0.00
0.6	14,027 040	60	0.816	8.12	9,351 360	40	5.41
0.5	11,689 200	50	0.680	6.77	11,689 200	50	6.76

The reality was completely another. There was a great flood from internal water surplus in the spring time 1999 and a detailed hydraulic assessment was necessary to be performed in the drainage channel system to get an answer why the pumping at the pumping plant was not successful. It is apparent that the pumping rate at the station is a very theoretical data and the efficiency of pumps decreases rapidly with the transport head of the pump. The evidence for this maintenance is the total pumping period of 33 days (Fig.2).

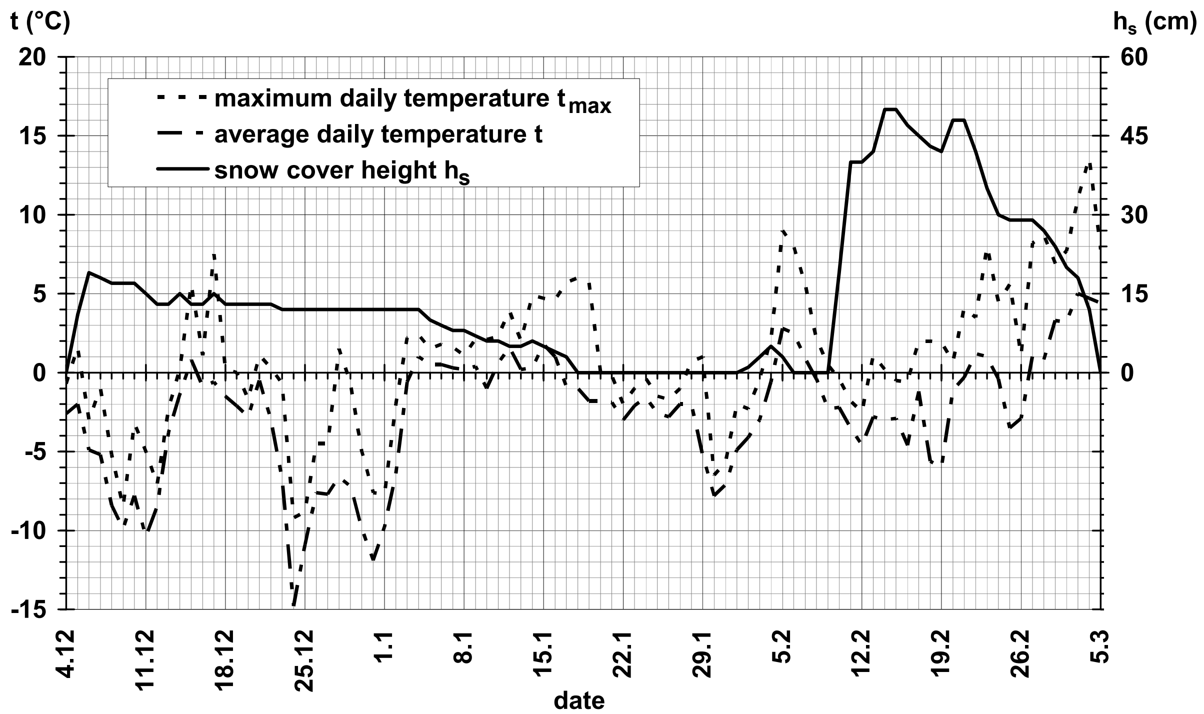

Fig. 1 Course of snow cover height (h_s), average and maximum daily temperature (t), in Somotor 4.12.1998 - 5.3.1999.

For the hydraulic solution of the problem mathematical modelling of the surface water regime in the whole channel system of the drainage basin of the Medzibodrožie region has been introduced. The methodological procedure came out from solution of Saint-Venant equations for unsteady surface water flow in open channels. On base of measured data the mathematical model was calibrated and afterwards used for design of computational scenarios for improving the hydraulic situation in the system. The most significant idea was the renovation of the pumping equipment at the station and its impact on the surface water regime. Very important seemed to be an introduction of preventive pumping on the station, as well. Both of these possibilities were taken into consideration when preparing scenarios for possible improvement of pumping at the station in Streda over Bodrog River. The hydraulic situation in front and behind the station is illustrated clearly in the Fig.2. It demonstrates evidently the differences between internal (drainage channel) and external (the Bodrog river) water levels. There are going up to 5 m. The designed scenarios were based on calibrated situation (Fig.3) where the real pumping rate (Jančina, D.,1984) was involved into calculation.

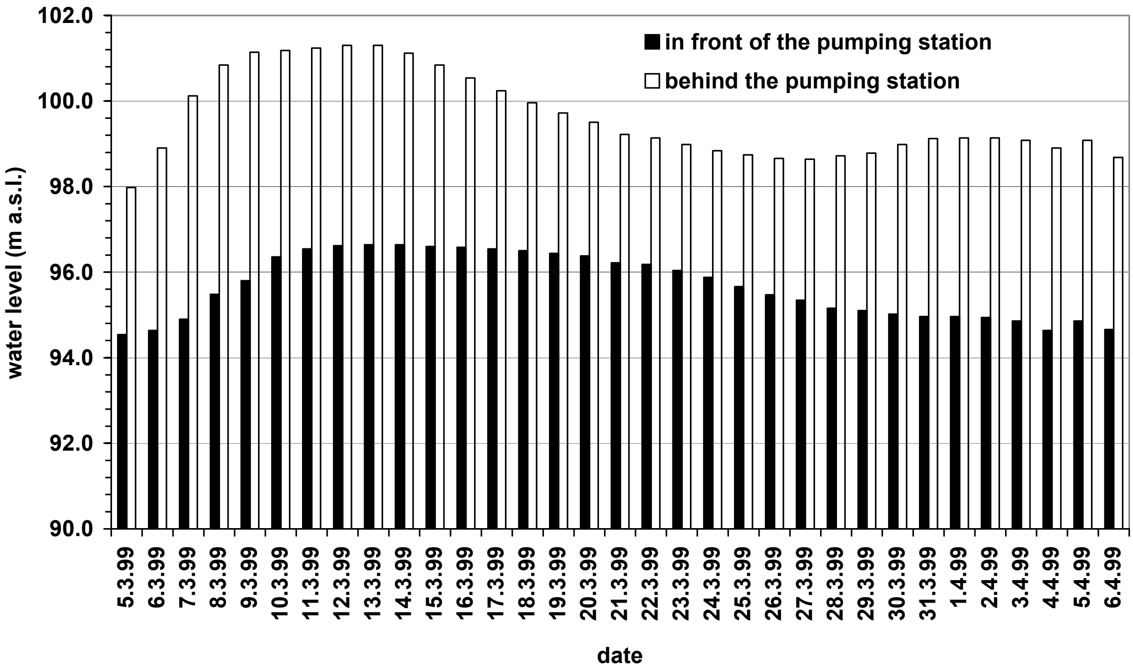

Fig. 2 Surface water levels in front and behind the pumping station in Streda over Bodrog.

First of all, it has to be mentioned that since 2001 new pumping units have been installed into the investigated pumping station. The capacity of the station stayed the same, the installed pumps are smaller and they give the possibility for real preventive pumping with respect to weather development. It means that they can operate more flexibly, in more shallow water as it was before. The next advantage of them is that they can provide continuous draining of the water from the channel at so small slopes which are given in drainage channels.

Next scenarios were based on introduction of preventive pumping into the modelling procedure of the unsteady water flow in the channel for different periods of preventive pumping. They have been changed in three-day intervals, from 3 to 15 days. The most important indicator for the pumping was not the maximum acceptable water level (as it is usual in operation rules) but the weather development. The proof of this fact is the year 1999 as well, where it can be seen from Fig.1 or Tab.1 that the starting date of snowmelt was 21.2.1999, i.e. two weeks before starting of pumping.

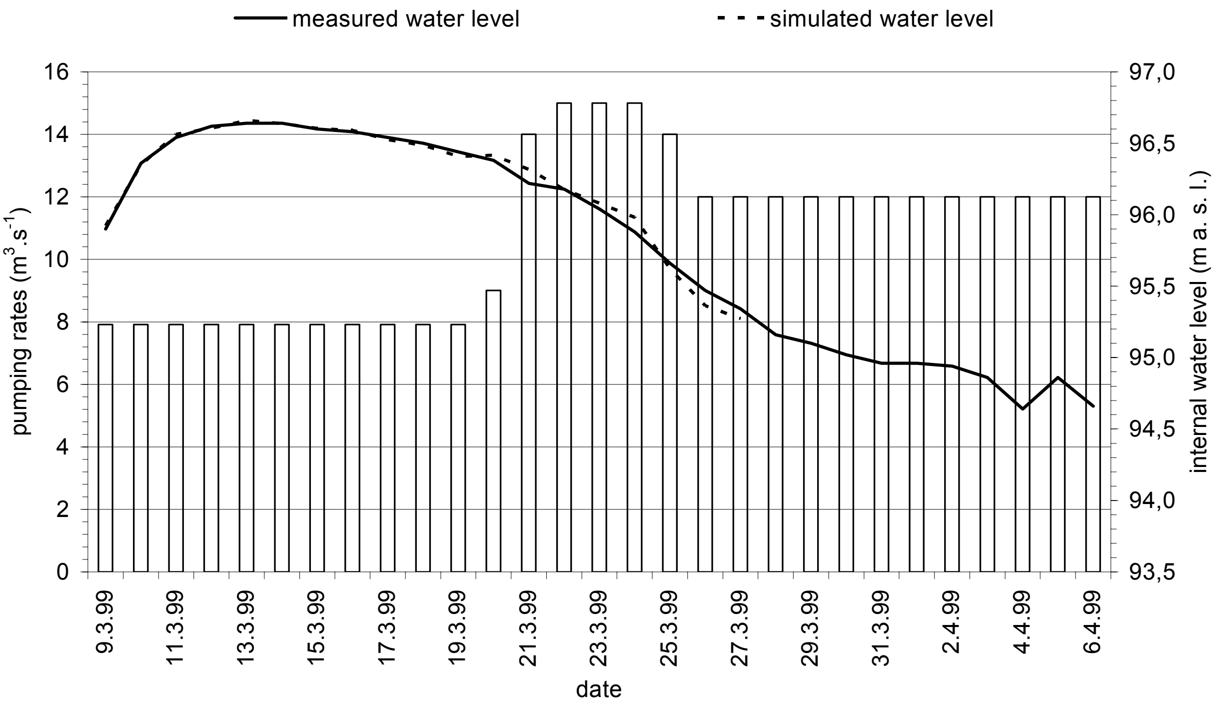

Fig. 3 Calibrated situation in the flood period in the spring time 1999.

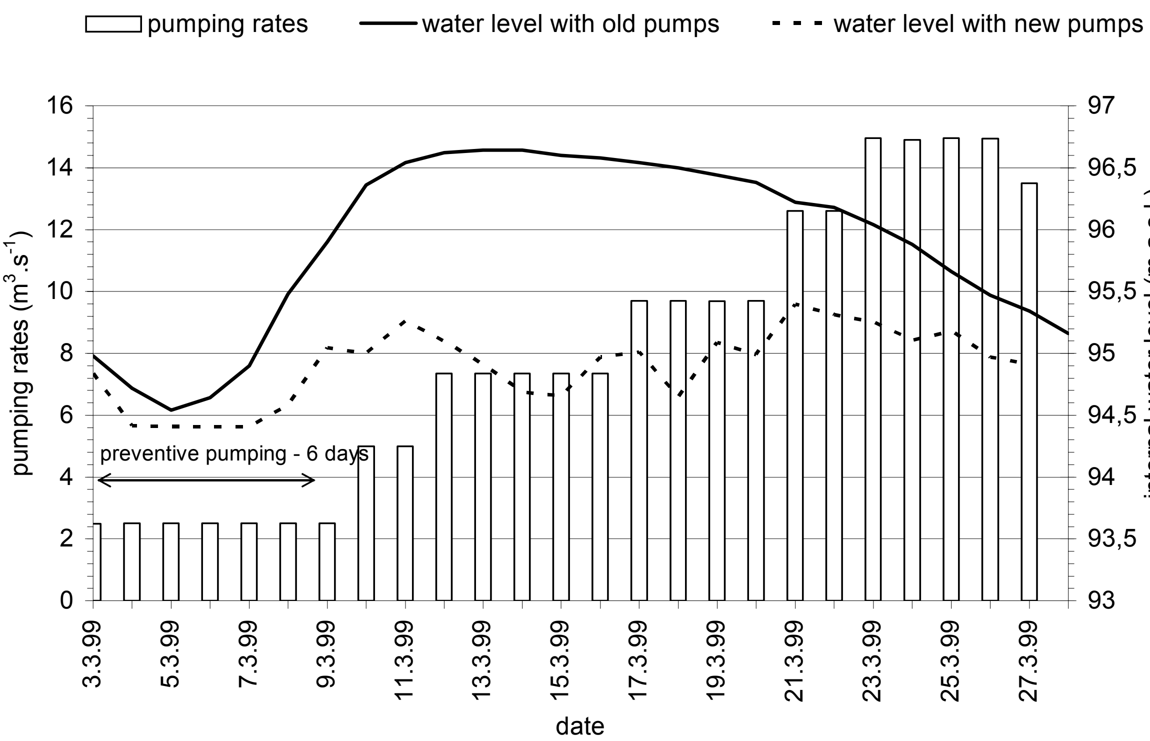

Fig. 4 Internal water course at preventive pumping for 6 days

The idea was not to change the total quantity of pumped water, just to start with pumping earlier with smaller rates (Šoltész, A. et al., 1997). It is apparent that the internal surface water level in the main Somotorsky channel decreased more than 1.5 m in the most critical period and is more convenient to operating rules given at the pumping station (Fig.4). The preventive pumping period in that case was 6 days. The same and slightly better results have been obtained for longer periods of preventive pumping. Cooperation of additional pumping stations with the main pumping station in the same drainage basin will be investigated in the future, as well. It would certainly help to avoid flood events in the basin of the Medzibodrožie region. The opposite function of the channels– irrigation –will be investigated mostly in the summer and autumn period.

3. Conclusion

East Slovakian Lowland is a very complicated river basin because of large areas with heavy (clayey) soils where the runoff coefficients can increase to highest values, especially in winter and spring period. Therefore the most critical period from investigated 20-year hydrological data is mostly situated temporally into the spring time. There are typical natural no-runoff areas where the water usually lies until the late may. Results of the hydrodynamic analysis can lead into better understanding of operation of internal water floods in this region. The cooperation of the research team from the faculty with the operating organisation – Water Board of the Bodrog River – will certainly lead to improvement of the operation rules not only at the observed pumping station but in all drainage river basins of the East Slovakian Lowland.

4. Summary

A detailed hydrologic and hydraulic analysis of creation and drainage of internal water has been carried out for a twenty year period. As the most critical period the flood event in spring time 1999 caused by snowmelt in unfavourable soil conditions (clayey soil) of the region was evaluated. For this period a numerical model of unsteady water flow in the channel system of the Medziborožie region was established concerning the tributaries of the main channel as well as the interaction of surface and ground water flow. Figures demonstrate the results of the modelling and show the advantage of preventive pumping based not only on operation rules but also on weather development. Results of the research team show clearly that the change of the pumping equipment on the pumping station is a step towards a more efficient draining of internal water in the Medzibodrožie region.

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Watershed Modelling as a Part of Geologic Investigations

Zvonimir Vukelic, Petko Pelivanoski
Marija Vukelic-Sutoska

Abstract: Methods of estimating yield from ungaged watershed may be by using of climatic factors, only geographic location and watershed and climatic factors. Methods for estimating yields, as regional analysis, water accounting, direct runoff method, climatic and geographic factors, should be considered as giving estimates so broad that the influence of specific factors have large margins or error. Hydrologic simulation models in watershed modelling use mathematical equations to calculate results like runoff volume or peak flow. Those models can be classified as either theoretical or empirical models. A theoretical model includes a set of general laws or theoretical principles. Further, depending on the character of the results obtained, models are classified as stochastic or deterministic.

In this paper are analysed methods for estimating yields, event versus continous models (empirical model, linear system transfer functions, explicit moisture accounting model-conceptual, physical process model) and generalities of watershed models (hydrodynamics and water quality of land surface runoff, groundwater and freshwater hydrodynamics and water).

Purpose and scope are in procedure and methods for conducting geologic investigations related to sedimentation, engineering geology, and groundwater for work plan development.

Keywords: Geologic investigations, information systems, watershed modeling, watershed yield

1. Introduction

The problem of hydrologic evaluation calls for an understanding of the particular processes which are operating in the watershed. Physical watershed characteristics or meteorological processes, which affect runoff, vary by sections of one country and even by the seasons of the year. For example, when direct runoff from general rainfall in the flood-producing factor, the Soil Conservation Service's (SCS) Hydrology Handbook describes a procedure which may be used. The following steps may be taken as a part of the work plan hydrologic investigations: Analysis of available meteorological and hydrologic data, Surveyers to collect information of stream reaches, channel capacities, and other hydraulic characteristics including structural data for evaluation purposes, Determination of the hydrologic conditions of the water shed taking into consideration soils, land use, topography, cover, geology, and erosion, Determination of rainfall-runoff relationships and frequency of occurances of hydrologic events, Determination of peak discharges under conditions which will exist due to: land-treatment measures, floodwater-retarding structures, etc., agricultural water development, etc.

2. Development of water-related information systems

During the International Workshop held in Washington, DC, USA, 1993, one of the Work Group Recommendations was Group for Minimum data Elements for an International Data Bank that Would Meet Local and Multitional Needs (Jea-Marc Faures, Sung Wong, Jung Sung Kim, Peter Gravesen, Frank Hodgson, Charles Morgan, Zelda Chapmen Bailey) and the Group gave recommendations:

The Group for Mechanism Such as an International Clearinghouse of Hydrogeological Information and How Should Such a Clearinghouse Be Implemented and Administrated (Ed Ongley, Zvonimir Vukelic, Jan-Anne Boswinkel, P. Nieuwenhuysen, Alan Hustard, Jane Thurman, Ashok Shahane) gave the following recommendations:

The work group was unanimous in their opinion that an institutionalized central place for hydrogeological information is needed. Currently the approach that is most commonly used by researchers to acquire hydrogeological information for multinational and international interpretation is to compile data from published sources and to use an informal network of professional colleagues. This approach is not an efficient or effective way to gather the desired hydrogeological data. The individual scientist may not have access or knowledge of the available literature that contains the desired data and network of colleagues may not be broad enough to capture the desired data in terms of scope, spatial and temporal coverage.

3. Geologic investigations for watershed planning

3.1 Sedimentation

Erosion is the detachment and movement of individual particles or masses of materials by various forces such as wind, water, ice, gravity, etc. Water erosion is of most concern in watershed planning work. In Figure (1) are shown the types of damages outlined.

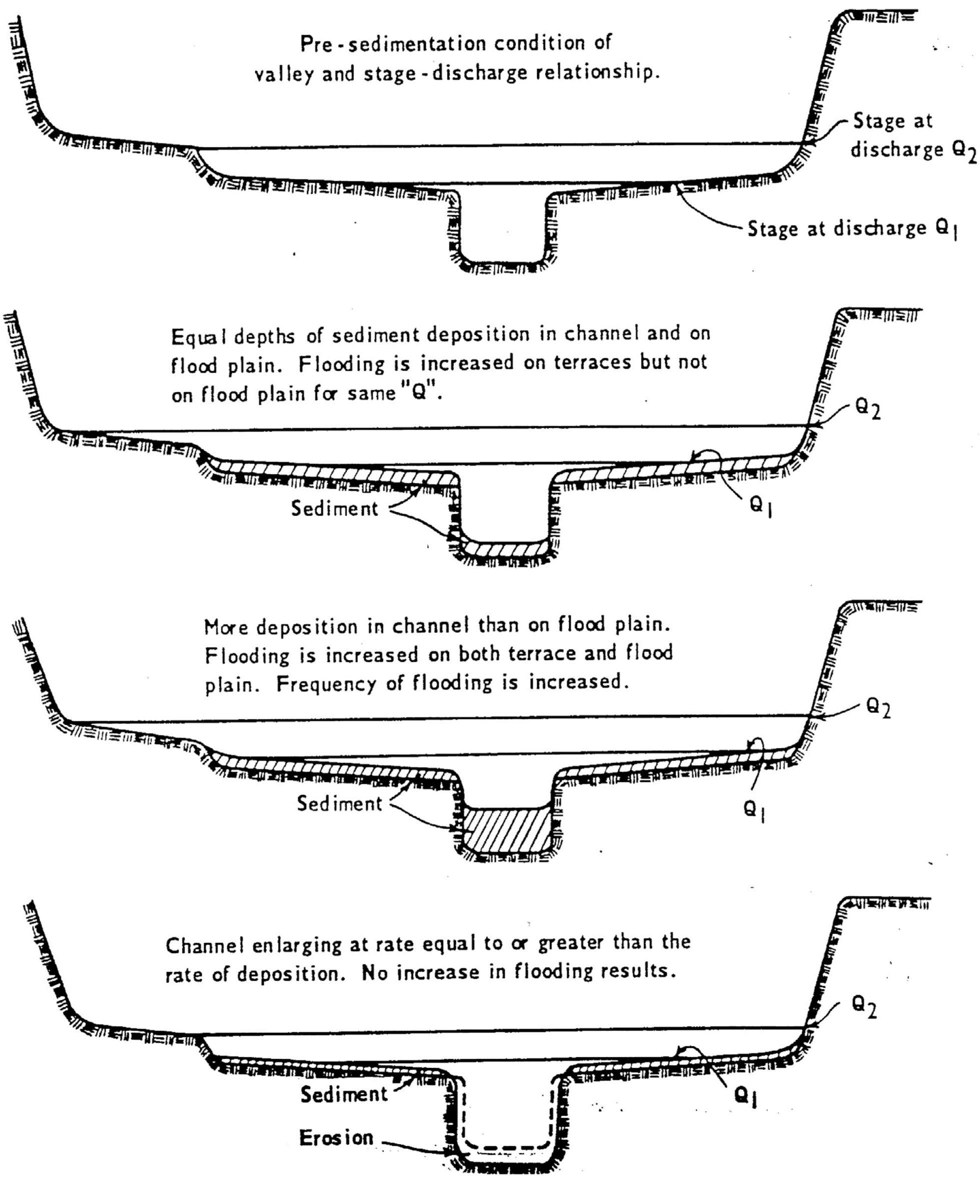

Figure 1 Interrelation between sedimentation and flood stages

Regarding the sedimentation it is of interest to analyse the following topics:

(1) Sediment damage, (2) Erosion damage, (3) Flood plain damage surveys, (4) Types of deposits, (5) Association of the genetic types of deposits. Characteristics of genetic types of valley deposit may be shown in this way: basis of comparison principil origin, usual place of deposit, dominant texture, relative distribution in the valley fill. Types of deposit: colluvial deposits and fluvial deposits (vertical accretion of deposits, splay deposits, lateral accretion deposits, channel-lag deposits, channel-fill deposits, (6) Field procedure for determining flood-plain damages, (7) Detailed sedimentation investigations.

In this topic it is of interest to know percentage of damage to flood-plain lands and estimated recovery as related to depth and texture (depth and texture, damage percent), recovery period (years), damage remaining after recovery (percent)). Estimation of occurring in the beds of streams composed of of noncohesive materials may be made by the use of the following equations: Mayer-Peter-gravel (coarse and fine) and sand (coarse), Schoklitsch-gravel (fine) and sand (coarse and medium), and, Haywood-sand (medium and fine) and silt, (8) Deposition on flood plains.

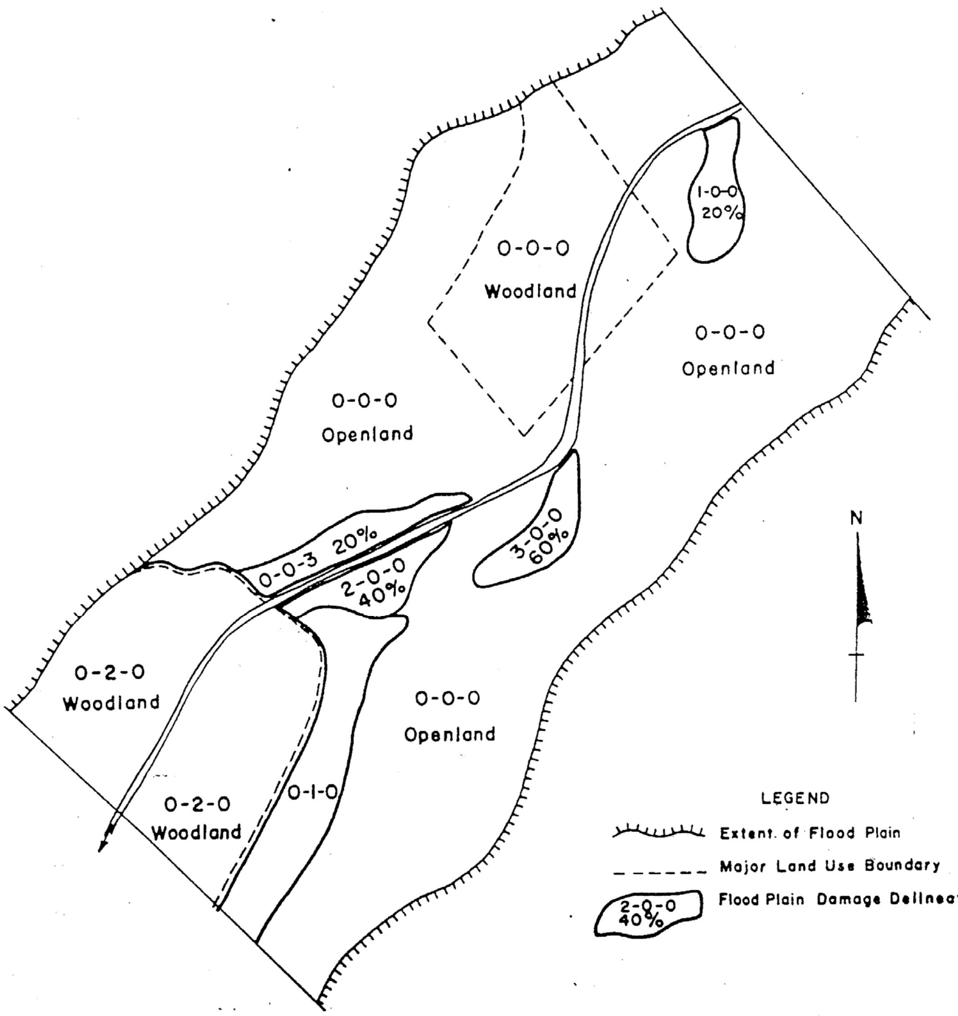

Figure 2 Flood plain damage survey

In Figure (2) is shown one case of flood plain damage survey. There is the following legend: 0-no deposition, no swamping damage, no scour damage, 1-(0-33) percent of the delineated flood plain area covered with damaging sediment, bottomland suitable for cultivation now to wet for crops, may be used for pasture, and no scour damage, 2-33-66 percent of the delineated flood-plain area covered with damaging sediment, bottomland formerly suitable for cultivation now too wet for agricultural use, suitable for timber, 33-66 percent of the delineated flood-plain area scoured, and 3-(66-100) delineated flood-plain area scoured, and 3-(66-100) percent of the delineated flood-plain area covered with damaging sediment, 66-

100 percent of the delineated flood-plain area scored, (9) Determining average annual flood-plain sedimentation and erosion damages, (10) Flood-plain deposition and scour damages (11) Channel erosion damage (distance/years=average annual distance), (12) Swamping damage, (13) Reservoir sedimentation surveys (purpose, detailed reservoir surveys, reporting results, evaluation of reservoir sedimentation damages), (14) Evaluating gully erosion damages (15)

Evaluating of other types of sedimentation damages (water supplies, hydro-electric power, transportation facilities, drainage ditches and irrigation channels, increased flood stages, urban and rural fixed improvements, recreation (16) Sediment sources (channel erosion, sheet erosion), (17) Sediment yields, (18) Relative sources of sediment, (19) Relative sources of sediment, (20) Evaluating effects of the watershed program, and (21) Evaluating of program benefits.

3.2 Groundwater

Usually the geologist will participate in these investigations only when the complexity of conditions requires geologic interpretation. At evaluating such problems as wet basements, impeded sewage disposal, unstable foundations, and impaired crop production may result from a high water table. This may be a natural condition or it may be the result of sedimentation or of a cultural development.

3.3 Geological investigations for structural works of improvement

The geologist also conducts investigations to obtain geologic information needed for the design and construction of structural improvements: site selection, preliminary site investigation, detailed site investigation, geologic investigations for channel improvements, geologic investigations of other Engineering works of improvement, intensity of investigations

4. Watershed yield

The water yield of a watershed, by years or seasons or months, is used in the planning and design of some watershed projects, especially those involving irrigation. Watershed yield is dependent on many factors, most of which usually cannot be quantitatively determined during ordinary field operations. Methods of estimating yield from ungaged watersheds may be classified as follows: (a) using only climatic factors (examples are graphs or equations using precipitation and temperature, or only precipitation, (b) using only geographic location (examples are maps having lines of equal runoff, or the practice of estimating yield by interpolation between gaged watersheds), (c) using watershed and climatic factors (examples are (1) water accounting method, (2) regional analysis, and (3) use of daily rainfall).

The choice of method often rests on the type of runoff to be estimated, which may be classified as: (a) yield as a residual of precipitation after evapotranspiration (examples are watersheds where base flow predominates, water accounting methods are useful with this type), (b) yield as an excess of surface supply over surface intake (examples are watersheds where surface runoff predominates, methods using rainfall and infiltration are needed), and (c) yield as a diverted flow (examples are watersheds having irrigation projects that get their supply outside of the watershed and their return flows occur inside, or watersheds with surface runoff predominating, whose streams carry return or waste flows from irrigation projects or municipal and industrial plants that pump their supplies from deep wells or receive them from outside the watershed).

In practice, the primary factors that can ordinarily be considered for ungaged streams are:

(1) streamflow on nearby watersheds, (2) precipitation, (3) hydrologic soil-cover complexes, (4) evapotranspiration, (5) temperature, (6) transmission losses, and (7) base flow accretions.

5. Methods for estimating yields

5.1 Regional Analysis

(a) Regional analysis (for water yield, the method is used with annual, seasonal, or monthly flows of gaged watersheds. The slopes of the frequency lines will vary, being flattest for annual yields and boceming steeper as smaller divisions of year are used. Transmission losses for example, may be insufficiently detected by this method, and additional field studies may be required to determine those losses.

(b) Water accounting (the flow chart assist in understanding the following steps: (1) obtain soils and land treatment data for the watershed, (2) obtain estimates of the water-holding capacity of each soil or soil group, expressed as inches depth of water between the amounts at field capacity and wilting point, the soil depth for which this capacity is needed is the depth of the intensive root zone, (3) compute the water-holding capacity of the watershed, weighting by areal extent of the soils or soil groups, (4) obtain watershed cover data for the season or seasons for which yields are to be estimated, (5) compute potential evapotranspiration (potential ET), or consumptive use by months for each major crop or land use, (6) compute monthly weighted potential ET for the watershed, (7) obtain monthly rainfall data for the watershed, for a period of years estimated to be long enough to give adequate yield values, (8) compute average rainfall over the watershed, by months, for each year of record, (9) Tabulate rainfall and ET data as shown on Table (4.1), and compute runoff, by months, for each year of record:

- In Table (4.1), the computation starts with a month when available soil moisture is fully depleted, it could start equally well with a month when the soils are fully saturated,
- If there is a break in the year, the first month after the break should have either of the moisture conditions given above,
- When the precipitation is snowfall, convert to water equivalent (watershed average) before using in line 1, watershed consistently having snowfall on one portion and

rainfall on the other should be subdivided and the yields of the subdivisions computed separately, then combined for total watershed yield,

- Work with subdivision if the watershed soils differ in water-holding capacities by more than about 100% of the smallest capacity or by more than about 25 mm , whichever is greater,
- Work with subdivisions if the watershed precipitation consistently varies widely in amount at different localities, this may be determined using average annual precipitation, the variation over a watershed (or subdivision) should not be greater than about 30% of the smallest value, 1-Average over the watershed for each month of record, 2-At start of month, same as "Final soil moisture" for previous month, 3-See text, step 9, notes (a) and (b), 4-Average annual values for the month, 5-Total available moisture, or potential ET, whichever is smaller, and 6-At end of month, same as "Initial soil moisture" for next month, this never larger than the waterholding capacity determined in step 3 of the text in this case,
- After completion of the computations for the selected length of record, test the runoff estimates for adequacy of length of record, the test should be made with values that will be used in planning or design, for example, if annual are to be used, when they are tested, if monthly values are to be used, then all October values are tested separately, next all November, and so on, if the length of record is not adequate, additional years of precipitation are added and the yield computations extended,

(c) Direct runoff method (the direct runoff method is usually very tedious, since all daily precipitation in a long period of record must be accounted for, day by day, using soil-cover complex numbers that vary from month to month or even more often, this method is more suitable for small watersheds than for large ones, since the large watersheds will have some base flow, which may be a significant proportion of total yield),

(d) Climatic and geographic factors (in areas where there is no abrupt change in precipitation, hydrologic soil-cover complexes, or geology, yield may be readily estimated using maps with lines of equal runoff)

6. Watershed modeling

6.1 Classification of Watershed Models

Hydrologic simulation models use mathematical equations to calculate results like runoff volume or peak flow. These models can be classified as either theoretical or empirical models. A theoretical model includes a set of general laws or theoretical principles.

A simplified spectrum of mathematical watershed models is shown in Figure (3).

An empirical model omits the general laws and is in reality a representation of data.

Depending on the character of the results obtained, models are classified as stochastic or deterministic. If one or more of the variables in a mathematical model are regarded as random variables having distribution in probability, then the model is stochastic. If all the variables are considered to be free from random variation, the model is deterministic (even yhough some “deterministic models” may include stochastic processes to add the dimension of spatial and temporal variability to some of the subprocesses, such as infiltration).

Figure 3 Mathematical watershed models (After Linsley)

Increasing physical information and increasing complexity			
Empirical Model	Linear systems transfer functions	Explicit moisture accounting model (conceptual)	Physical process model
Regression Equations FSR (UK)	Unit Hydrograph-SCS methods	HSPF, SHE, WATFLOOD	None available

An event model is one that represents a single runoff event occurring over a period of time, ranging from about an hour to several days. A continuous watershed model is one that operates over an extended period of time, determining flow rates and conditions during both runoff periods and periods of no surface runoff.

Complete or comprehensive watershed models are models for which the primary input is precipitation and other meteorologic data and the output is the watershed hydrograph. The model represents in more or less detail all hydrologic processes significantly affecting runoff and maintains the water balance by solving the continuity equation of precipitation, evapotranspiration and runoff (i.e. hydrologic cycle):

$$\text{Precipitation} - \text{actual evapotranspiration} = \text{runoff} \pm \text{change in storage} \qquad (1)$$

A partial model represents only a part of the overall runoff process.

A calibrated parameter model is one that has one or more parameters that can be evaluated only by fitting computed hydrographs to the observed hydrographs.

A measured parameter model, on the other hand, is one for which all parameters can be determined satisfactorily from known watershed characteristics, either by measurement or by estimation.

Regarding lumped versus distributed models, lumped models do not explicity take into account the spatial variability of inputs, outputs or parameters. Distributed models include spatial variations in inputs, outputs and parameters. In general, the watershed area is divided into a number of elements and runoff volumes are the first calculated separately for each element.

Regarding general models versus special purpose models, a general model is one that is acceptable, without modifications, to watersheds of various types and sizes. A special purpose model is one that is applicable to a particular type of watershed in terms of topography, geology or land use, e.g. an urban runoff model.

For simulation with the HSPF, the basin has to be represented in terms of land segments and reaches/reservoirs. The SHE model from the Danish Hydraulic Institute is a deterministic, distributed and physically based modeling system for simulation of all the major hydrological processes of the land phase of the hydrological cycle.

HEC-1 can stimulate the hydrologic processes during flood events. While primarily for looking at unsready flow in riverine system, the preceding hydrologic studies also include it as a program capable of watershed modeling. Other programs commonly used for this purpose include: MITCAT, SWMM, ILLUDAD, STORM, USGS, DWOPER, etc., for which information is available from NTIS (US National Technical Information Services) or the US EPA.

Table 4.1 Sample computation by water accounting method

Line		**Item**	**X…………. …………V**	**Seasonal runoff**
			2002 - 2003	
1	1	Average rainfall		
2	2	Initial soil moisture		
3		Total available moisture		
4	4	Potential evapotranspiration		
5	5	Actual evapotranspiration		
6		Remaining available moisture		
7	6	Final soil moisture		
8		Runoff		Value

7. Conclusion

The problem of hydrologic evaluation calls for an understanding of the particular hydrologic processes which are operating in the watershed under study. Since so many factors enter into the estimating of watershed yields, and since both the relative importance and quantitative influences of some factors are nearly always unknown, estimates of yield should be conservative, according to the use they will have. The planners and designers who will use the yield estimates will be best able to state the direction and degree of conservative required. Regarding the generalities of watershed models catchment models may include all or some of the following capabilities: model the hydrodynamics and water quality of land surface runoff, model the groundwater, and model the freshwater hydrodynamics and water quality. The first step is to define the problem and determine what information is needed and what questions need to be answered, use the simplest method that will yield adequate accuracy, and do not forget the assumption underlying the model used and do not read more significance into the simulation results than is actually there.

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Groundwater Vulnerability in Western Herzegovina

Mladen Zelenika, Ivan Slišković, Božo Soldo

Abstract: The full and partial hydrogeological barriers together with terrains of low and high permeability, springs and surface watercourses have been mapped. The Mayor karst aquifers are highly vulnerable but the quality of groundwater is reasonably good due to the low development in the high karst, where the chief recharge is occurring. The adequate protection zones in the basins and around the springs should be determined and legalized as soon as possible in order to frame a sustainable development as the policy for protection of the groundwater recharge area.
Keywords: vulnerability, karst spring, aquifer, hydrogeological barriers, ground water protection

1. Introduction

Groundwater vulnerability map is one of the applied environmental maps (Vrana, 1984). It is also one of hydrogeological maps having scarce and heterogeneous data from various sources (Struckmeier, 1989). Vulnerability of groundwater is a relative, nonmeasurable, dimensionless property. The accuracy of its assessment depends above all on the amount and quality of representative and reliable data (Vrba and Zaporozec, 1994). When assessing vulnerability, the attributes or their parameters may be assigned different weights and rating according to their considered importance for the vulnerability assessment. Despite the differences in opinion as to the weight and rating of the individual attributes, it is generally recognized that groundwater vulnerability can be assessed only when the principal basic parameters of the attributes are known. The natural (intrinsic) attributes of primary importance include: the amount and quality of recharge, soil properties, and the characteristics of the unsaturated and saturated zone (Vrba, 1991).

Natural attributes of secondary importance include: topography, contact with surface and/or sea water and the nature of the underlying geological unit of the aquifer. Beside the principal parameters of the natural (intrinsic) attributes, there are the supplemental parameters which, includes: vegetative cover, evapotranspiration, air-temperature, weathering rates etc.

The vulnerability of groundwater in karst aquifers in the region has been assessed using HCS (Hydrogeological complex and Setting methods) with estimation values of 4 basic parameters (Vrana, 1984). The vulnerability has been assessed also using PCSM method - Point Count System Model (Aller et al, 1987) in terms of the risk of the system becoming exposed to contaminant loading. The important parameter in the assessment of specific groundwater vulnerability is the attenuation capacity of the soil, of the unsaturated zone, and of the aquifer with respect to the properties of individual contaminants. Specific vulnerability of groundwater is assessed using the estimated data for: characteristic of the unsaturated zones, average depth to water, hydrogeological features, thickness, textures and mineralogy in the region.

The exploration area (Fig. 1) predominantly comprises the drainage basin of the Mostarsko Blato and the Trebizat river. It includes the Cvrsnica and Cabulja mountains and the right bank of the Neretva river, from Jablanica to Gabela. Identification of karst aquifers and basins in the region is stated in order to determine the protection zones around the springs and the sustainable development as a measure to protect the groundwater recharge area from contamination. The Perm-Triassic sediments have been tectonically uplifted to the land surface at Sobac and consist of gypsum and anhydrites. The undifferentiated Middle and Lower Triassic is composed mainly of dolomite and dolomitic limestone in the region of Cvrsnica Mountain and along the northern slopes of Cabulja Mt. The Jurassic limestone and dolomite are found in the central region of the river Drezanka. The Cretaceous deposits are complete and cover the largest area (Figure 1). Tertiary sediments cover a considerably smaller area. The Quaternary deposits extend along the rivers and in karst poljes (Sliskovic, 1994).

Groundwater potential of this area has been scarcely explored. Although the basic data about major springs and streams were collected during the budgeting of groundwater of the of the river Neretva drainage basin. This includes data on the springs used for water supply of: Mostar, Grude, Studenci Ljubuski etc.. The conclusion is that the considerable amount of unpolluted groundwater could be extracted even during the hydrological minimum in the places where the water supply/demand problem is very acute, eg. the areas of Posusje, Citluk, Grude and the northern and southern edge of Mostarsko Blato (Sliskovic, 1991).

Since the river Neretva is a transboundary watercourse, all appropriate measures should be taken to prevent, control and reduce pollution of waters causing or likely to cause transboundary impact. The groundwater in the Major aquifers of the region facilitates the quality and quantity of water in all surface streams including Neretva. They are sources for majority water supply use: Studenac and Radobolja for Mostar, Grudsko Vrelo for Grude, Listica for Široki Brijeg, Klokun for Klobuk, Crnasnica for Knespolje, Tribistovo for Posusje, Studencica for Studenci, Vriostica for Ljubuski etc. (Figure 1). This paper will discuss the issue related to the vulnerability of the Mayor aquifers in order to determine the protection zones around the springs and contribute to a sustainable development as a measure to protect the recharge area of the major aquifers.

2. Hydrogeological characteristics of aquifers and basins

Western Herzegovina belongs to one of the High Karst zones; the Orogenic Accumulated Karst (Herak, 1977), characterized by the development of specific geomorphologic, geological and hydrogeological features. Hydrogeological functions of rocks in the region related to their permeability, genetic and sedimentation characteristics, as interpreted from previous exploration data. Limestones are predominant rock units in this region. Within them, fissures and other openings are very heterogenous and reflect the tectonic processes that occurred during the Alpine orogeny. Limestones, dolomitic limestones and some dolomites are classified as high-permeable rocks with cavernous and fissured porosity. The permeability of these rocks depends on their karstification, intensity and density of fissures, structural position, stratification, formation thickness and the relative proportions of their calcitic and dolomitic components.

Dolomites of different ages occur from the Lower Triassic to Upper Cretaceous, and range from pure dolomites in the central part of the <u>Neretva</u> river banks, trough a gradual transition to pure limestone in the area of Posusje and Siroki Brijeg. The Upper Triassic dolomites are occasionally karstified but, as a formation, they are impermeable and form barriers to groundwater flows. Dolomite plays a significant role in the formation of water divides in the central Neretva river-banks, in the area of <u>Diva Grabovica</u> and <u>Dreznica</u>. The drainage basin of <u>Veliki Praporac</u> and <u>Mali Praporac</u> (Figure 1) is confined toward the north and south by the dolomites of the rivers <u>Doljanka</u> and <u>Drezanka</u>, while the western border is obscure. The drainage basin of Veliki Praporac and Mali Praporac is separated from the basin of <u>Crno Vrelo</u> by the dolomitic anticline of Diva Grabovica (Figure 1, 2)

Taking into account the hydrogeological and morphological conditions of the region, the area can be divided into seven drainage basins having their aquifers of the Major importance regards to their use and protection.

Boundaries of the drainage basins were determined on the basis of geological structures, hydrological parameters, and water budget, as well as dynamics of permanent and intermittent spring occurrence and their spatial arrangement. Experimental colorings were performed in all major drainage basins. The testing locations include Grudsko vrelo, Studenac, vrelo Radobolje, vrelo Ugrovace, as well as majority of wells used in water supply. Additional hydrogeological, hydrological and geophysical explorations and exploratory drillings have also been completed. Traced paths of groundwater are presented in Figure 1. Full lines mark proven underground connections during measurements and dashed lines mark probable paths of the ground water.

The basins could be shown in the following review:

1 The river Trebizat receives the largest part of its water during a hydrological minimum from Pec Mlini spring (Figure 1) of Tihaljina river, Q_{min} = 1,2 m^3/s and the springs Kordici, Jaksenica and Klokun. Downstream of Klobuk, the appearance of major permanent and intermittent springs is associated with the Ljubuski-Klobuk thrust of upper Cretaceous limestone over Eocene flysh deposits. The largest springs there are (Figure 1): Podgrab, Radišici, Vriostica - Q_{min} = 1,25 m^3/s, Studen (No. 19) and the three springs which are creating the river Studencica (No. 22 to 24).

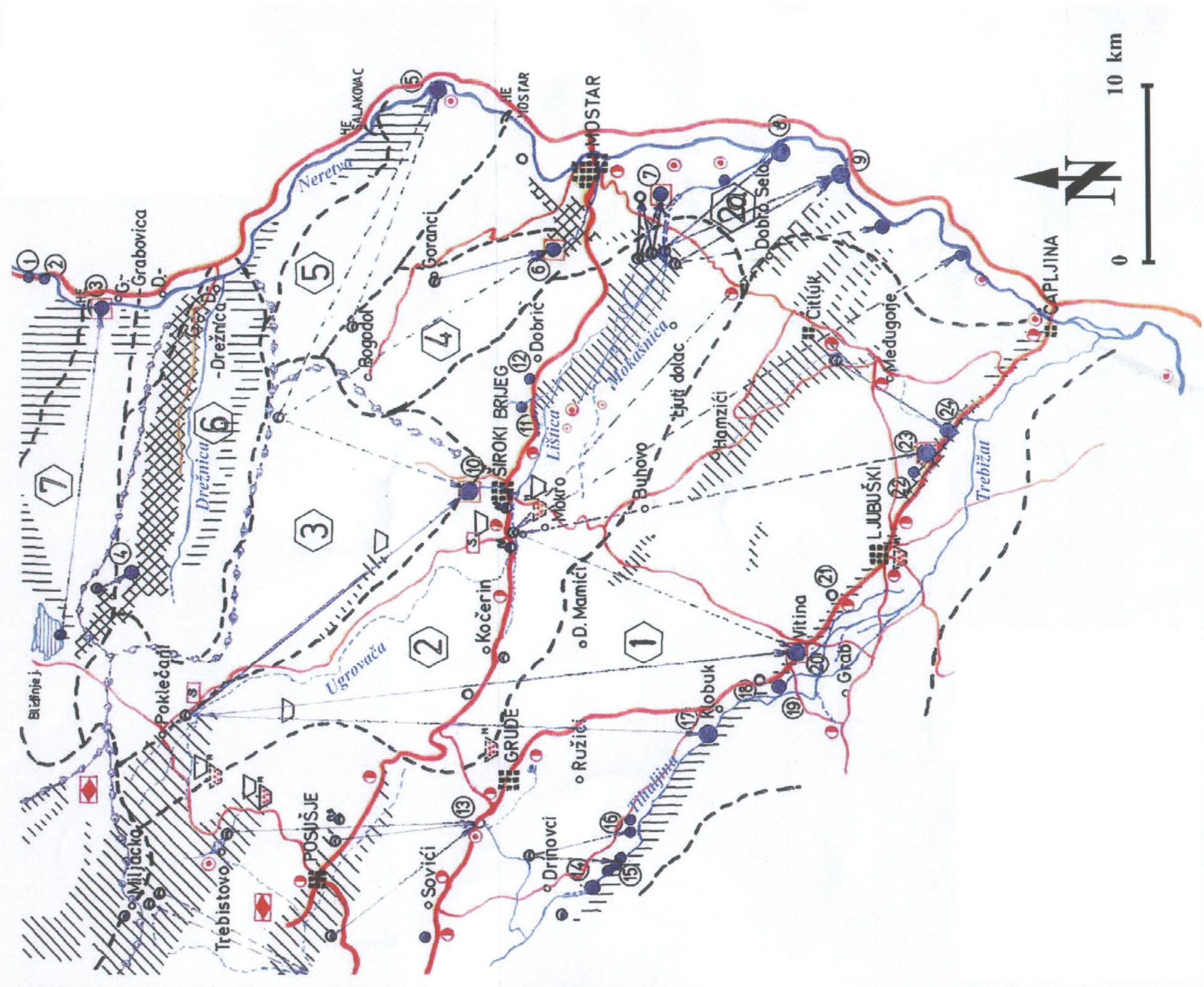

Figure 1 Drainage basins and chief springs in Western Herzegovina (Zelenika, Sliškovic and Muhovec, 1999)

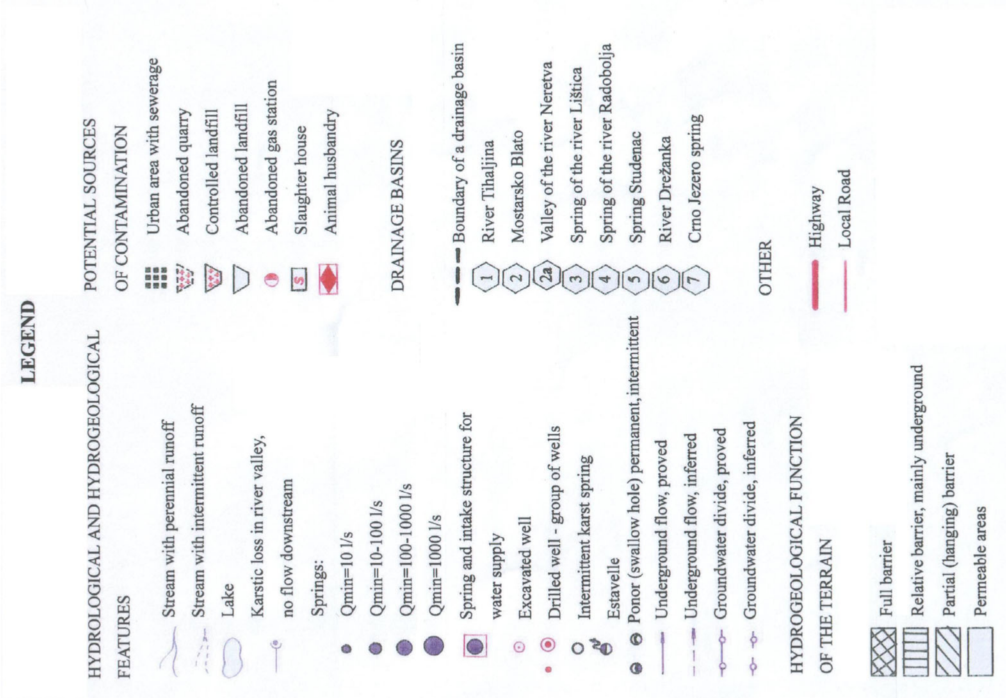

Figure 1A Legend for Figure 1

2 The drainage basin of the intermittent course of river Ugrovača, belongs to the bigger drainage basin of Mostarsko Blato as well as the river Lištica drainage basin.
3 The drainage basin of the Mostarsko Blato, having combined Q_{min} = 3,04 m^3/s from springs, Listica, Crnasnica and Zvatic (No. 10 to 12), gives enough water for the regional water supply system in design and partial use. Exploratory drilling along the northern and southern edges of the Mostarsko Blato, proved occurrence of groundwater in limestones at depths of 20 to 70 m. That is a "transit" area for groundwater flow from Kocerin and more distant recharge areas at northwest, toward springs in the Neretva river valley: Jasenica, Crno Oko and Arape Mlin (No. 7 to 9).
4 and 5 The drainage basin of the spring Studenac (No. 5) and the spring of river Radobolja (No. 6), having combined Q_{min} = 1,4 m^3/s under natural conditions and additional 1,2 m^3/s after filling the hydro power reservoir Salakovac, offer a safe water supply for Mostar and its surroundings.
6 The spring of Drezanka river (No. 4), Q_{min} = 0,150 m^3/s, only 0,002 m^3/s is used for public water supply and 0,1 m^3/s for irrigation.
7 Veliki Praporac (No.1, Figure 1), Q_{min} = 0,4 m^3/s, is used for the water supply of Jablanica town. Mali Praporac (No. 2) and Crno Vrelo (No. 3) flow out free into river Neretva.

Minimal dynamic reserves of springs which contribute their water to Trebizat river (total Q_{min} = 17,36 m^3/s) indicate significant residual reserves in the hinterland of the springs. It makes possible the proven underground hydraulic connection of these springs with ponors in Rakitno (Figure 1), Trebistovo and Posusko polje. Groundwater, flowing from the polje Rakitno, encounters hanging hydrogeological barriers in the areas of Posusko and Imotsko Poljes, resulting in decreasing flow and recharging of the aquifer occurring upstream to the barriers. Therefore, the area from Rakitno to springs is mainly a "transit" one through which groundwater flows from the north and northwest toward its erosion base, in the south and southeast, toward Mostarsko Blato and the river Trebizat, i.e. toward the river Neretva (Sliskovic, 1994).

Jurassic and Cretaceous dolomites are relatively insignificant contributors to the impermeability of rock complexes but, where they form the core of anticlines, they function as barriers to ground water flow. The Lower Cretaceous dolomites of the river Listica spring are of this important type (Figure 1).

Paleogene and Neogene clastic sediments, deposited in basins of different age and origin, are considerably more important factors in the flow of groundwater. During the Illyrian orogeny, the Paleogene sediments were lifted, repeatedly folded and simultaneously overthrusted by Upper Cretaceous deposits, resulting in the inclination of the syncline toward the southwest. The sea bottom subsided during the Lower Miocene, and Neogene sediments were subsequently deposited. Pliocene and Quaternary neotectonic movements produced the current morphology of this area.

By the end of Pliocene and early Pleistocene, certain poljes and plateaus underwent heavy tectonism during which karst poljes were formed without marine or lacustrine deposition. Examples include the Imotsko and Dugo Polje in the north and northwest and the Mostarsko Blato in the south of the area of the study. The karst poljes are at different altitudes in western Herzegovina. From the Rakitsko Polje (950 m above sea level), Duvanjsko Polje (860 m), Posusko Polje (Figure 1) (600 m), Imotsko Polje (250 m) and Mostarsko Blato (240 m above sea level) toward the river Neretva drainage basin at the southeastern side of this area (less than 50 m above sea level).

The drainage area of the lower Neretva river (Figure 1), to the river Trebizat, is very narrow because the drainage basin of the Mostarsko Blato is considered as a separate drainage area. Attempts to define the divide of the Mostarsko Blato (Figure 1) drainage basin have been futile because the underground divide is, most probably mainly zonal. Groundwater flow tracing in this large drainage basin is lacking and more attention should be focused on this problem in the future. In the northern part of the drainage basin, the relationships with neighboring drainage basins are more obvious because the impermeable rocks along the Drezanka river occur in an anticline, thus directing groundwater flow towards the south (Figure 1). The northwestern divide is approximately defined by tracing of the ponor (swallow hole) Miljacka at the Studena Vrela and by the position of hanging deposits north of Poklecani (Figure 1).

Tracing in the Rakitsko Polje proved that the divide with the river Trebizat is wide zone. Within this expressly karst area, the Upper Cretaceous dolomites are often permeable because they occur within imbricated structures. The Upper Cretaceous anticline, in the Siroki Brijeg area, is the reason for the appearance of the major karst spring of the Listica river (Figure 1). Water from the aquifer in northwestern area flows out at this spring. Groundwater may flow in the dry season from this aquifer toward the springs of the river Tihaljina, thus explaining the existence of the intermittent river Ugrovaca by which most of surface water flows toward the Mostarsko Blato during the rainy season (Komatina, 1975).

Additional tracings are necessary to prove to which drainage basin the groundwater of minor Neogene poljes, Kocerin, Trn and Mokro Polje pertains. At present, it is most logical to consider that this water belongs to drainage basin of the springs: Crno Oko, Arapa Mlin and the Jasenica river spring in Neretva river valley downstream from Mostar (Figure 1) (Sliskovic, 1994).

Paleogene and Neogene deposits in the west of the study area form a significant barrier. In addition, narrow Paleogene zones extending from river Neretva towards the northwest act as hanging barriers to groundwater flow. The best known barrier occurs at the front of the Ljubusko-Klobuk overthrust and it extends along the left bank of the Tihaljina river (Figure 1). The area from Mostar to Citluk and Hamzici is composed of hanging barriers, which extend in several zones up to the Duvanjsko Polje. Impermeable rock units occur at high levels and act as hanging barriers to groundwater formed as an aquifer in the deep karst.

3. Aquifers in the region and their vulnerability

In the circumstances of the most intensive recharge of the karst aquifer by precipitation in the higher areas with very limited sources of pollution, the present quality of groundwater in the region is good. Lower parts of the region are populated and fortunately have more overlying clay to protect the aquifer. The soil and groundwater there, could be contaminated by run-off water percolated from landfill sites for domestic and industrial wastes, roads, urban areas, farms, storage of manure and slurries from waste water, agricultural land treated by fertilizers and pesticides, and by accidental spillage from leaking oil, fuel and from septic tanks.

The scheme on Figure 1 shows the mayor aquifers in the areas of the chief catchment basins in the region. The Major aquifers occur there in Cretaceous limestones and dolomite, which extend from Cabulja Mt. (1683 m), Stitar Mt. (1365 m) and Studena vrela (Ostrc 1309 m) to Gabela (15 m above sea level). Saturated parts of this karstified formation in the contact with the impermeable Tertiary or/and Quartenary formations, are the most important perennial springs utilized for urban and rural water supply (Fig. 1): Jelica (Ugrovaca 980 m above sea level), Listica (300 m, Crnasnica (240 m), Zvatic (240 m), Tihaljina (120 m), Klokun (110 m), Vriostica (100 m), Studenci (60 m) and intermittent springs: Grudsko vrelo (260 m), Kocerin (300 m), Mokasnica (255), Lukoc (300 m), Blaz (245 m), Gromolj, (250), Zelenikove Babe (255 m), Zvec (245 m) etc.

The Minor aquifers occur in Tertiary limestones (Dobrinj, Zabljak, Orovnik, Soldino vrilo, Konjsko vrilo etc.), and in Quartarnary clastic layers (wells at Mostarsko Blato, Kocerinsko, Mokro, Imotsko-Bekijsko polje etc.).

The chemical composition of the karst springs reflects the comparatively high homogeneity of carbonate rocks, mostly limestones, and increased velocity of groundwater flow. The results of water samples taken from different springs show no specific contaminants in drainage basins. The groundwater in the region is still of a satisfactory quality (Zika & Muhovec,1999)). The total dissolved solids content (200-300 mg/l) is lowest in springs with rapid circulation of groundwater. Intermediate contents (300-450 mg/l) total dissolved solids content occurs in the springs of Crnasnica and Trebizat. The springs in Tihaljina drainage area have increased mineralization. It is characteristic for Klokun, Kordici, Jaksenica and Nezdravica springs located in the upper parts of the river Tihaljina. These are in full agreement with the hydrogeological conditions as observed in the filed. It positively established that groundwater during its flow from Trebistovo to the Tihaljina river, in the area northeast of Posusje, come in contact with the Sobac Permian-Triassic structure. The increased contents of dissolved sulphate in water at springs of the rivers Listica and Crnasnica, indicating deeper circulation of groundwater, even up to Lower Triassic clastic sediments, and prolonged storage within the aquifer (Sliskovic, 1994).

4. Discussion and conclusions

Drainage basins having Major karst aquifers and springs captured for water supply in the region are shown in Figure 1. The poljes in the region have clay and sand layers of the lower permeability where landfill, waste water treatment plant and some kind of industrial or other contaminants could be located. In the marginal parts of poljes are usually available ponors connected with groundwater, and watercourses convenient to carry contaminants to pollute surface waters, including the river Neretva. Therefore adequate protection zones should be defined and legalized as soon as possible.

An additional comprehensive hydrogeological investigation should take place in order to define the policy of a sustainable development and use of land in the high karst of the region. There occurs the most intensive recharge of all 7 drainage basins/major aquifers, and special precautions should be taken to avoid pollution of groundwater in the region. Therefore the establishment should set up an Environment Agency to coordinate a working relationship among the governments authorities, scientific and professional institution, and chief business corporations including industry and agriculture. This Agency should identify those activities presenting the greatest risk of pollution to groundwater, but which are currently not well regulated in order to develop appropriate controls, including necessary legislation. The Agency should provide a better focus for the prevention of pollution of natural resources in the upper region and downstream.

Karstification is considerably deeper than the altitude of the sites where groundwater discharges into the rivers Neretva, Tihaljina and Listica. This was discovered from the boreholes at the southern edge of Mostarsko Blato, in the Imotsko Polje and at Salakovac and Studenac in the river Neretva valley (Sliskovic, 1994).

Protection of groundwater in karst region against pollution is different from that in unconsolidated water bearing formations. In karstified rock rain water in the rule, enters into terrain carrying its suspended part of the contaminating substances, and these rocks usually have no straining or filtrating capacity. Therefore, little filtration occurs during the flow in such kind of aquifer. On the other side, the velocity of the flow is much higher, even several hundreds or thousands of meters per day. Therefore, the protection zones should be determined after an extended investigation.

Since groundwater flows trough fissures, cracks and caves mostly along faults which are crossed by other fissures caused also by faults or other tectonic disturbances, there is a net of many fissures, some extending to the surface, through which water may receive fresh pollution from surface. These facts call for a special and strict regime of protection of ground water in karst aquifers. It is recommended to avoid construction of any industrial structure discharging polluted effluents in the recharging area of aquifer. There should be prohibited discharge of any toxic substance in the soils, caves or ponors. Waste water should be treated and controlled before discharge in the stream or at the soil.

In the circumstances of the most intensive recharge of the karst aquifer by precipitation in the higher areas with very limited sources of pollution, the groundwater in the region is yet non-contaminated. The lower parts of the region are populated and more developed, fortunately there are more overlying clay to protect the groundwater in aquifer**. The** quality of groundwater in the chief springs of the region is still satisfactory. Springs 15, 16 and 17 in the upper part of the Tihaljina river are an exception and their water must be treated before the use in water supply. A comprehensive hydrogeological investigation should take place in order to define the policy of a sustainable development and use of land in the high karst region. where the most intensive recharge of all 7 drainage basins/major aquifers occurs. Special precautions should be taken to avoid pollution of groundwater in these highly vulnerable aquifers.

In the region should be established a reliable laboratory for a regular control of water quality in water sources and in observation wells. Beside standard control bacteriological and aesthetic quality, the laboratory should be equipped for a regular control of radioactive materials, Pesticides, Trihalomethans, Benzene, Chlorophenol, and inorganic constituents of health significance, like: Arsenic, Cadmium, Chromium, Cyanide, Phenols, Fluoride, Lead, Mercury, Nitrate and Selenium.

It could be recommended to the establishment to set up an Environment Agency to coordinate a working relationship among the governments authorities, scientific and professional institutions, and chief business corporations. This Agency should identify those activities presenting the greatest risk of pollution to groundwater, but which are currently not well regulated in order to develop appropriate controls, including necessary legislation. The Agency should provide also a better focus for the prevention of pollution of natural resources in region and downstream.

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The Numerical Simulation of Two-Dimensional Vertical Incompressible Viscous Flow

Piotr Zima

Abstract: The paper presents the numerical simulation of two-dimensional vertical incompressible viscous free surface flow. The model is based on the Navier-Stokes equations. To solve governing equation the pressure method (SIMPLE) was used. In this case the divergence of Navier-Stokes equations are considered and Poisson equation on pressure is obtained. To solve this problem the Lax-Wendroff scheme with staggered Euler grid was applied. This approach ensures second order accuracy of numerical scheme. The Poisson equation was solved using the Successive Over-Relaxation (SOR) method. The domain (where the fluid is present) was defined using Marker and Cell (MAC) method. In order to assess the accuracy of the models same simple test cases were analyzed. The results of laboratory experiments presented by S. Koshizuka et al. were used to verify the calculations. Collapse of a water column is analyzed using the presented method.
Keywords: two-dimensional vertical incompressible viscous flow, Navier-Stokes equation, water splash effects, numerical simulation

1. Introduction

In many hydraulic engineering problems the free surface rapidly varied flows are encountered, especially including water jets, hydraulic jumps, flow over and under gates, collapse of water wall. Numerical simulations of these problems are often made with the Navier-Stokes equations and continuity equation (mass conservation law). The Navier-Stokes equations (NSE) are the system of partial differential equations describing incompressible viscous flow (Sawicki, 1998). Unfortunately, an analytical solution of NSE, in most real cases, does not exist and they must be solved using numerical methods. A lot of classical methods for solving the water flow equations are well known and successfully applied

(Fletcher 1991, Anderson 1995, Tannehill 1984). They are based on the three fundamental methods: finite difference method (FDM), finite element method (FEM) and finite volume method (FVM) (Gryboś, 1998). One of the most popular and often used method is SIMPLE method (Semi Implicit Method for Pressure Linked Equations)(Potter,1982). In this algorithm we consider a divergence of NSE. It leads to the Poisson equation which describes pressure field evolution (pressure-correction equation). SIMPLE method will be used to solve two-dimensional vertical NSE on rectangular, staggered grid.

2. Governing equations

Governing equations for incompressible viscous flow are the continuity equation (1) and NSE (2) in the following form (Sawicki, 1998):

$$\mathrm{D} = \nabla \mathbf{u} = 0 \tag{1}$$

$$\frac{D\mathbf{u}}{Dt} = \mathbf{f} - \frac{1}{\rho}\nabla p + \nu\Delta\mathbf{u} \tag{2}$$

where: D – velocity divergence, **u** – velocity vector, **f** – external forces vector, p –pressure, ρ - density, ν – kinetic viscosity factor.

Pressure-correction equation can be written:

$$\Delta\overline{p} = -\nabla\cdot\left(\mathbf{u}\cdot\nabla\mathbf{u}\right) \tag{3}$$

where: $\overline{p}$ – normalized pressure, $\overline{p} = p/\rho$.

These equations are solved for the horizontal and vertical velocity variables u_x and u_y and the pressure p. The external body forces vector includes the acceleration due to gravity g with components ρg, in the directions of the x-y cartesian coordinate system. We can rewrite above equations in differential form for both velocity components:

- continuity equation

$$\mathrm{D} = \frac{\partial u_x}{\partial x} + \frac{\partial u_y}{\partial y} = 0 \tag{4}$$

- momentum equations

$$\begin{cases} \dfrac{\partial u_x}{\partial t} + u_x\dfrac{\partial u_x}{\partial x} + u_y\dfrac{\partial u_x}{\partial y} = g_x - \dfrac{1}{\rho}\dfrac{\partial p}{\partial x} + \nu\left(\dfrac{\partial^2 u_x}{\partial x^2} + \dfrac{\partial^2 u_x}{\partial y^2}\right) \\ \dfrac{\partial u_y}{\partial t} + u_x\dfrac{\partial u_y}{\partial x} + u_y\dfrac{\partial u_y}{\partial y} = g_y - \dfrac{1}{\rho}\dfrac{\partial p}{\partial y} + \nu\left(\dfrac{\partial^2 u_y}{\partial x^2} + \dfrac{\partial^2 u_y}{\partial y^2}\right) \end{cases} \tag{5}$$

- pressure-correction equation

$$\frac{\partial^2 \overline{p}}{\partial x^2}+\frac{\partial^2 \overline{p}}{\partial y^2}=-\left\{\left(\frac{\partial u_x}{\partial x}\right)^2+2\left(\frac{\partial u_x}{\partial y}\right)\left(\frac{\partial u_y}{\partial x}\right)+\left(\frac{\partial u_y}{\partial y}\right)^2\right\} \tag{6}$$

For all test cases presented in this paper no turbulence model is incorporated into solution and surface tension effects are neglected.

3. Numerical methods

Equations (3) and (4) will be solved by FDM using SIMPLE method. The main idea of this method is splitting the governing equations solution in two steps:

- first step is prediction of the velocity field integrating equation (4), the explicit scheme is used to obtain values of velocity components with values of pressure from previous time step,
- second step is computation of the pressure-correction field, which then corrects the velocity to satisfy the zero divergence condition (4).

To determine the free surface location the flow domain (where the fluid is present) is defined using Marker and Cell (MAC) method (Welch, 1965).

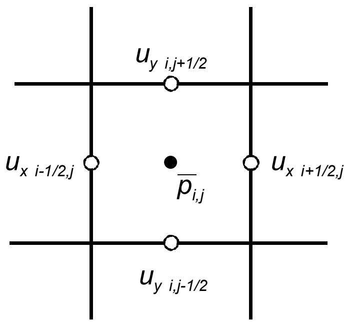

Fig. 1 Cell specification for Euler staggered grid

To integrate the NSE system (4) and Poisson equation (6) in space by FDM the two-dimensional domain *z-x* should be discretized into a set of cells. To make this discretion the Euler staggered grid was used. In each cell variables u_x, u_y and p are located in different places (see Fig.1). There are four types of cells (Fig.2): full (F), boundary (B), surface (S) and empty (E).

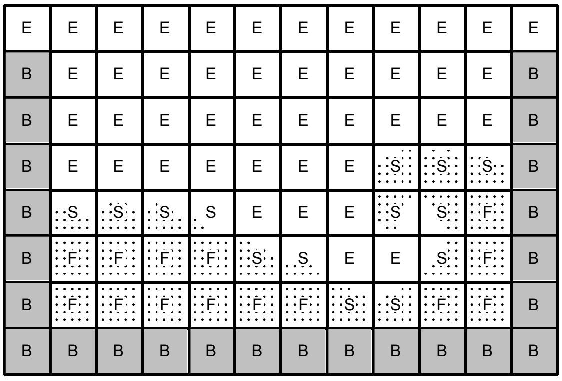

Fig. 2 Notations for cells in MAC method

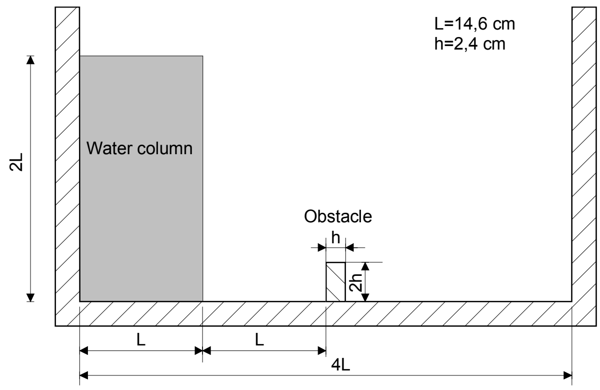

Fig. 3 Geometry of domain and initial shape of water column

The velocities are specified at the cell faces while the pressure is specified at the cell center. To discretization NSE on a staggered grid the Lax-Wendroff scheme was applied. This approach ensures second order accuracy of numerical scheme (Potter,1982). The pressure-correction equation is solved using the Successive Over-Relaxation (SOR) method. The boundary conditions for these equations and their solution stability conditions must be satisfied. The initial velocity field **u**, pressure $\overline{p}$ and the initial domain fill (initial position of the free surface) are specified by the initial conditions. The velocity terms are explicit, computed using known values, but the pressure term is implicit, based on the unknown pressure values at the next time step. The position of the markers in cells for the new time level is computed using the corrected velocity **u** and Newton's second low.

4. Numerical simulations and experimental verifications of two-dimensional vertical free surface flow

The numerical algorithm presented above was verified for typical Computational Fluid Dynamics (CFD) test case: collapse of water column (Dam-Break Flow - DBF) (Mohapatra, 1999). There were considered two problems: simulation with and without the obstacle on the domain bottom. Geometry of domain and initial position of water column is presented in Fig.3. The results of laboratory experiments carried out by S. Koshizuka et al. (1995) were used to verify the calculations. In the experiment, the box made of glass with the scale L=14.6 cm was used. The water column is supported by a vertical wall, which was drawn up rapidly (approximately 0.05 sec) for the beginning of collapse. Shapes of water column was recorded by a video camera. The numerical simulation was made with following parameters: number of cells 1891 ($dx = dy = 0.973$), number n of particles 512, 1152 and 2048. Gravitation unit $g_y = -9.806$ m/s^2 and kinetic viscosity factor $\nu = 1.0 \times 10^{-3}$ were imposed. Viscous effect on the boundary was neglected.

The first presented problem is a simulation of collapsing of water column. The results of experiment and computing (particle locations and velocity vectors) for the example calculations are presented in Fig 4 (test case without obstacle on the bottom).

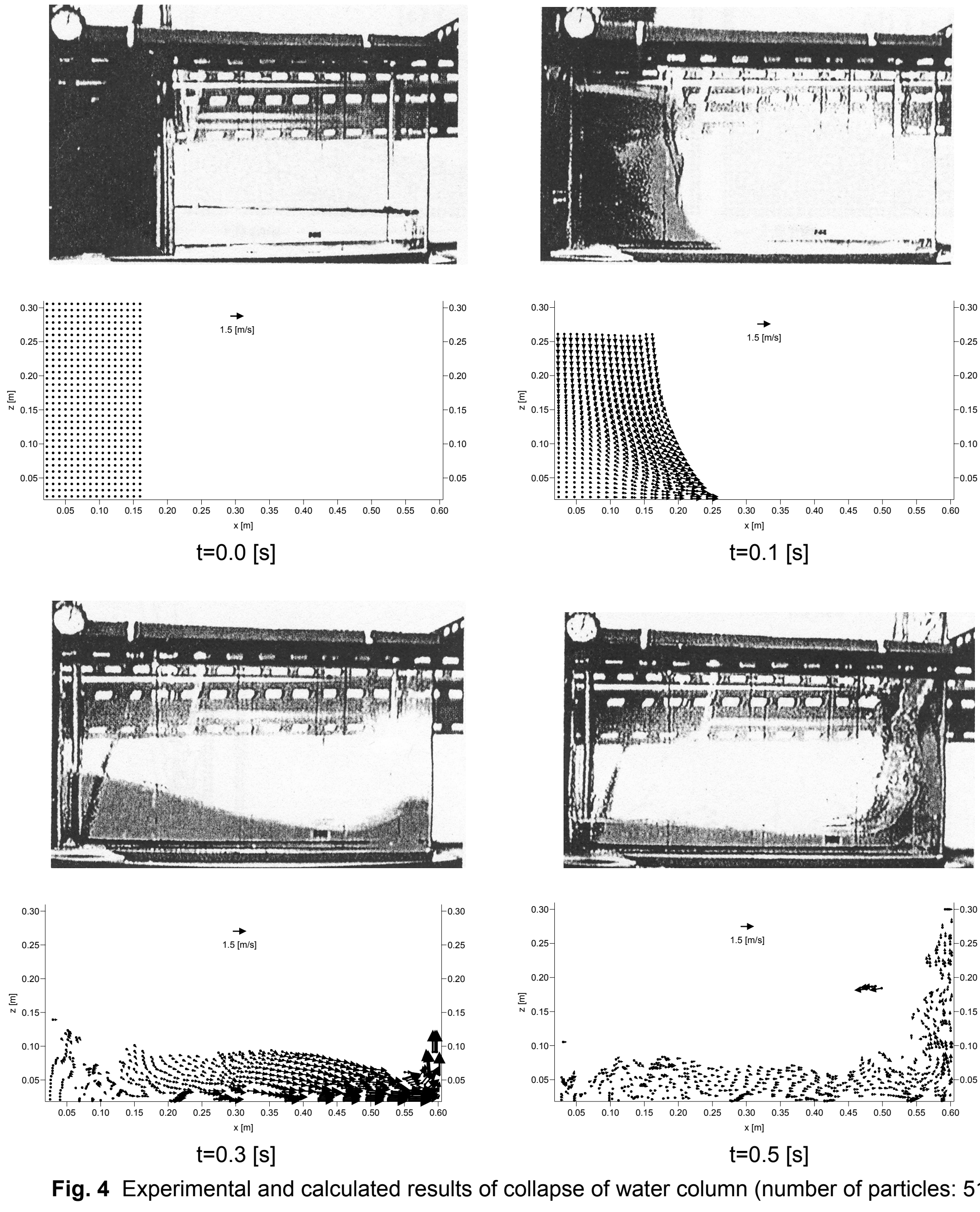

Fig. 4 Experimental and calculated results of collapse of water column (number of particles: 512)

The position of the water wave front of collapsed water column is shown in Fig 6. The experimental and computing results (maximal position of particles in relation *x/L*) are shown on the normalized time ($t_n=t\sqrt{2g/L}$) background. The significant disagreement comparison is observed to t_n=2.0. To this time, the delay between the calculated and observed wave front location is evident. The reason is lack of bottom friction (the viscous effect was neglected). After t_n=2.0 the conformity of experimental and calculation result is quite good. There is no dependence on number of particles *n*.

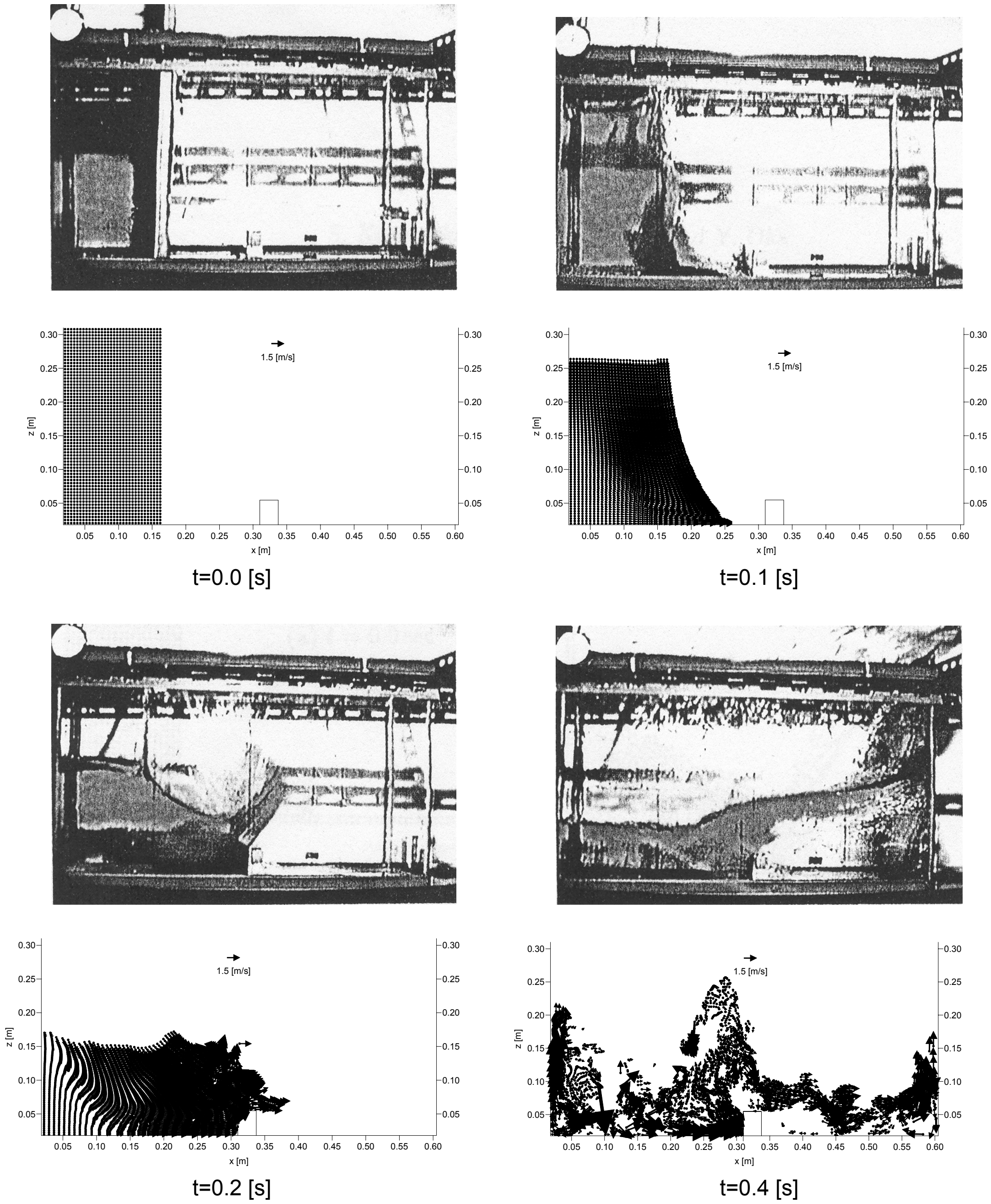

Fig. 5 Experimental and calculated results of collapse of water column (n = 2048) with an obstacle

The collapse of water column with obstacle was considered as a next test case. A obstacle is located on the bottom wall at *2L* far from the left wall. The size is *hx2h*, where *h* = 2.4 [cm]. Comparison of the calculated and experimental results are presented in Fig.6. After time t=0.2 [s], the run water crashes on the obstacle and splashed toward the upper-right direction. In the calculation result, the water is splashing more horizontally. The reason of this difference is probably neglecting of reaction between air and water on the free surface (assumption – the pressure is constant on the free surface).

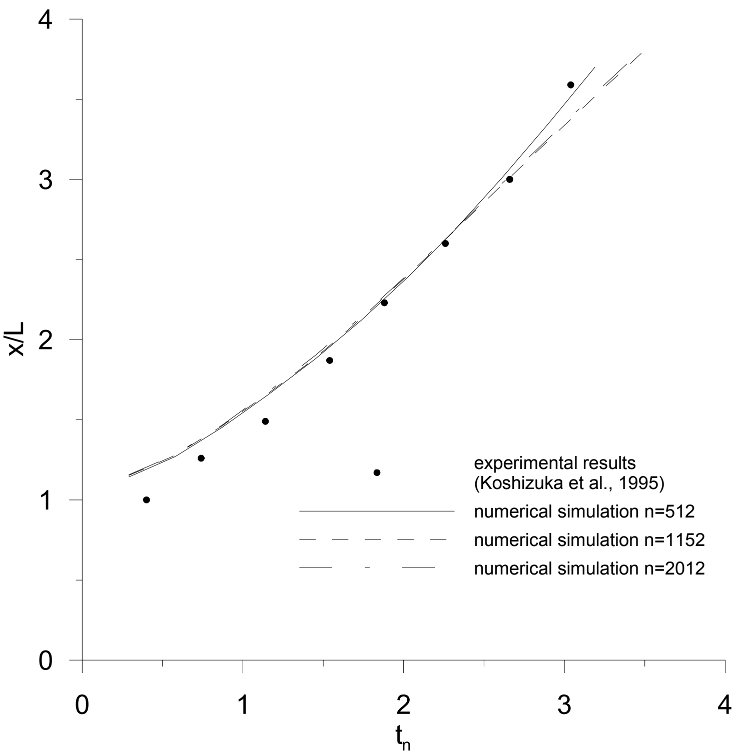

Fig. 6 Location of front of water wave - experimental and calculated results

5. Summary and conclusions

In this paper, the numerical simulation of the collapse of the water column (standard test case in CFD) was considered. This numerical simulation is based on the results obtained using two-dimensional vertical flow equations (NSE) that allow the computation of the velocity and pressure fields. The numerical solution is based on SIMPLE algorithm. The FDM with staggered Euler grid was applied. The pressure-correction equation was solved using the SOR method. To define the free surface and spatial arrangement of fluid the MAC method was used. The results of laboratory experiments carried out by S. Koshizuka et al. were used to verify the calculations.

Following are the conclusions drawn from presented in this paper study:

- presented method can be applied to numeric solutions of many hydraulic problems where the strong deformation, splashing or rapidly varied free surface is occur;

- presented numerical calculations results of collapse of water column confirm suitability of this model to two-dimensional vertical analysis of the DBF mechanics, especially for dry-bed conditions.

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Theme VI

River Restoration Projects
(Strategies and Experiences)

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Synthesis Assessment of Urban River Corridors: Ecohydrological, Urban Structure and Utilitarian Viewpoints

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Abstract: Within the scope of the project URBEM (Urban River Basin Enhancement Methods) (http://www.urbem.net/), financed by the European Commission as part of the 5th Framework Programme for the implementation of the Water Framework Directive (Directive 2000 / 60 / EC), in cooperation with the experts from Great Britain, Germany, Austria, France, Portugal and Slovenia, the proposal of the method for the synthesis assessment of urban river corridors status has been developed. The main aim of the method is to assess the quality of the visual environment of urban rivers for the determination of river reaches, which should be prioritized for the implementation of restorative and rehabilitative measures. The method is based on three groups of viewpoints: "river" (ecohydrology), "city" (urban structure), and "people" (user's perception), which provide a basis for the synthesis assessment of the aesthetic value of urban rivers. In Slovenia, the method was tested on three urban rivers in Ljubljana urban area whose river corridors were intensively modified by urbanization, however different regarding stream orders and river typology: on the Ljubljanica River, Mali Graben River and Glinscica River. The application of the method has shown that certain elements and procedures should be enhanced and adapted to the needs as determined by the management of urban river rehabilitation.
Keywords: urban river corridors, synthesis assessment, visual environment, ecohydrology, urban structure, user's perception

1. Introduction

Environmental goals of the European Water Framework Directive (hereinafter called: WFD) (The European Parliament and the Council, 2000), adopted by the European Community in 2000, foresee the achievement of the Good Ecological Status of all surface water bodies by the year 2015. Goals should be achieved by the way of applying the measures, such as restoration or rehabilitation works. In the context of the modern care of the river environment, rivers in urban environment areas should be considered as a special category. European project URBEM (Urban River Basin Enhancement Methods) (http://www.urbem.net/), funded by the European Commission in the 5th Framework Programme for the implementation of the WFD and developed in co-operation of partners from 5 EU countries (Great Britain, Germany, Austria, Portugal and Slovenia), has aimed to prepare a tool for the needs of municipal administration in the decision-making process in relation to renewal and rehabilitation schemes of urban corridors.

As a part of the project, the partner from Portugal has in co-operation with other partners in the project drawn up a draft method for a combined expert and survey assessment of the aesthetic value of urban rivers (URBEM, 2004). The aim of the method is to provide a synthesis assessment (ecohydrological, spatial and utilitarian) of urban river corridors in order to facilitate decision-making when prioritising the approach to rehabilitation of urban rivers. In comparison to the existing methods for valuation of the hydromorphological state of rivers, which are based on the assessment of the anthropogenic alterations or ecological deficit of the hydromorphological process in the river corridor, the URBEM method provides an assessment of the river corridor in a wider sense: ecological, spatial and social. In Slovenia, the method was tested on three urban rivers in the capital city of Ljubljana. In the course of testing, several strengths as well as weaknesses of the proposed method were identified.

2. The method

The aim of the method for the assessment of the aesthetic value is to establish the value and potential of aesthetics of urban river reaches in order to identify the priorities and possible approaches to their rehabilitation or restoration (e.g. interventions into the aquatic environment, which would on one hand help mitigate water ecosystem degradation and on the other hand improve its ecological state). The draft method is based on three dimensions: “River“, “City“, and “People“, which are separately assessed and evaluated according to the state of viewpoints (“Fundamental viewpoints” and “Elementary viewpoints”), they consist of. Combined they provide a basis for the assessment of the aesthetic value of urban rivers (URBEM 2004).

Two main spatial units have been considered in the method: river corridor and riverfront. The river corridor is defined as the area that contains both sides of the stream with a width of approximately 500 m on each side, corresponding to about a 10-minute walking access to the water, rather than a landscape ecology category. Local and site-specific corrections to this theoretical limit are advisable. Another important area is the riverfront, i.e. the area between the river and the first line of buildings, including these buildings. The identification of the riverfront area is important from the aspect of relationship or interconnectedness between the river and the city.

The performances of the dimensions with respect to the “Fundamental” and “Elementary viewpoints” (Tables 1 to 3) are measured through the proposed indicators (descriptors) and standardized to the common scale of performance. Simple linear functions are used to convert real scales to a common scale that varies from 0 (the worst plausible level) to 100 (the best plausible leve l). The final result of the method is a profile of aesthetic performance for a selected river reach which enables a further analysis of overall performance of the river reach or an investigation of performance of selected dimensions.

2.1 Dimension “River”

The dimension “River” is delineated by the “Fundamental viewpoints”: River Morphology, Biological Components and Natural and Technological Hazards, with the corresponding s “Elementary viewpoints”. The viewpoints of dimension “River” are given in Table 1.

Table 1 Dimension “River“

*Fundamental viewpoint “River Typology“ does not influence the aesthetic performance of the river.

Fundamental viewpoint	*Elementary viewpoint*	*Code*
*River Typology**	Basin size Stream order River width Valley morphology	*R1* *R2* *R3* *R4*
River Morphology	Degree of disturbance of the natural dynamics Sinuosity Bank shape Presence of hydromorphological elements in the channel	*R5* *R6* *R7* *R8*
Biological Components	Biological diversity Presence of riparian vegetation in the river banks Width of riparian vegetation Presence of different type of vegetation species	*R9* *R10* *R11* *R12*
Natural and Technological Hazards	Flood vulnerability Bank erosion and landslide risk	*R13* *R14*

Table 2 Dimension “City“

Fundamental viewpoint	*Elementary viewpoint*	*Code*
Urban Space Quality	Visual Permeability: Visual contact Depth of views Width of views Density of landmarks Built space quality Public utility of riverfront Intensity of construction	*C1* *C2* *C3* *C4* *C5* *C6* *C7*
Cultural Heritage	Cultural heritage	*C8*
Activities	Diversity of uses Attractiveness of riverfront	*C9* *C10*
Accessibility	River crossings: Bridges Use of bridges Surface of parking Public transport Walkways and bikeways Level of disruptionAnchorage places Use of river by boats	*C11* *C12* *C13* *C14* *C15* *C16* *C17* *C18*
Pollution	Pollution	*C19*

2.2 Dimension “City”

The dimension “City” is characterized by the “Fundamental viewpoints”: Urban Space Quality, Cultural Heritage, Activities, Accessibility and Pollution. Within the dimension “City” the relationship between the built urban space with the water body is identified; the viewpoints are shown in Table 2.

2.3 Dimension “People”

The dimension “People” is characterized by these fundamental viewpoints: Public Perception, Place Identity and Restorative Capacity.

Table 3 Dimension “People“

Fundamental viewpoint	*Elementary viewpoint*		*Code*
Public Perception	In relation to the River	Aesthetic	*P1*
		Water	*P2*
		Biodiversity	*P3*
		Flood risk	*P4*
		Pollution	*P5*
	In relation to the City	Urban quality	*P6*
		Accessibility	*P7*
		Security infrastructure	*P8*
	Relation People-River	Relax	*P9*
		Attachment	*P10*
Place Identity	Continuity		*P11*
	Self-esteem		*P12*
	Self-efficacy		*P13*
	Distinctiveness		*P14*
Restorative Capacity	Being away		*P15*
	Fascination		*P16*
	Extent		*P17*
	Compatibility		*P18*

3. Case studies

3.1 The Ljubljanica River (study reach 2,600 m)

The catchment area of the Ljubljanica River comprises 785.9 km^2. According to the Strahler stream ordering system, it is a 3rd order stream. The average width of the active cross section and the average bankful width range is between 20–200 m. The valley morphology type is asymmetric.

The Ljubljanica River study area includes a large part of the old city centre of Ljubljana with a high density of buildings. Numerous spatial activities and uses connected with the Ljubljanica River have been developed in the areas around the river. The river has been used as an important transport line, port, entertainment area, market place, and also as a conduit for sewer and refuse. To reduce the flood risk, a diversion channel was excavated in the period from 1772 to 1780 between the Castle hill and the hill of Golovec according to the plan of a Jesuit, Gabriel Gruber. In the 19th century, the Ljubljanica River and surrounding areas provided a continuously attractive social space. Later, the regulation and deepening of the Ljubljanica River channel was carried out in the reach of the river through the Ljubljana city centre. The image of the river changed drastically between 1913 and 1918, when banks on the river section through the city were heavily reinforced with high concrete walls. The plan for regulation was developed by engineer Alfred Keller. The natural Ljubljanica River channel was transformed into a ditch, which alienated the river body from the city life. The monotony of the river channel regulation was changed by architect Jože Plečnik in the 1920's and 1930's (Figure 1).

Figure 1 The Ljubljanica River through the city center

3.2 The Mali Graben River (study reach 3,750 m)

The catchment area of the Mali Graben River comprises 154.3 km^2. According to the Strahler stream ordering system, the section of the Mali Graben is a stream of 1^{st} order. The average width of the active cross section and the average bankful width are in the range of 5–20 m. In terms of morphology type, the stream is a broad floodplain along most of the course. Although the Mali Graben was intensively regulated, mainly to assure the conveyance of a discharge up to 170 m^3/s, the hydraulic conductivity is not sufficient. Therefore, the surrounding areas are often flooded.

In the past, the course of the Mali Graben was situated apart from the urban area of the city of Ljubljana. Due to the fast development of the city that eventually grew into an important cultural, political and economic regional centre, the Mali Graben became the boundary between the managed urban space and the green urban space on the periphery. The densely built-up areas are located mostly to the north of the river, directed towards the city centre. During the last two decades, the urbanization spread to the right bank of the Mali Graben. As anticipated, these areas of Ljubljana will face further building expansion (Figure 2).

Figure 2 The Mali Graben River

3.3 The Glinscica Stream (study reach 2,150 m)

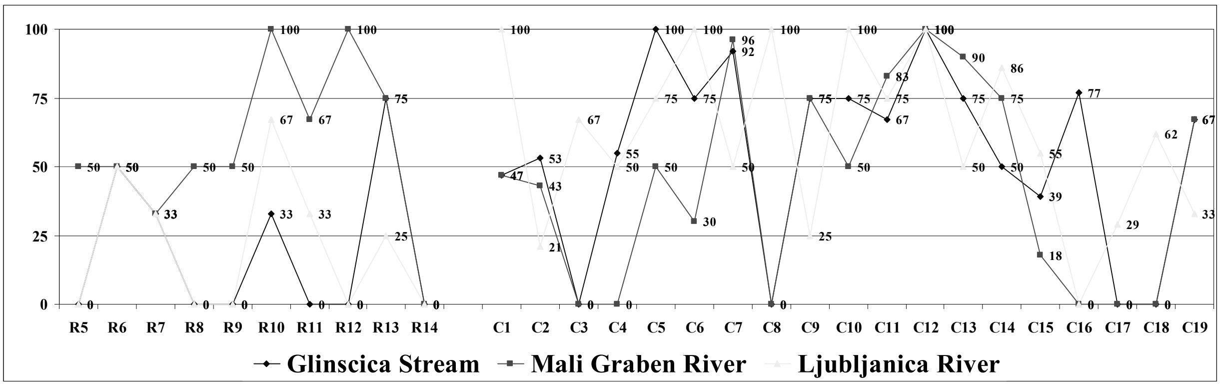
The catchment area of the Glinscica Stream comprises 19.3 km^2, according to the Strahler stream ordering system, the Glinscica is a 1st order stream. The average width of the active cross section and the average bankful width are classified in class 0–5 m. In terms of morphology, the valley profile type is mainly a large broad floodplain, except at the joint of the Glinscica corridor with the slopes of the Rožnik hill. In the downstream part of the study area, the river corridor is more densely urbanised.

The Glinscica Stream has its source under the northeastern slopes of Toško čelo and at Podutik passes into the plain area of the Ljubljana Plain. The topography of the basin is comprised of a hilly area to the east and west and a plain area that spreads out in the southern part. The relief of the Glinscica drainage basin is versatile, comprising hilly headwater areas as well as plains. The precipitation watershed area of the Glinscica comprises 17.4 km^2. The position of the runoff within the urban area is determined by the removal of rainfall water by way of a sewage system, thus the orographic barrier fails to coincide with the Glinscica drainage. The total drainage area of the Glinscica up to its outlet into the Gradaščica is somewhat bigger and comprises 19.3 km^2 of the catchment area. There are an estimated 38 % of urban areas, that is 6.6 km^2 (Figure 3).

Figure 3 The Glinscica Stream

4. Results

The results of the application of the method in three test reaches on the Ljubljanica River, Mali Graben River and Glinscica Stream are shown in Graph 1. In the continuation the performance of the test reaches with consideration to dimensions of “River“ and “City“ is discussed. The performance of the dimension “People” has not yet been evaluated for all three case studies, thus it is not included in this paper.



Graph 1 Profile of aesthetical performance for the Ljubljanica River, the Mali Graben River and the Glinscica Stream (100–best, 0–worst; R-dimension “River”; C-dimension “City”)

4.1 River morphology (viewpoints R5–R8)

The degree of the disturbance of natural dynamic processes in the Ljubljanica River corridor is high due to the high building density in the city centre (Figure 1). The river channel was deepened several times, the banks are reinforced with almost vertical concrete walls. On short subsections, the banks are grassed above the concrete walls. In the study reach, some individual trees and lines of trees are present on the top of the banks. Inside the narrow city centre, the riparian vegetation is completely absent. The local sinuosity of the river was diminished with the regulation works. Along the entire study reach, the cross section is of trapezoidal shape with unchanged slope and bank arrangements. The hydro-morphological elements of the natural river channels (runs, pools, riffles, weirs, asymmetric cross sections) were removed from the stream channel during the regulation works in the 19th century. The average performance of the fundamental viewpoint “River Morphology” for the Ljubljanica River is 20.8 %.

The degree of disturbance of natural dynamic processes on the Mali Graben is high in the upper reach of the study area, where the river corridor is disconnected by the traffic infrastructure (Figure 2). In the lower, maintained and managed study area, the degree of disturbance is moderate. The bottom of the river is natural, the banks are technically arranged, however, they are densely grassed and covered in riparian vegetation. The index of sinuosity of the stream is 1.15, calculated according to the method proposed; the local sinuosity is further diminished due to past regulations. Characteristic of the entire study reach is the trapezoidal cross section with steady slope and bank formation. The hydro-morphological elements (runs, pools, riffles, rocks, weirs) were partly removed from the channel. During the restoration works in the 1980's several weirs were built into the channel, which changed the morphological structure of the channel bottom (emergence of pools) and the river course structure. The average performance of the fundamental viewpoint “River Morphology” for the Mali Graben River is 45.8 %.

The degree of the disturbance of natural dynamic processes in the Stream Glinscica corridor is high and remains mainly unchanged along the entire study reach (Figure 3). The bottom of the Glinscica Stream channel is paved with concrete plates, the river banks are technically arranged and grassed. The index of sinuosity of the stream, calculated according to the proposed method, is 1.13. The local sinuosity of the Glinscica channel is further diminished due to past regulations. Along the entire study reach, the cross section is of trapezoidal shape with unchanged slope and bank arrangements. The hydromorphological elements of the natural river channels (runs, pools, riffles, weirs, asymmetric cross sections) were removed from the stream channel during the regulation work. The average performance of the fundamental viewpoint “River Morphology” for the Glinscica Stream is 20.8 %.

4.2 Biological components (viewpoints R9–R12)

The Ljubljanica River corridor has undergone intensive regulation works combined with the development of the urban tissue in the entire study reach. In some subsections, buildings were situated right next to the river channel. As mentioned, the riparian vegetation mainly includes sparse trees; only short subsections of the river channel are partly shaded. The width of the riparian vegetation is in a range of 0–12 m. The variety of species of the riparian and aquatic vegetation is very low. The average performance of the fundamental viewpoint “Biological components” for the Ljubljanica River is 25 %.

Even though the river corridor of the Mali Graben River has undergone intensive regulations, the riparian vegetation on the study reach remained well developed, ranging in a width of 12–20 m. During the vegetation period the channel is strongly overgrown and shaded (Figure 2). The variety of species (aquatic, amphibian and terrestrial vegetation) is high. The average performance of the fundamental viewpoint “Biological components” for the Mali Graben River is 79.3 %.

The biological diversity of the Glinscica Stream corridor was highly disturbed during the regulations (Figure 3). The riparian vegetation was almost entirely removed. In the downstream section of the study reach, the banks have slowly become overgrown with vegetation, which is advancing from the private gardens along the Glinscica channel. The variety of species of riparian and aquatic vegetation is low. The average performance of the fundamental viewpoint “Biological components” for the Glinscica Stream is 8.3 %.

4.3 Natural and technological hazards (viewpoints R13–R14)

Flood vulnerability of the surrounding urban areas in the study area was high in the past. Intensive regulations of the Ljubljanica River channel (widening of the cross section, deepening, introduction of the water barrier) and the excavation of the Gruber channel (diversion of the water away from the city centre) have diminished the flood vulnerability. We estimate that the area of a 100-year flood event spreads over 25 % of the urbanised part of the river corridor. Bank erosion processes and landslide risks are not present in the study reach. The average performance of the fundamental viewpoint “Natural and technological hazards” for the Ljubljanica River is 12.5 % (performance is measured in reversed scale).

Flood vulnerability of the urban areas in the Mali Graben River corridor is high. It has been estimated that more than 75 % of river corridor areas are 100-year flood events. No bank erosion processes and landslide risks have been identified. The average performance of the fundamental viewpoint “Natural and technological hazards” for the Mali Graben River is 37.5 % (performance is measured in reversed scale).

Flood vulnerability of the urban areas in the Glinscica Stream corridor is high. We estimate that the area of a 100-year flood event spreads over 75 % of the urbanised part of the river corridor. There are no bank erosion processes and landslide risks present in the study reach. The average performance of the fundamental viewpoint “Natural and technological hazards” for the Glinscica Stream is 37.5 % (performance is measured in reversed scale).

4.4 Urban space quality (viewpoints C1–C7)

The visual permeability of the urban space along the Ljubljanica river is characterised by longitudinally and transversally oriented visual axes, which were designed in detail. The linear density of the visual intersections is 11 visual intersections / km of river length. The average length of a visual axis is 100 m. An important landmark in the study area is the tower of the Ljubljana castle on top of Castle hill. High quality constructions (residential, business and commercial) with developed sewage and rainwater drainage system are characteristic. The rainfall runoff from the urban area drains directly into the Ljubljanica River. Poor quality constructions are to be found in some areas of the old city centre where several old buildings require reconstruction. Footpaths are arranged along the entire study reach on both sides of the river channel. The green system of the city of Ljubljana is especially well developed in the upper part of the study reach (area called Trnovski pristan), before the river enters the narrow city centre in the middle part of the study reach. In the downstream part of the study area, traffic and parking surfaces prevail along the channel. The average performance of the fundamental viewpoint “Urban space quality” for the Ljubljanica River is 66.1 %.

The built-up area on the left bank of the Mali graben River is characterised by urban visual axes. The linear density of the visual intersections is 4 visual intersections per km[1] of river length. The average length of the visual axis is 200 m. There are no typical belvederes in the study area, nor any landscape points. Characteristic of the study area on the left bank of the Mali Graben and to the north, is quality housing in private ownership and commercial areas in the central part of the study reach with proper public utility infrastructure. The rainfall runoff is diverted into the river. Individual buildings with poor quality public utility infrastructure spread mostly on the right bank of the Mali Graben and in the area south of the river. Poor quality urban environment is also in the upper part of the study area with a dense traffic network. The Path around Ljubljana as an important element of the urban design runs parallel to the left bank of the Mali Graben and adds to the amenity value of the area. In general, the state of the green system within the study area is good. The average performance of the fundamental viewpoint "Urban space quality" for the Mali Graben River is 38 %.

The urban and suburban space along the Glinscica Stream is characterised by long and wide visual axes of open, mainly non-urbanised areas in the upper part of the study reach. In a more densely urbanised lower part of the study reach, the urban visual axes are narrower and shorter. The linear density of the visual intersections is 4 visual intersections / km of river length. The average length of the visual axes is 250 m. There are no typical belvederes in the study area, the buildings of the Biotechnical Faculty and the Department of Biology feature as landscape points (landmarks). For the downstream section of the study reach, quality residential housing in private ownership with good sanitary conditions is characteristic. The sewage system is well developed. The rainfall runoff is diverted into the Glinscica channel through the surface water drainage system. The upper part of the study area is rural, the meadows along the stream are in private ownership. The path around the city of Ljubljana as an important part of the urban design is in public property. In the study area, the state of the green system is good. The average performance of the fundamental viewpoint "Urban space quality" for the Glinscica Stream is 60.3 %.

4.5 Cultural heritage (viewpoint C8)

In the part of the old city centre, which is directly connected with the Ljubljanica River, the cultural heritage is extremely abundant and attractive. It draws numerous inhabitants of Ljubljana, daily commuters and tourists every day and all year long, especially in the summer time. Cultural heritage undoubtedly contributes to extremely high aesthetic value of the study area Performance of the fundamental viewpoint "Cultural heritage" for the Ljubljanica River is 100 %. There is no element of cultural heritage present within the river corridors of the Mali Graben River and the Glinscica Stream, therefore the performance is 0 %.

4.6 Activities on the riverfront (viewpoints C9–C10)

In the upstream part of the Ljubljanica River study area, a partially urbanized use of the riverfront area prevails. The green system of the city of Ljubljana is well arranged and enables access to the Ljubljanica River water body. In the area of the city centre, there is a diversity of urban activities with predominantly urbanized use of riverfront. The attractiveness of the entire study area is high due to numerous possibilities of spatial uses and activities (footpaths, bikeways, cultural and social events, tourist activities). The performance of the fundamental viewpoint “Activities on the riverfront“ for the Ljubljanica River is 62.5 %.

Urban activities prevail in the upper study area of the Mali Graben River. The lower study area is an open suburban space with an abundance of green areas. The amenity value of the riparian areas is highest on the left bank and to the north of the river due to the characteristically high residential quality and recreational possibilities (footpaths, bikeways), and the Path around the city of Ljubljana. The quality of the riverfront areas is poor in the upper part of the study area due to the traffic network, and on the right bank in the area to the south. The performance of the fundamental viewpoint “Activities on the riverfront” for the Mali Graben River is 62.5 %.

In the lower part of the Glinscica Stream study area, urban activities prevail. The upper part of the study area is open suburban space with the dominance of green area. The attractiveness of the riverfront area is high in the lower, more densely urbanised area, and also in the upper, open suburban area due to the quality of the residential area and recreational possibilities (footpaths, bikeways, ZOO, botanical gardens, river crossings, path around the city of Ljubljana). The performance of the fundamental viewpoint “Activities on the riverfront” for the Glinscica Stream is 75 %.

4.7 Accessibility (viewpoints C11–C18)

There are 9 river crossings in the study area of the Ljubljanica River (six for automobile traffic and three for pedestrians and cyclists). The network of public transport lines is well developed. Tourist navigation with boats of all sizes is organised on the river. The performance of the fundamental viewpoint “Accessibility” for the Ljubljanica River is 57.1 %.

Out of 10 river crossings along the Mali Graben River study area, 7 are intended for automobile traffic, 1 for rail traffic, 2 for pedestrians and cyclists. Well-maintained footpaths and bikeways are characteristic for the area north to the river. The bridges of the south by-pass, rail, Tržaška Road, Cesta v Mestni log Road and Barjanska Road are the most disruptive elements in the river corridor. With regard to the size of the Mali Graben, the navigaton on the river is not possible, also there are no anchorage points. The performance of the fundamental viewpoint “Accessibility” for the Mali Graben River is 45.8 %.

In the Glinscica Stream study area, there are 6 river crossings (two bridges for automobile traffic and 4 for pedestrians and cyclists). Near the Biotechnical Faculty, parking lots are arranged next to the Glinscica channel. The public traffic route passes along Cesta na Brdo Street and crosses the Glinscica channel in the lower part of the study reach. In the upper part of the study reach, there is a well-planned arrangement of footpaths and bikeways along the Glinscica channel. The most disruptive elements in the river corridor are bridges on Cesta na Brdo Street and Brdnikova Street in the uppermost and lowermost sections of the study reach. Because of the size of the Glinscica Stream channel, navigation on the river is not possible and there are no anchorage points. The performance of the fundamental viewpoint “Accessibility” for the Glinscica Stream is 51 %.

4.8 Pollution (viewpoint C19)

The Ljubljanica River is moderately polluted with litter and other pollutants, which are deposited in the channel because of the weak river flow. The water has a dark blue to green colour, it is not transparent and it has no odour. The performance of the fundamental viewpoint "Pollution" for the Ljubljanica River is 33 %.

Within the study reach, the Mali Graben is partly polluted with litter and other pollutants (occlusion of alluvial waste material because of the intensive riparian vegetation); the water is transparent and has no colour or unpleasant odour. The performance of the fundamental viewpoint "Pollution" for the Mali Graben River is 67 %.

In the study reach, the Glinscica Stream is not polluted with litter and other pollutants, the water has no specific colour and odour. The performance of the fundamental viewpoint "Pollution" for the Glinscica Stream is 67 %.

5. Discussion

According to the results of the application of the method in the test case studies, shown in Tables 4 and 5, the performance of the dimension "River" has proven best for the Mali Graben River, however the method has indicated a similarly poor state for the Ljubljanica River as well as for the Glinscica Stream. In pursuing the environmental goals of the WFD by the year 2015, the results of the method have suggested that the Ljubljanica River and Glinscica stream should be prioritized in terms of rehabilitation and renewal works. However, the designation of the probable status of a heavily modified water body actually applies only to the Glinscica stream, but not to the Ljubljanica River.

In terms of dimension "City" the method yields the highest rating for the Ljubljanica River, followed by the Glinščica stream and the Mali Graben River. From the aspect of improving the living environment of inhabitants and other users of space, the Mali Graben River should be prioritized for an adequate urban upgrading and connection of the river corridor and built-up urban tissue.

Based on the expert study and field assessment of the status and comparison of test study reaches it can be established that the results of the application of the method reveal a good response of the method. The values of dimensions "River" and "City" as well as single "Fundamental viewpoints" for dimensions "River" (River Morphology, Biological Components, Natural and Technological hazards) and "City" (Urban Space Quality, Cultural Heritage, Activities, Accessibility, Pollution) provide an objective assessment of the status of test river reaches.

However, it should be emphasised that the overall assessment of the dimensions "River" and "City", which is similar for all three rivers, cannot be regarded as a relevant indicator of status, since it contains the assessment of the ecological status of the river corridor and spatial integrity of the urban tissue and river corridor, which are not comparable values of the urban environment. In this manner, the results of the method have shown that the aesthetic value cannot be equivalent with its hydromorphological status. Accordingly, a hydromorphologically heavily modified test reach of the Ljubljanica through the city centre can have high aesthetic value.

Besides the expert review of the adequacy of assessments of the status of dimensions "River" and "City", we provide some further conclusions. When applying the method, the influence of subjectivity of corresponding viewpoints should be considered (dimension: "City"), where the degree of aesthetic value of a specific viewpoint in a concrete area is established on the basis of expert assessment (e.g. most fitting number of potential river crossings in the area, appropriate density of landmarks). Having this in mind, the accuracy and repeatability (robustness of the method) should be checked. In addition, the method cannot be applied in a simple manner: several data are required, which are often not available for the area (e.g. intensity of construction, number of people that use the bridges daily etc.).

Table 4 Average performance values of fundamental viewpoints of the three case studies in Ljubljana

Fundamental viewpoint	Average Performance Ljubljanica River [%]	Average Performance Mali Graben River [%]	Average Performance Glinscica Stream [%]
Dimension "River"			
River Morphology	20.8	45.8	20.8
Biological Components	25.0	79.3	8.3
N & T Hazards	12.5	37.5	37.5
Dimension "City"			
Urban Space Quality	66.1	38.0	60.3
Cultural Heritage	100.0	0.0	0.0
Activities	62.5	62.5	75.0
Accessibility	57.1	45.8	51.0
Pollution	33.0	67.0	67.0

Table 5 Average performance values of dimensions "River" and "City" of the three case studies in Ljubljana

Dimension	Average Performance Ljubljanica River [%]	Average Performance Mali Graben River [%]	Average Performance Glinscica Stream [%]
River	19,4	54,2	22,2
City	63,7	42,6	50,6
Overall	41,6	48,4	36,4

The aesthetic value is undoubtedly an important element in the process of renewal and rehabilitation of rivers, certainly, it is the element that is usually noticed first (Shannon et al. 1995, Ortolano 1997). Urban rivers are particular from at least two points of view: narrowness of the river corridor inside the urban tissue and also a variety of uses inside the urban river corridor for everyday and leisure activities of city population (Bizjak and Mikoš 2001, Mikoš and Kavčič 1998, Perspektiven 1994, 2002). That is the reason why the determination of urban river corridors, which should be prioritised for the implementation of revitalisation or rehabilitation measures, also requires the analysis of the aesthetic value and aesthetic potential of urban rivers.

6. Summary

The application of the method for assessment of the aesthetic value of three test rivers in Slovenia has raised some theoretical dilemmas and has also shown certain problems in terms of practical application of the method. In our opinion, the method offers a good basis for further research in the field of assessment of aesthetic value of urban rivers and streams. From the view of further optimisation of work procedures and methodological processes, some recommendations and comments should be considered. This could be done through the involvement of the existing methods for the analysis of the hydromorphological status and established procedures for restoration or rehabilitation of urban rivers and streams in combination with the experiences gained through practical realisation of such projects.

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Evaluation of the Ecological Status of Surface Water in Poland – Experiences and Problems to Solve

Zofia Gręplowska

Abstract: The author of the paper concentrates on the analysis of the hydromorphological quality elements of the surface flowing waters, indicating the difficulties in the identification of the pressures and impacts, which are caused by the lack of proper monitoring in Poland. Furthermore the paper discusses: types of the characteristics accepted as the base for an initial identification of the hydromorphological status of the surface water body, evaluation criteria of the quantity and morphological status, threshold values of the indicators allowing to evaluate the hydromorphological status, principles of making an initial evaluation of the hydromorphological status of surface waters, and identification principles of significant hydromorphological pressures on water bodies in risk of failing the specified objectives.
The paper also discusses the difficulties and basis of the accepted procedure of the initial evaluation of risk of failing the objectives.
Finally, the kinds of measurements needed in Poland to verify the result of the presented methodology are indicated.
Keywords: hydromorphological status, pressures and impacts analysis

1. Introduction

In May 2004 Poland became a member of the EU. Since that moment the timetable of Water Framework Directive implementation has been obligatory for Poland as it is for all other EU members. One of the first and very urgent tasks was to indicate the water bodies at risk of failing the proper environmental objectives. Consequently, the need to draft the methodology of the realization for this task arose.

The presented methodology was prepared within the framework of a grant entitled “Methodological basis of national plan of integrated development of water management in

Poland", financed by the Polish Ministry for Science. The work was presented in the monograph: "Identification and evaluation of anthropogenic pressures and impacts on water resources for indication of water bodies at risk of failing their environmental objectives". It should be pointed out that the methodology described in the above mentioned monograph refers to the initial identification of risk for all types of surface and ground water bodies.

In this paper the methodology for running surface waters is presented.
The environmental objective for the running surface waters until 2015, defined in the Water Framework Directive, is a good status, which means good ecological and good chemical status. Ecological status is defined by three groups of elements: biological elements, hydromorphological elements and chemical and physico-chemical elements. Hydromorphological elements should be treated as elements supporting the evaluation of ecological status. In principle this evaluation should be based on the biological elements. No systematic biological monitoring has been performed in Poland till now. As a result, the initial evaluation of the ecological status has been based on the analysis of the physico-chemical and hydromorphological elements (hydromorphological elements enable to evaluate the biological quality indirectly).

In the paper the methodology of the initial identification of the water bodies at risk of failing good hydromorphological status, adequate to Polish conditions is presented.

2. Hydromorphological quality of rivers and Polish conditions of its evaluation

The Water Framework Directive and the respective decree of Polish Ministry of Environment (DzU Nr 32, poz. 284, 2004) distinguish the following hydromorphological quality elements:

- hydromorphological regime,
- river continuity
- morphological conditions described by the variation of river depth and width, structure of substrate and riparian zone.

Both of the above mentioned documents provide analogical definitions of a very good, good and moderate ecological status respectively.

The full assessment of the hydromorphological status requires:

- reference conditions defined for specified types of the river landscapes,
- morphological monitoring,
- characteristics of the river dynamic (including data about flow regulation) and the dynamic of floodplains as a result of survey.

Methodology of the initial assessment of the hydromorphological status of rivers proposed in Poland has been based on such data which is attainable for the whole country only. It is not therefore the whole evaluation. The basic scarcity concerns reference conditions and systematic morphological monitoring. The surveying and mapping of the river and floodplain structure, riparian vegetation and type of floodplain use has not been done too.

Methodology of the first (initial) evaluation of the status, which should help to indicate the water bodies at risk of failing to achieve the good hydromorphological status was based on the identification and evaluation of the significant pressures on the river hydromorphology.

3. Indicators for identification of hydromorphological status

It has been assumed, that the identification of the hydromorphological status would be based on the following parameters (indicators):

a) for changes of the flow regime:
- value of the not returned water abstraction,
- ratio of the active capacity of all retention reservoirs to the mean annual outflow,
- ratio of the mean annual flow for the period 1981 – 2000 to the mean annual flow for the period 1951 – 1970 (admitted as natural);

b) for morphological changes in the channel and floodplains:
- ratio of the total length of both sides levees to the river length,
- ratio of the total height of cross structures to the total drop of analysed river section[1],
- ratio of the total length of the river sections, which are separated by the cross structures h>0,7 m to the length of the significant rivers,
- ratio of the river stretches with the longitudinal regulation structures to the river length.

4. Methodology of the initial identification of the surface, flowing water bodies at risk of failing to achieve the good hydromorphological status

The presented methodology of the initial identification of the surface, flowing water bodies at risk of failing the good hydromorphological status comprises two stages. The aim of the first one is to identify the water bodies of a good hydromorphological status and the water bodies of a hydromorfological status worse than good. The second stage comprises a more detailed analysis of the water bodies of a worse than good hydromorfological status. This analysis allows for an identification of the water bodies, which are actually at risk of failing the good hydromorphological status.

4.1 First stage of the evaluation of the hydromorphological status

Selection of the water bodies of an unquestionably good hydromorphological status and water bodies, which are potentially at risk of failing the good hydromorphological status is done on the basis of the threshold values of the coefficients, which are listed in the tables 1 and 2.

Table 1 includes the coefficients enabling to asses the hydrological status and table 2 helps to assess the morphological status of the water body.

Table 1 Threshold values of the coefficients for the evaluation of the hydrological status of the water body

N°	Description of the coefficient	Definition of the coefficient	Threshold value
1.	Ratio of the total active capacity of the retention reservoirs V_a to the mean annual outflow in the cross-section closing the water body catchment V_{MA}.	$e_1 = \Sigma V_a / V_{MA}$ [mln m^3/mln m^3/year]	0.03 (3%)
2.	Ratio of the total unreturnable water abstraction[2] P_{ua} to the mean annual flow (MAF)	$e_2 = \Sigma P_{ua.} / MAF$ [m^3/s / m^3/s]	0.05 (5%)
3.	Absolute value of the complement to 1 of the ratio of the mean annual flow (MAF) calculated for the last period (e.g. years 1981 – 2000) to the mean annual flow for the period acknowledged as natural MAF_n	$e_3 = \lvert 1 - MAF / MAF_n \rvert$ [-]	0.1 (10%)

Table 2 Threshold values of the coefficients for the evaluation of the morphological status of the water body

N°	Description of the coefficient	Definition of the coefficient	Threshold value
1.	Ratio of the total levee length L_l to the total length of the significant rivers L_r	$e_4 = \Sigma L_l / \Sigma L_r$ [km/km]	0.3 (30%)
2.	Ratio of the total height of the structures H_s to the total fall of the rivers H_r	$e_5 = \Sigma H_s / \Sigma H_r$ [m/m]	0.1 (10%)
3.	Ratio of the total length of the river riches L_c cut off by the cross structures having the height h>0,7 m to the total length of the important rivers L_r	$e_6 = \Sigma L_c / \Sigma L_r$ [km/km]	0,30 (30%)
4.	Ratio of the total length of the regulated river riches (longitudinal structures and/or alteration of the river course) L_{reg} to the total length of the signif. rivers L_r	$e_7 = \Sigma L_{regul} / \Sigma L_r$ [km/km]	0.2 (20%)

A water body is assumed to have a good hydromorphological status if values of all coefficients listed in the tables 1 and 2 are less than their threshold values.

Water bodies, which don't fulfil this condition are initially classified as the water bodies at risk of failing the good hydromorphological status. They undergo more detailed analysis to indicate the water bodies, which are really at risk in perspective of the year 2015.

4.2 Second stage of the initial evaluation of the hydromorphological status

The second stage of the initial evaluation of risk of failing to achieve the good hydromorphological status consists of the identification of risk causes and of possibilities to prevent this risk. The evaluation of the risk causes consists in the identification of the significant pressures. The evaluation of their impact and possibilities to limit them takes the form of an expert assessment.

Significant pressures on a hydrological state

Water abstractions and influence of the retention reservoirs are the causes of the changes in the hydrological regime.

The proposed procedure for the identification of the significant water abstractions in the area of the water body consist of:

- identification of water intakes causing changes of the hydrological regime (great number of small, dispersed water intakes or specified number of great, listed water intakes);
- assessment of the range of impact of the significant water abstractions on the hydrological regime of the water bodies downstream.

The evaluation of the hydrological changes caused by the retention reservoirs have to be based on a case-by-case analysis.

Significant pressures on the morphological state

Identification and evaluation of the significant morphological pressures have to be done by expert assessments at the present time. Criteria of this evaluation are adjusted to the particular cases.

Indication of the water bodies at risk of failing the environmental objectives

The list of the water bodies at risk, indicated in the first stage of the analysis, should be completed by adding the list of those water bodies which have been admitted as being at risk during the analysis of the significant pressures. The decision about identifying a water body as being at risk is based on an expert assessment.

The classification according to the degree of exceeding of the threshold values, listed in the tables 1 and 2 can be helpful. The classification by the degree to which the threshold values listed in tables 1 and 2 have been exceeded is presented below:

up to 10 %	- low
up to 30 %	- medium
up to 100 %	- high
more than 100 %	- very high.

5. Example – Raba river catchment

Raba river catchment (Figure 1) is one of the pilot areas for the implementation of the WFD in Poland.

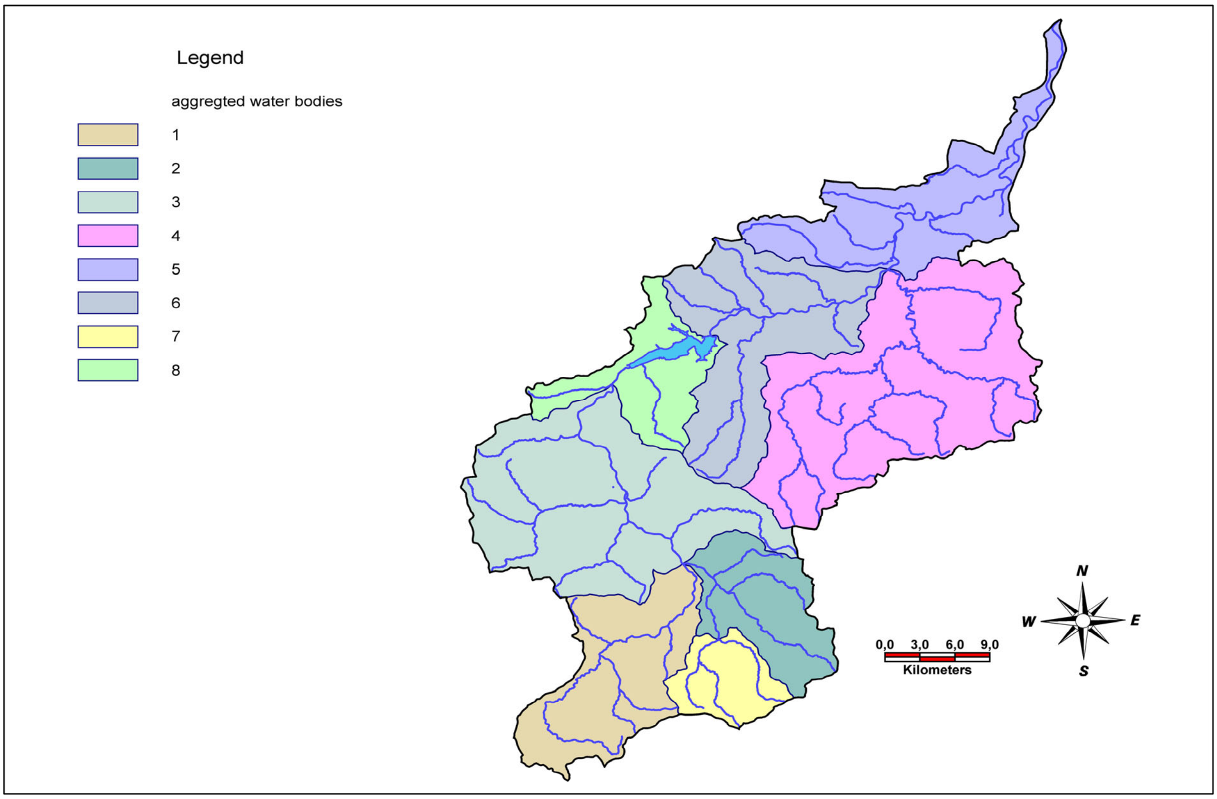

Figure 1 Raba river catchment – water bodies aggregated for the pressures and impacts analysis

In the first stage of the evaluation of the hydromorphologica status it was determined that the threshold values of the hydromorphological coefficients in the aggregated water body N^{o} 4 (Stradomka river catchment) had been exceeded. The values of the exceeded coefficients are as follows:

a) hydrological status

$$e_3 = \left| 1\text{-}(MAF/MAF_n) \right| = 21{,}8\% \qquad \text{(threshold value - 10\%);}$$

b) morphological status

$$e_6 = \Sigma L_c \,/\, \Sigma L_r = 45{,}6\% \qquad \text{(threshold value - 30\%)}$$

$$e_7 = \Sigma L_{regul} \,/\, \Sigma L_r = 42{,}1\% \qquad \text{(threshold value - 20\%)}$$

In the second stage of the hydrological status evaluation the identification of the significant quantity pressures was attempted. In the case of the analyzed water body the following could be stated:

- no significant water intakes but high population density (117 inhabitants per km^2),
- middle level of farming (44 heads per 1000 ha of arable land)
- middle level of water supply needs (0,033 $l/s/km^2$)
- intensive exploitation of ground water resources (on the base of the quantity evaluation of the relevant ground water body).

It can be stated, that the distinct drop of MAF for the period 1981 – 2000 compared to the period 1951 – 1970 is a result of the greater water use by households, supplied from springs and wells and of the periodical, significant, natural MAF changes. The water body N^{o}4 should therefore be counted among the water bodies at risk of failing the good hydrological status but this evaluation should be verified in the future.

In the second stage of the morphological status evaluation the causes of such an intensive longitudinal and cross regulation was recognized as well as its efficiency and the consequences for the conditions of fish life.

It was found that the regulation was done for the stabilization of the channel. This regulation is not fully justified. The cross structures (thresholds), which are 70 or more cm high make the free migration of fish impossible. Artificial character of cross sections of some river reaches causes inconvenient changes in the life conditions for the characteristic (for this river) fish kinds.

Morphological pressures could be estimated as significant enough for the water body to be acknowledged as being at risk of failing the good ecological status but only if the rehabilitation of the river would not have been done.

The above mentioned situation is an example indicating the need to analyze the risk level and the social conditions from the point of view of the „heavily modified" status for the water body under discussion.

6. Problems to solve

The methodology of the initial evaluation of risk of failing the environmental objectives in case of surface water bodies and its first applications indicate that the list of the water bodies at risk should be verified in the next years and the base of this verification (despite the proper designed monitoring) should be:

- screening of the channels and river valleys from the point of view of their morphological status,
- more detailed hydrological analysis based on available data.

Screening of the river channels and valleys should be based on the maps and aerial photography as well as on the field survey. Different descriptive and numerical data should be also used in the screening. Especially the following should be done:

- the characteristics of the water course in relation to the planform of the water bodies of good ecological status,
- identification of causes and functions of artificial cross structures in relation to the existing land-use in the adjacent area,
- identification of vegetation types and range of their existence,
- identification of types and range of flood protection structures (e.g. levees) in relation to the land-use in the adjacent area,
- identification and characteristics of bank zone,
- evaluation of the possibilities for river to overflow the adjacent area.

The scope and the method of data gathering is now being investigated. The evaluation procedures are also being tested. The problem of the designation of the heavily modified water bodies is still open.

7. Summary

The presented methodology concerns the initial evaluation of the risk of failing the environmental objectives in case of surface, flowing water bodies in Poland. It is based on the existing data/information only. List of the water bodies at risk should be verified in the coming years. This verification has to be based on the output from the diagnostic monitoring, which should be properly planned and properly realized. In the case under discussion the river channels survey from the point of view of their morphological status is of basic importance. The creation and development of an adequate data base is of equal importance.

[1] A dam is taken into consideration if a reservoir has not been qualified as a heavily modified water body only
[2] If there is no information about unreturnable surface water abstraction (water loses at users) this abstraction can be estimated, using loses coefficient typical for the given type of user

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Rivers without Obstacles for Fish Migration – Demands on River Stabilisation Methods

Michael Hengl
Ursula Stephan

Abstract: The main purpose of this contribution is to stimulate the hydraulic research to develop bed stabilisation measures for practice, to reduce the environmental influence of human activities on our rivers. Therefore, this issue is generally addressed in order to present some basic ideas for future research. Due to bed degradation many rivers in the alpine region are stabilised with help of different kinds of sills. Usually the fish migration in upstream direction is interrupted at these sills. In many rivers, bed degradation constantly continues and must be stopped and/or transferred into a controlled aggradation process to prevent further damage.
Currently different bed stabilisation methods exist for practical use. But many of these methods are not sufficiently examined from all points of view such as bed stabilisation, morphology, fish migration, land use, costs or leisure activities. Each method has its own advantages and disadvantages, e.g. many bed stabilisation measures are passable only for certain fish species. Nevertheless, each adult fish specie, which is typical for a bio region, should be able to ascend towards its spawning-ground without running into an obstacle.
For biological, morphological and financial reasons, each type of watercourse needs its own bed stabilisation method. Some examples are rough ramps, which are used mainly in the salmon region, fixations in sections of the river bed keeping the original bed slope or being slightly steeper in potamal river regions and river widening. Naturally, these measures are to be in equilibrium with the bed load regime of the concerned river reach.
Keywords: river stabilisation, morphology, fish migration, sustainable solutions

1. Why river stabilisation and what are the problems?

1.1 Morphological basics

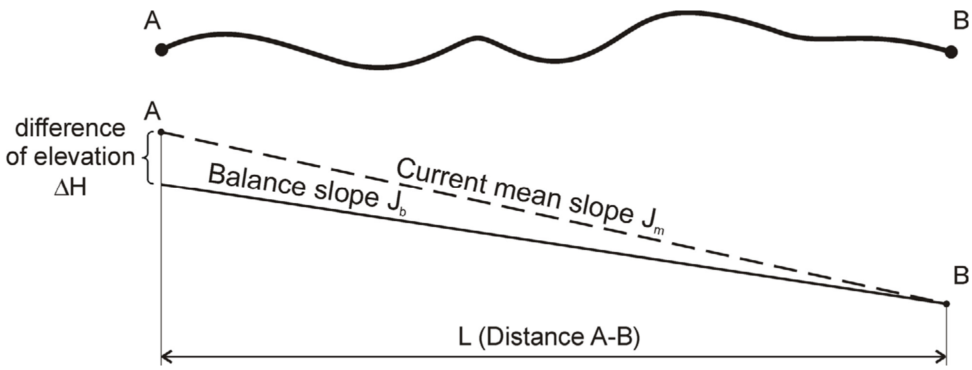

Figure 1 River degradation – difference between current unstable mean slope and final balance slope

Each river with a current mean slope higher than the balance slope shows a state of degradation (see fig. 1). The difference in elevation ΔH which results from this slope difference mainly depends on the river hydrology, bed load (amount and grain size) and the cross sectional geometry and causes a surplus kinetic energy. To stabilise a degrading river bed either means increasing the balance slope by, e.g. increasing the bed load transport rates, widening the river bed, lengthen the river reach, or dissipating the surplus kinetic energy at one special, manageable location such as sills, weirs, ramps or hydroelectric power plants. The latter was the main used solution in the past with energy dissipation by means of a hydraulic jump (sills, weirs, hydroelectric power plants) and the roughness of the ramp surface (ramps), respectively.

The current discrepancy between the mean river slope and the balance slope of many European rivers such as Salzach, Isar, Inn, Danube or Drau has evolved from various human activities in the past. Rivers which originally took up the whole valley bottom were significantly straightened and constricted in order to gain new agricultural areas and to reduce annual floods. These measures, causing an increased bed load transport capacity, deliberately initiated an erosion process. In addition, the river was forced into a fixed bed. Bed load is often entrapped in the higher regions of the catchment (e.g. sediment-control dams to prevent settlements from debris flows) or upstream of hydroelectric power stations with too little bed load transport capacity. And finally, gravel was dredged from the river bed for various purposes. Thus, as a typical consequence the river system adjusts in a long-term geomorphological manner to these human influences.

1.2 Ecological problems

Sills and ramps used to be built mainly to stabilise the river bed without considering fish migration, because questions addressing the river ecosystem were less important than river stabilisation and directives to account for, e.g., fish habitats were missing. Consequently, sills and ramps often act as migration barriers.

Figure 2 Example of a fish migration barrier due to bed erosion downstream of a ramp

In case of a balance slope lower than the current slope of the river and, thus, ongoing erosion, the bed stabilisation structure can interrupt fish migration also if the structure itself is suitable for fish migration (see fig. 2). At the same time, this might endanger leisure activities such as rafting and canoeing. In case of subsidence of the foundation due to a wrong ramp construction, formerly ecologically favourable ramps can also develop to ecological barriers (see fig. 3).

The present situation in Austria shows that only 60 % of all river reaches with catchment areas larger than 100 km² are expected to meet the demands of the Water Framework Directive. The reasons are mainly barriers for fish migration due to measures to stabilise the river bed such as sills, ramps, hydroelectric power plants.

1.3 Why is river stabilisation still needed?

Many rivers are still in a state of degradation. Since no changes of bed load budget are to be expected, existing obstacles such as sills must not be removed without offering appropriate alternatives.

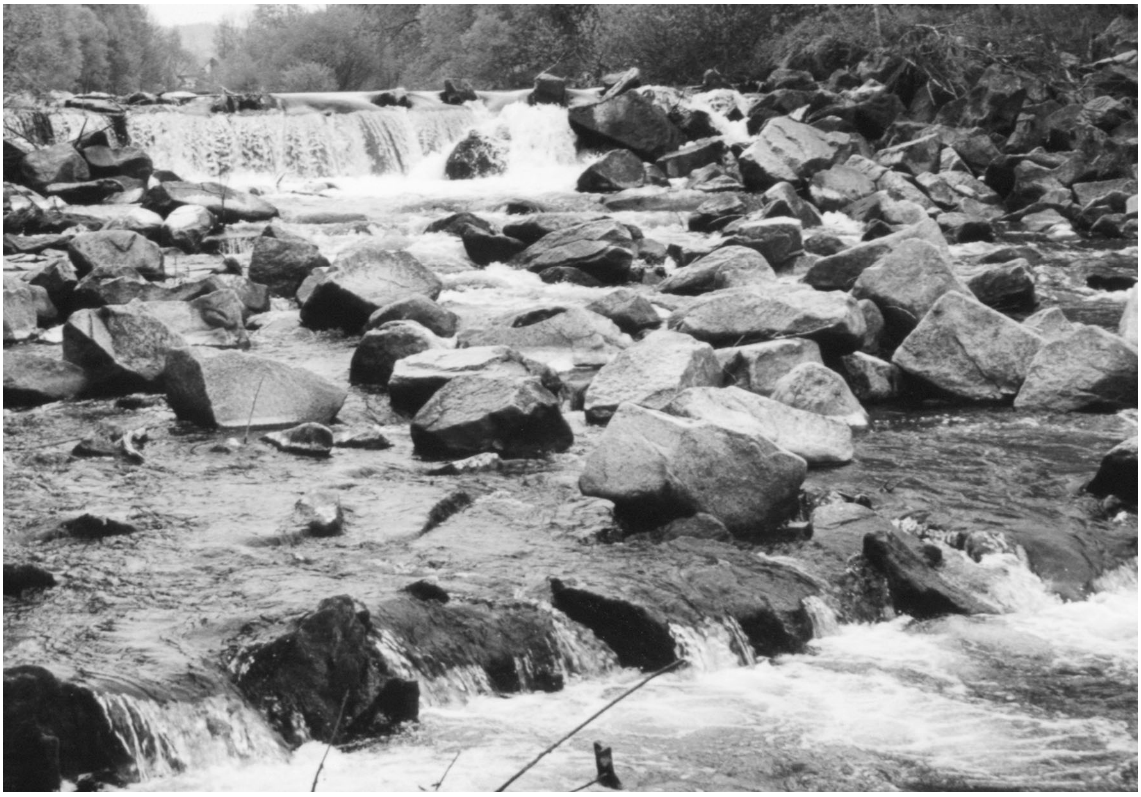

Figure 3 Example of a fish migration barrier due to subsidence of a ramp after decades because of insufficient filter construction

Sometimes, bed erosion can progress to such an extent that bridge piers as well as bank structures and, consequently, buildings and infrastructure near the banks become endangered. The groundwater table decreases remarkably. The river wetlands are cut off from the river itself, which is closely connected to a loss of habitats and of population diversity in the wetland ecosystems.

Additional to degradation processes, river aggradation might also cause problems. Wrongly estimated morphological changes following river restoration measures, which for example initiate an aggradation process, can lead to flood protection problems. In this case, sustainable bed stabilisation measures (river adjustments) are also needed.

2. What are the demands?

The European Water Framework Directive and national water laws prescribe rivers without barriers for fish migration in accordance with the concerned bio region. Additional demands, which must be considered too, are the stability of the construction in case of floods, assured flood protection, consideration of possible changes of the ground water level and the ecological and morphological sustainability, i.e. the long-term stability of the construction as well as the adjacent river reaches upstream and downstream of the structure.

The necessity to achieve a state of sustainable dynamic equilibrium does not entail armouring the river bed, i.e. fixing the bed at one specific level. Rather, it means that the river bed varies within acceptable limits according to time dependent sediment transport and discharge, respectively. In this manner, the river bed can adapt to both time-dependent and morphological boundary conditions. Nevertheless, the range of bed level changes should not conflict with other demands, such as recreational activities (rafting, canoeing) or costs and maintenance of the measures. Finally, the design of the measures should also settle esthetical claims, since controversial discussions sometimes occur in the field of architecture as experience shows.

3. On-hand solutions and what is unsatisfying?

According to the state of the art, there are different possibilities to stabilise a degrading or aggrading river bed and, thus, to reach the balance slope. The measures can either influence the mean or the balance slope to reduce the discrepancy between mean and balance slope. One is to vary the length of the course of a river. Shortening the river course means to increase the mean slope and lengthening means to reduce it. Another is to vary the river width. Widening the river means to increase the balance slope, constricting the river to reduce it and, thus, aggradation and degradation, respectively, might be stopped.

To lengthen a river or, to be more precise, to reconnect bayous or old meanders, is the easiest way to approach to the balance slope. Usually, this solution fails due to agriculturally used, river adjacent areas which are either unavailable for river restoration measures or too expensive to acquire. In addition, in some cases even nature conservation prevents a change of the river course due to, e.g., rare species in the concerned area. In case of river widening, the same problems of conflicting interests between river ecology and nature conservation can occur. Furthermore, it is difficult to precisely estimate which river width the river will sustainably accept. It is well known that a formerly braided river morphology may pass into a flat and narrow river bed with a considerably reduced balance slope in case of bed load shortage (Zarn, 1997).

Generally, all methods like ramps, rip raps or groynes use different sizes of stones or boulders. For dimensioning the boulder size, usually a characteristic stone diameter, the equivalent spherical diameter, is calculated from design equations. But slightly changing the diameter also means enormously changing mass and volume of a stone, e.g. a 10 % increase in spherical diameter results in a 33 % increase in weight and, thus, also in costs. At worst, the general acceptance is refused because the stones used for bed stabilisation are significantly larger than the natural stone sizes in the river.

The ecological impacts of ramps and rip raps on different kinds of bio regions are still not fully understood. An example is the acceptable (maximum) slope for a ramps in salmon regions. Should it be 1:8 or 1:10 or 1:12 or even less inclined? An unnecessary flat ramp might lead to unnecessary expensive measures, since a flatter ramp increases the ramp length and, thus, the material requirements. At the same time, a flatter ramp does not necessarily mean unfavourable effects on costs, since a flatter ramp might also improve the energy dissipation on the ramp due to its increased length compared to a steep, but shorter ramp and, therefore, might reduce the necessity of scour protection downstream of the ramp.

4. What is needed?

This chapter invites researchers to take part in developing new measures and improving existing methods which meet the demands mentioned above. The output might be a catalogue of possible measures for each bio region and morphological situation with advantages and disadvantages regarding ecology, morphology, costs, land use etc. (see fig. 4). Additionally, it is not sufficient to develop a morphologically and ecologically satisfying solution for a certain situation. The measures should also afford a kind of flexibility to be adjusted in accordance with the demands and adapted to changing boundary conditions, such as a changing or uncertain bed load budget (Hengl, Stephan, 2003).

For a successful design of a measure, it is important to likewise consider ecological, hydraulic and morphological principles. To define future research questions it would be even more important to encourage interdisciplinary co-operation between biologists and engineers, since hydraulically and morphologically suitable measures for stabilising a river bed are not necessarily ecologically suitable and vice versa.

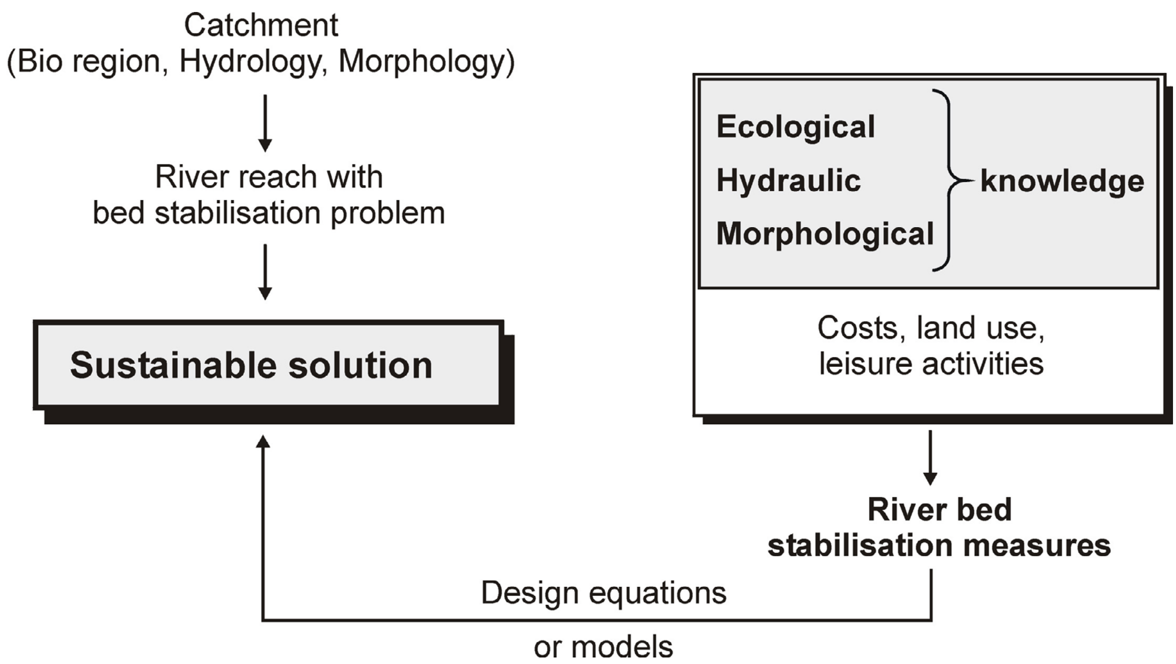

Figure 4 Development of sustainable river bed stabilisation measures

4.1 Enlarge the ecological knowledge

What are the limits for fish migration for different species? Which species need migration? Can natural reproduction be assured without migration facilities because of adequate spawning grounds within the concerned river reach and connected waters? During which period is migration needed - during the whole year or only during spawning season?

It would further be helpful to reduce the conflicting interests between ecology focusing on natural dynamic of waters and species and nature conservation preserving the existing system without changes to a common denominator.

4.2 Enlarge the hydraulic knowledge

What is the flow resistance of different bed stabilisation methods like ramps to ensure flood protection? Which local flow velocities and turbulence must not be exceeded during the migration season? These data should be available during the planning process for not constructing a migration barrier finally. What is the minimum stone size to ensure the stability for the whole range of discharges for ramps and similar stabilisation methods? Is it possible to use different stone sizes in different parts of the bed stabilisation measure? How much kinetic energy is dissipated in a particular river reach?

4.3 Enlarge the morphological knowledge

How does a bed stabilisation measure influence the general morphology of the river reach (including habitat change)? The morphological knowledge is closely connected to the hydraulic knowledge since flow velocity and turbulence are the driving forces for each morphological change. Thus, developed methods should improve the quality of balance slope estimation and answer the question which bed level ranges are to be expected upstream and downstream of a bed stabilisation measure (e.g. weir height upstream and scour depth downstream of a ramp).

4.4 Develop design equations or models which are easy to handle for engineers

If new measures are planned, the planner needs to know how the river system will adjust to this measure in the future. Thus, it is important to develop design equations for an easy and convenient handling. One possible way to solve such kind of problems effectively could be more co-operation between engineers, biologists and professional software developers. A database with main information on range of application, necessary data to run the model, model output and last but not least how to license the model or who offers model applications would be useful to get a general idea about existing and available models.

Summary

A large amount of problems, mainly concerning rivers, which are not in a state of morphological equilibrium, is highlighted, since many of these questions are not fully answered yet. Therefore, a joint effort of the scientific community is needed to develop ecological, economical and sustainable solutions for river stabilisation. The water management administrations need good answers to achieve a high or at least a good ecological status for our rivers as demanded in the EC Water Framework Directive.

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River Restoration in Hungary – Some of the First Years, Some of the First Projects

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Abstract: Rivers are lifelines in all kind of aspects. They are dynamic, diverse and complex ecosystems and they always have been a focus for many human activities. Rapidly increasing populations and growing demands for water and land have led to the degradation of many river systems. Currently many river restoration projects are being implemented in Europe. Rivers in Europe show many similarities on one hand and important differences on the other hand. This is reflected in the restoration approaches. The experiences gained can be used in future river restoration projects. We hope this conference with the help of our paper will supports the development of river restoration as an integral part of sustainable water management throughout Europe ensuring that projects will be more cost effective, more likely to succeed, and will encompass multifunctional objectives. At the end of this conference we hope to have a number of technical recommendations on river restoration as well as a clear view of what we expect and need work for.

1. Antecedents of river management

New principles and approaches of river management at the end of the 20th century

River regulation works, such as utilization, regulation, management, after all generation and preservation values to be gained using running water can only be completed by a number of highly responsible technical-economical decisions. To the best of our knowledge regulation of rivers is a dinamically changing interference, which always keeps social and economical demands in view and may influence life circumstances of entire regions of the country. The reasonableness of the topic was at the end of 1996 and the beginnings of 1997 also supported by a great number of economical-social changes made at the end of the 80s, which forced several branches of the water management service to develop their updated programmr strategies according to the new conditions.

Conception of river management

The conception deals first of all with opinions of the water management sector referring to river management, summarizes theories principally matching recent natural-social conditions, but it is not to be considered as an isolated document.

It fundamentally matches the principles of the announcement "The water management policy of the European Union" published by the EU, those of the Act of Water Management and the strategies of the Hungarian Water Management and Water Damage Prevention Policy. The scheme also fits the Regional Development Conception besides expressing its water management principles further on.

The chief aim of the conception

Composition of integrated river management principles and aims referring the whole country and making different kinds of approach possible is one of the most important questions nowadays. The conception meets the requirements of the society, is economically effective, ecologically acceptable, reliable and looks like a kind of European solution. In order to be accepted in the long term the conception is based on real facts and actually considers political and economical situation, resources, geographical circumstances, and possibilities of technical development.

Strategic programme of water management

Having accepted the conception, during developing the strategic programme of water management it was essential to work out a draft plan of the programme based on the "Integrated conception of water management" made in 1998 – so that it can be realized. In the draft plan – setting out from the requirements given – duties had to be listed in 15-20 ranges, then guiding principles, development directions, financial effects of development and regulation activities of the next 20-30 years had to be defined. The study was the basis to go on to make a proposition for Ministry and Government.

Government proposition of realization integrated river management

Higher requirement standards of rivers also means facing new challenge of river management. Performing several functions of rivers forces integrated approach. Integrated river management is a new system of planning and decisions, which provides a frame and gives a method in order to develop abilities to achieve management aims requiring the knowledge of a number of disciplines, and to make society accept and appreciate its efforts. The tendency, which is only in its early stages, also has an initiative in Hungary, called the programme of "Development of river management", and the programme of integrated river management. There is a close connection between particular duties of the programme, because several sectors of economy have an interest in all of them, and close co-operation of various branches is needed to realize them successfully.

Realization of integrated river management can be planned in two phases:

In the first phase aims, interested parties, objects, activities of the new management system must be defined, methods of reconciliation of interest must be worked out and basic means of management must be developed. This is actually the preparatory programme of the integrated river management.

In the second phase the so called application programme can be started, which has to work out river management plans, operate and post-evaluate, based on regulation plans aiming realization of river management.

Following expediency, as the first step, governing development principles of management of our rivers have been worked out, containing altogether 14 strategical duties, which define our most important tasks to do. This programme was accepted by the Government, which decided to form an inter-departmental committee in order to realize the preparatory phase. The schedule contains explanation and account of the particular tasks, conditions or requirements of realization and the essence of the job to do.

2. Achievements of river management between 2001-2002

River management plan of the River Tisza, stage I. – 2001

Completed duties:

A,

- Definition of participants of integrated river management:
- definition of circles laying claim to river management,
- presenting reasonable claims lenghtways the river,
- classification of tasks (possible utilization) according to the interests

B,

- Working-out a plan of river management ("agreed plan")

The method of working-out, updating and extraordinary change of a plan of river management, including specimen, in order to assert short- and long-term claims.

The material completed in the first phase deals first of all with opinions of river management given by water management experts, it summarizes all the theories matching natural and social conditions the most.

Working-out of a value-scheme in river management – 2001

In order for the Integrated River Management Programme to achieve its aims, and for it to be actually able to function, a kind of value-scheme helping judgement of utilization had to be made during the Preparatory Programme. This is a determinative document file of river management planning, which enables registration of utilization-functions of a river, documentation of harmonization of communal interests, following temporal change of dynamic values, interferences of development and maintenance. The value-scheme had to match the water management policy and Water Framework Directive of the EU and the characteristic of the Danube Basin. The model, suitable for comprehensive comparison, also had to pay attention to the aspects of regional development and settlement protection, provincial development, agricultural utilization, silviculture, utilization of industrial- and drinking water, protection of water quality, shipping, underground water resources, biodiversity, protection of ecological values and conservation of riverside ecosystems.

Operation of the inter-departmental committee of river management – 2001

The following tasks had to be completed:

- antecedents had to be get to know,
- information about up-to-date tendencies had to be gathered,
- the aims of river management to be achieved had to be analysed,
- long-term development, future effects, influences, and possible conflicts of river management had to be defined,
- the material for the inter-departmental committee had to be summarized.

Development strategies of the Danube branches – 2001

Completed works:

- compilation of a short summary about possible and necessary rehabilitation of Danube branches defined through analysis of value,
- sending the issue to the interested (ministry, interested authorities of environmental protection, national parks, self-governments and other organisations),
- detailed surveying of the branches in the order of importance, preparatory works of detailed rehabilitation plans with the help of the interested
- defining future rehabilitation expenses based on the completed plans, partition of expenses according to the interest relations (own capital, central budget, applications, other sources).

Organising a discussion: Achievements and future plans of the river management works of the River Rába – 2002

During this work river management plans for the following 10 years had to be defined based on the recent achievements of river management planning of the River Rába. The task had to be discussed, and the agreed development aims had to be published in a high circulation issue.

Founding an information system of river management (FOLYÓINFÓ) – 2002

During analysing the information system of river management it was a central aim to define the value of the existing information system and set the main development directions for the new one based on the requirements.

3. Achievements of river management in 2003

The programme: Rába is the river of the decade – 2003

Constantly based on the achievements of the river management planning of the River Rába the most important river-management aims of the following year had to be set every year. The "RÁBA strategy" had to be controlled and fitted to the national programmes. The Operation-Programme had to be worked out and the annual check-up of planning operations and expenses had to be updated. These works continued on in 2004 with a LIFE application.

River management planning of the River Tisza, stage III. – 2003

Using the experience gained by river management planning of the River-Rába, the river management planning system had to be updated, and worked on in the case of the River Tisza. It was supported by the Ministry of Environmental Protection and Water Management with the initiative "Further Development of Vásárhelyi Plan" .

Application of this plan in practice was the task of year 2003. Referring do the „Further Development of Vásárhelyi Plan" there was a reach of the River Tisza designated to try out the value scheme in practice. Having designated the appropriate reach works started with collecting data of water damage prevention, water management, water utilization, ecology and regional development. The work was supported by a successful LIFE application.

River management planning of the River Tisza, stage III. – 2003

Soon it was obvious, that there are some more essential means supporting water management works, first of all a united database, based on the sovereign official data of the interested, such as several registries and balances. The plan was realized founding the Information System of River Management (FOLYÓINFÓ), which is a system establishing, serving and managing river management. Basic data were defined by involving all the special fields of river management, and they can be found in a united database. Regarding to its structure "FOLYÓINFÓ" is a so called modelling - analysing – decision-preparing information system.

Working-out development strategies of Lower Danube branches – 2003

During recent works the team of experts completed jobs from recent valuation of branches to developing interference strategies in order to search for further financial sources. Future aims to be achieved:

- matching "Give space to rivers" EU conception,
- joining the EU Water Framework Directive (WFD),
- helping with the reduction of nutrient and pollution of the Danube,
- reducing flood-danger,
- portecting unique habitats (principles of bird- and habitat protection)
- esemplary data, surveyings (project-schemes)
- possibility of changing agricultural branches / using fields
- increasing of the population-saving abilities of the area.

Summary

River restoration embraces a great variety of measures that have in common, that they restore natural functions of rivers, that were lost or degraded by human intervention. Disposal of waste water into the river, for example, has negative effects upon many different functions, like drink water supply, irrigation water supply, maintenance of fisheries and maintenance of biodiversity. Dams for the generation of hydropower, as another example, interrupt the migration routes for migratory fish andby doing so, have a negative impact upon the income from fisheries. River restoration usually aims at restoring a multifunctional use of rivers, often by restoring more natural conditions in the river system. It is becoming increasingly clear that future river management should not focus upon adjusting the river system to human needs, but upon adjusting the human use to the natural river system. Many European rivers have often been modified in the past decades to serve only one dominant function. With some exaggeration we may say that some rivers were cut by dams into a chain of basins for the production of hydropower, some rivers were transformed into an open sewage discharge system, some rivers transformed into highways for shipping. All over Europe, however, changing environmental, economic and social preferences exert influence on river management. One-sided use with disregard of different functions is no longer considered optimal. As every function influences the different functions, an integrated approach of river restoration is prerequisite for success. With additional attention, and often additional investments, a more multi-functional river system can be created. Such 'cleaner, more complete and healthier rivers' produce more benefits for society. Decision making in the European societies is becoming ever more complex. Concerning rivers, water managers no longer dominate the decision making process. Negotiated agreement, interactive planning and the involvement of the public opinion and various stake holders are the promising ways to reconcile conflicting interests. This implies that the technical and ecological considerations are only part of the game. Raising support and the promotion of public awareness are just as essential to obtain results.

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Operation of Multistage Constructed Wetlands Systems in Temporary Climate

Hanna Obarska-Pempkowiak, Magdalena Gajewska

Abstract: In Poland the idea of sewage treatment in constructed wetland was developed mainly in one stage facilities with horizontal flow of sewage. For many reasons most of the facilities didn't ensure efficient removal of nitrogen removal, thus an idea of sewage treatment in hybrid system with mixed flow of sewage was developed. In Poland is applying the configuration with horizontal flow bed at the beginning of biological treatment.

The investigation was carried out at two different hybrid constructed wetlands, which treated municipal sewage after mechanical treatment from small villages in Pomeranian Region. In all facilities first step of biological treatment were horizontal flow beds. The main difference concern configuration of the facilities and sewage distributions systems into vertical flow beds. The measurement period was divided into vegetation (from April to October) and outside vegetation seasons (from November to March) for all analysed facilities.

The results allowed for estimation of efficiency removal of different contaminations in both measurements seasons. Comparison of the amount of sewage and loads of contamination inflowing and outflowing from the facilities are presented. Achieved results make it possible to estimate the rate of organic matter decomposition and nitrogen removal in vegetation and outside vegetation seasons.

Keywords: domestic sewage, hybrid constructed wetlands, contaminations removal, vegetation and outside vegetation season

1. Introduction

At present in Poland increases awareness for solving the problems connected with wastewater treatment in rural areas. There is a huge gap between water supply and wastewater treatment in the countryside. In 2001 about 9.0% of sewage generated in rural areas was collected and only 3.1% was treated before discharging to the recipient. Recently an idea of treating wastewater in the constructed wetland systems has emerged. In Poland these systems have been in operation for about 20 years. Nowadays more than 100 constructed wetlands are in operation in Poland. Most of them are one stage facilities with horizontal flow of sewage, which from many different reasons didn't ensure stable removal of nitrogen compounds.

New polish standards implemented in 2004 create better condition for emerged of constructed wetland methods. Discharged sewage from less than 2000 pe and above 50 pe (majority of constructed wetland in Poland) has to fulfil following standards: $BOD_5 \leq 40$ mg O_2/l, COD $_{Cr} \leq 150$ mg O_2/l, SS ≤ 50 mg/dm^3 and if the outflow is discharged to the lake: N_{Tot} $\leq$ 30mg N/l, P$\leq$ 5 mg P/l. The alleviate requirements seems to be more realistic and constructed wetland ensure efficient removal of suspended solids and organic matter.

It was proved that higher efficiency of nitrogen removal is possible in systems with at least two beds with horizontal and vertical flow of sewage, so called Hybrid Constructed Wetlands (HCW).

Both type of beds have advantages and disadvantages. Horizontal beds (HF-CW) are responsible for effective suspended solids and organic matter removal. In suitable conditions where nitrate are present in sewage HF-CWs contribute to denitrification process due to anoxic conditions created in matrix of roots. According to European standards the minimal unit are for person for this type of bed is 5 m^{2}pe^{-1} what resulted in huge areas requirements (Bärner i in. 1998, Brix i Schierup 1989, Cooper i in. 1997, Laber i in. 1997,). In addition HF-CW are more sensitive for extent organic matter loads. Vertical bed due to better oxygen condition ensure appropriate environment for transformation of ammonium to nitrate and further efficient organic matter removal (Cooper 1998, Kunst at all 2001). According to Cooper et all..(1997) unit area of the VF-CW used only for organic matter removal should be more than 1,0 m^{2}M^{-1}, while for ensure the nitrification should be more than 2,0 m^{2}pe^{-1}.As to Cooper et all (1997b) third step of treatment is conducted in HF-CW beds whit unit areas 0,7 - 1,0 m^{2}pe^{-1}. According to Birkedal at all. (1993) applying of recirculation of the sewage after VF-CW beds to the HF-CW in multistage system ensure efficient removal of total nitrogen and allows for diminishing of total unit areas for the whole system to 10 m^{2}pe^{-1}.

Up till now we can distinguish two types of HCW depending which bed HF or VF is situated at the beginning of biological treatment. In Poland is applied only configuration with HF-CW bed as a first step of treatment, than VF-CW beds. The guide lines for such system were given by Brix and Johansen in 1993 (thus the system are called Brix configurations) and the third step of treatment could be realized in second HF-CW or recirculation of sewage after VF-CW bed could be applied. While in Europe the opposite configuration with VF-CW beds as a first is more popular.

Appling all those roles during drawing and operation result in high and stable efficiency of contamination removal. Such technological solutions are effective and stable with removal even such sensitive elements like nitrogen compounds.

Both type of beds have different working conditions and ensure different characters of the processes, also applied configuration determines the efficiency removal and working conditions. The aim of the paper is to analyse how removal processes occurs during domestic sewage treatment in multistage constructed wetlands. Also still exist a question how low temperature in winter time influence on different contamination removal. Are the effectives high enough?

2. Methods

The studies were carried out at two hybrid constructed wetlands in Wiklino near Słupsk and Darżlubie near Puck (Pomeranian Region) supplied with domestic wastewater. Both facilities were built and planted with *Phragmites australis* in 1994. The sewage after mechanical treatment was pumped into biological treatment which consists of beds with HF-CW and VF-CW in different configurations. The schemes of Wiklino and Darżlubie facilities is given on Fig.1 a i b. The Wiklino consists of three beds horizontal (HF-CW I), two vertical beds which work parallel and again horizontal bed (HF-CW II). The additional bed HF-CW II was to ensure denitrification process of nitrate created in vertical beds. The Darżlubie facilities consist of five beds with mixed flow of sewage. After HF-CW I a cascade filter is located than HF-CW II bed and next are two VF-CW which work parallel and alternatively and at the end again HF-CW III is located. The cascade filter is composed of five consecutive small beds with alternatively vertical and horizontal flow. It consists of three beds with vertical and two beds with horizontal flow of sewage. After treatment in HF-CW II 50,0% of sewage is drained into VF-CW beds and the rest goes directly to HF-CW III. Such mixing of sewage is carried out in order to secure organic matter for intensive denitrification expected in last bed. The characteristic of analysed facilities is given in Table 1.

Table 1 The characteristic of analysed constructed wetlands

WWTP	Flow, [m^3d^{-1}]	Configuration	Area, [m^2]	Depth, [m]	Unit area, [m^2PE^{-1}]
Wiklino	18.7	HF	1 050	0.6	7.0
		VF	624	0.4	4.0
		HF	540	0.6	3.4
			Total 2 214		Total 14.4
Darżlubie	56.7	HF-CW I	1 200	0.6	1.6
		Cascade filter	400		
		HF-CW II	500	0.6	0.5
		VF-CW	250	1.0	0.7
		HF-CW III	1 000	0.6	1.3
			Total 3 350		Total 4.1

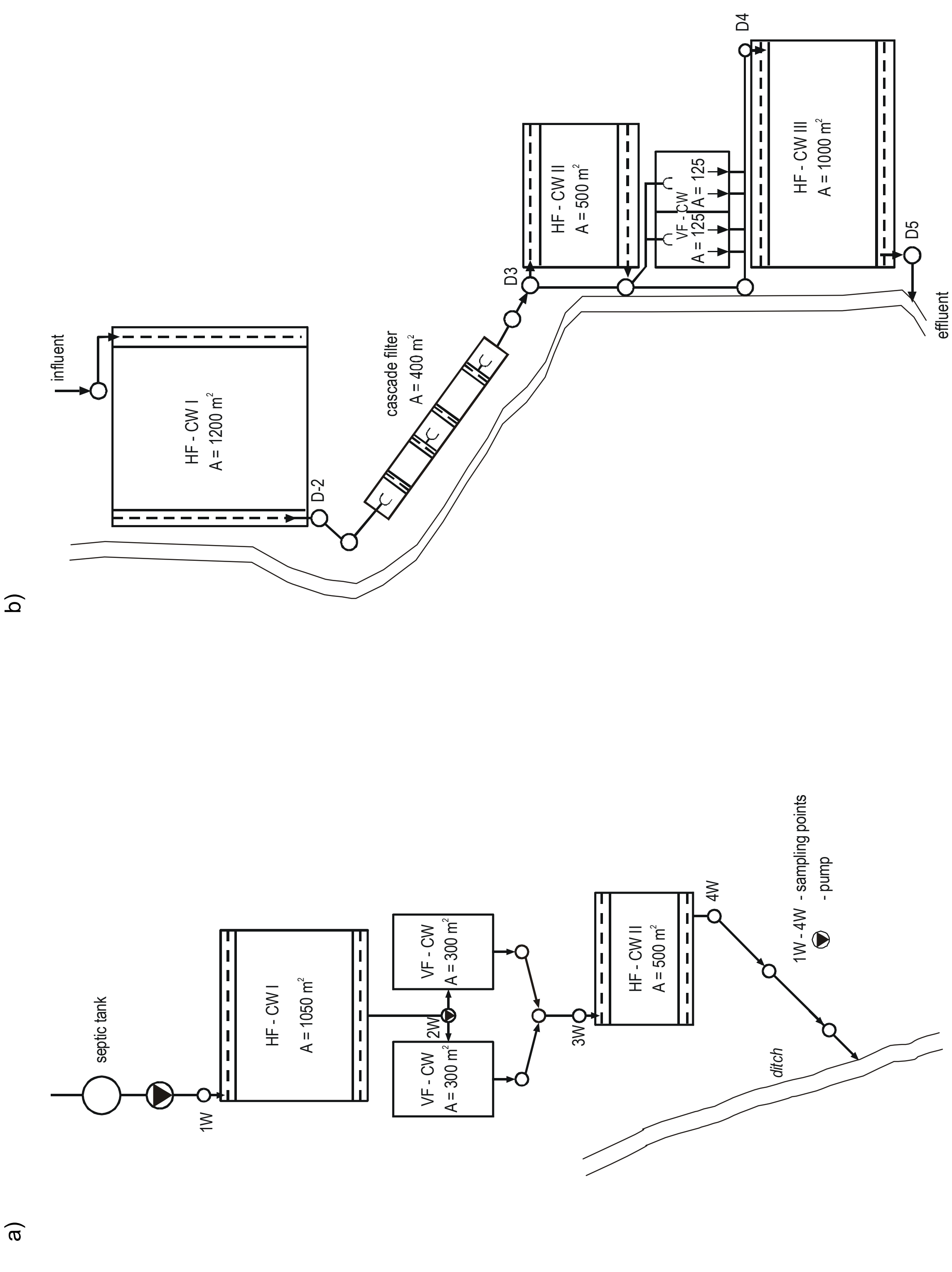

Fig. 1. The schematic of constructed wetland with sampling points in Wiklino (a) and Darżlubie (b)

The averages samples of sewage were collected twice a month in the period four years in Darżlubie and six years for facilities in Wiklino. The samples were collected after subsequent stage of treatment in Wiklino (sampling points 1W-4W) and in five sampling points along the Darżlubie facility (1D-5D) (Fig.1).

The measurement period was divided into vegetation (from April to October) and outside vegetation seasons (from November to March) for all analysed facilities. Time of vegetation season was estimated on the basis of vegetation time of common reed and air temperature. It was assumed that sewage temperature is not important criterion since temperature of sewage outflowing from septic tank in winter is elevated.

Measurements of physical and chemical parameters included: temperature of sewage and air, total suspended solids, BOD_5, COD_{Cr}, ammonium nitrogen ($N\text{-}NH_4^+$), nitrate, nitrite and organic nitrogen, total phosphorous, phosphates and alkalinity. All analyses were performed at Gdańsk University of Technology, according to the Polish Standard Methods. Removal efficiency was calculated as a quotient of contaminants concentration difference in influent (C_o) and effluent (C) after subsequent stages of constructed wetland and concentration in influent (C_o), $\eta = (C_o\text{-}C)/C_o$.

In order to estimate removal and transformation rate calculations of the decay rates of organic matter and both total and organic nitrogen were done. It was assumed that the decay can be described by the first-order reaction constant (Cooper 1998, Reed *et al* 1995). Based on the retention time and concentrations of organic matter and nitrogen the constant decay rates for the HF beds were found and corresponding modified decay rates for VF-CW bed.

3. Result and discussion

3.1 Wiklino constructed wetlands

During vegetation season the average amount of discharged sewage in Wiklino facilities was 19.0 m^3day^{-1} and was slightly higher than the average amount discharged in outside vegetation season (18.4 m^3day^{-1}).The average amounts of sewage outflowing from the facility were equal to 12.9 m^3day^{-1} in vegetation season and 13.9 m^3day^{-1} in outside vegetation one.

The average concentrations of organic matter (expressed by BOD_5, COD and suspended solids) discharged to the facilities in Wiklino was higher in outside vegetation season than in vegetation one (Fig 2a). The average concentrations of nitrogen compounds in sewage inflowing to the facilities during vegetation were lower than in outside vegetation season (Fig. 2b). Also in discharged sewage from the facilities the concentration of all contamination were slightly higher but still below permissible value.

Because of higher concentration of contamination and lower amount of sewage in outside vegetation season the load of contaminations discharged to the facilities in this season were higher than in adequate loads in vegetation one.

The efficiency removal of different contamination did not varied very much in both seasons (Fig. 3). The biggest different were observed for nitrogen compounds removal. In the cease of ammonium nitrogen the efficiency in vegetation season was equal to 85.1% and in outside vegetation season decreased to 78.3% (Fig. 3).

a)

b)

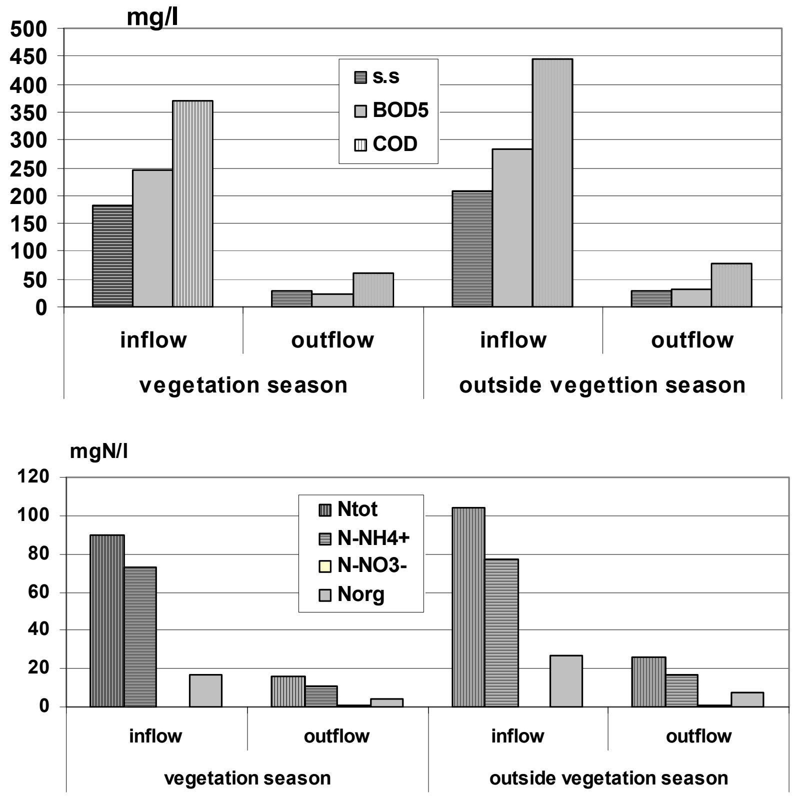

Fig. 2 The average concentration of organic matter (a) and nitrogen compounds (b) in sewage inflowing and outflowing from the facilities in Wiklino

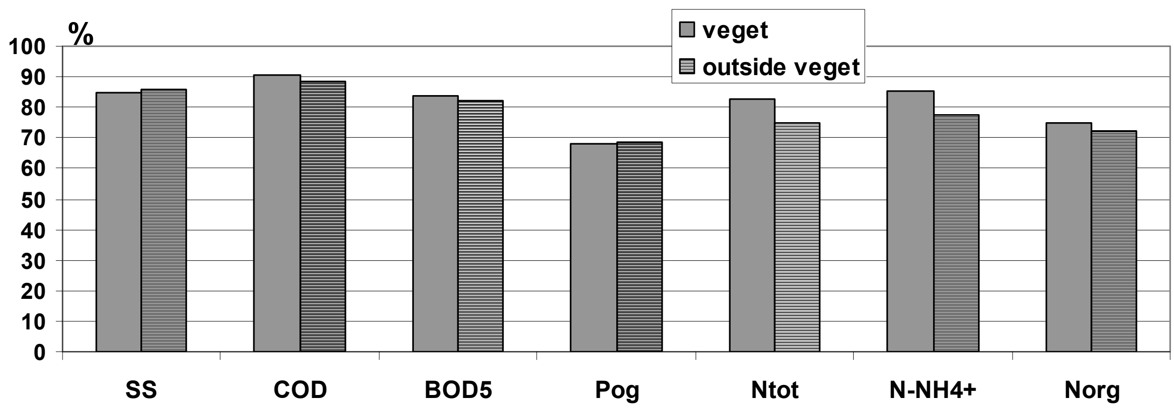

Fig. 3 Comparison of average efficiency removal of contaminations in vegetated and outside vegetation seasons in Wiklino facilities

The values of $k_{p(T)}$ for the temperature of 20°C, taking into account average monthly temperatures, were calculated using the relationship $k_{p(T)} = k_{p(20)}\ (1.1)^{T-20}$. The obtained average values of reaction rate constant for the HF-CW I and HF-CW II beds and corresponding modified constant rates for VF-CW bed are given in Table 2.

Table 2 The average values of analysed reaction rate constants in the temperature 20^{O}C, day^{-1} and $m{\cdot}day^{-1}$

REACTION RATE CONSTANTS	HF-CW I	HF-CW II	VF-CW
	VEGETATION SEASON		
$k_{pBOD(20)}/ k_{BOD(20)}$	0.122	0.27	0.031
$k_{pNtot(20)}/ k_{Ntot(20)}$	0.094	0.124	0.025
$k_{pNorg(20)}/ k_{Norg(20)}$	0.061	0.061	0.024
	OUTSIDE VEGETATION SEASON		
$k_{pBOD(20)}/ k_{BOD(20)}$	0.071	0.111	0.019
$k_{pNtot(20)}/ k_{Ntot(20)}$	0.045	0.062	0.019
$k_{pNorg(20)}/ k_{Norg(20)}$	0.048	0.052	0.018

The obtained values of decay rates of organic matter and both total and organic nitrogen that organic substances decomposition was the fastest process, while total nitrogen removal was slightly slower in both seasons. Mineralization of organic nitrogen was the slowest process, both during vegetation season and outside one. According to Obarska-Pempkowiak and Gajewska (2003) it was also proved that substantial part of organic nitrogen was amonificated in the septic tank. The temperature influence on the amonification process effectiveness was negligible. The average values of the constant rates $k_{pBOD\ (20)}$ and $k_{pNtot(20)}$ were by 42.0 and 52.0 % higher in the vegetation season than outside it for the HF-CW I bed. For the HF-CW II the average values of the constant rates were over 50,0% higher for the vegetation season than outside it.

The average values of modified constant $k_{BOD(20)}$ and $k_{Ntot(20)}$ in vegetation season were equal to 0.031 and 0.025 md^{-1}, respectively and were higher than in outside vegetation season. Similar results were obtained by Birkedal et al. (1993) for 37 VF-CW beds in Denmark. The average value of modified $k_{Ntot(20)}$ constant rate was 0.0247 $m{\cdot}day^{-1}$ (Birkedal et al., 1993). The values of $k_{BOD(20)}$ obtained during investigation carried out in Austria and Great Britain varied from 0.067 to 0.1 $m{\cdot}day^{-1}$ and were higher than the values calculated for the VF-CW beds in Wiklino. This could result from higher average air temperature (Harbel et al., 1998, Cooper & de Maeseneer, 1996)

3.2 Darżlubie constructed wetland

The seasonal changes of concentration of contaminations discharged to and from the Darżlubie facilities are presented on Fig. 4 The average concentration of contaminations discharged to the facilities in Darżlubie during outside vegetation season were lower than in vegetation season. The loads of contamination of inflowing sewage increased during the period of investigation. It was caused by pig manure begging discharged combine with domestic sewage from some farms. The agriculture production was more extensive during spring and summer and it caused higher concentration of contaminants during vegetation season. The biggest differences in discharged concentrations were observed for organic matter expressed by BOD_5 and organic nitrogen. During the vegetation season the concentration of BOD_5 and Norg were twice higher than in inflowing sewage than in outside vegetation season. The smallest differences in inflowing sewage was observed for total phosphorous: 8.5 mg P/l in outside vegetation season and 9.9 mg P/l in vegetation season.

a)

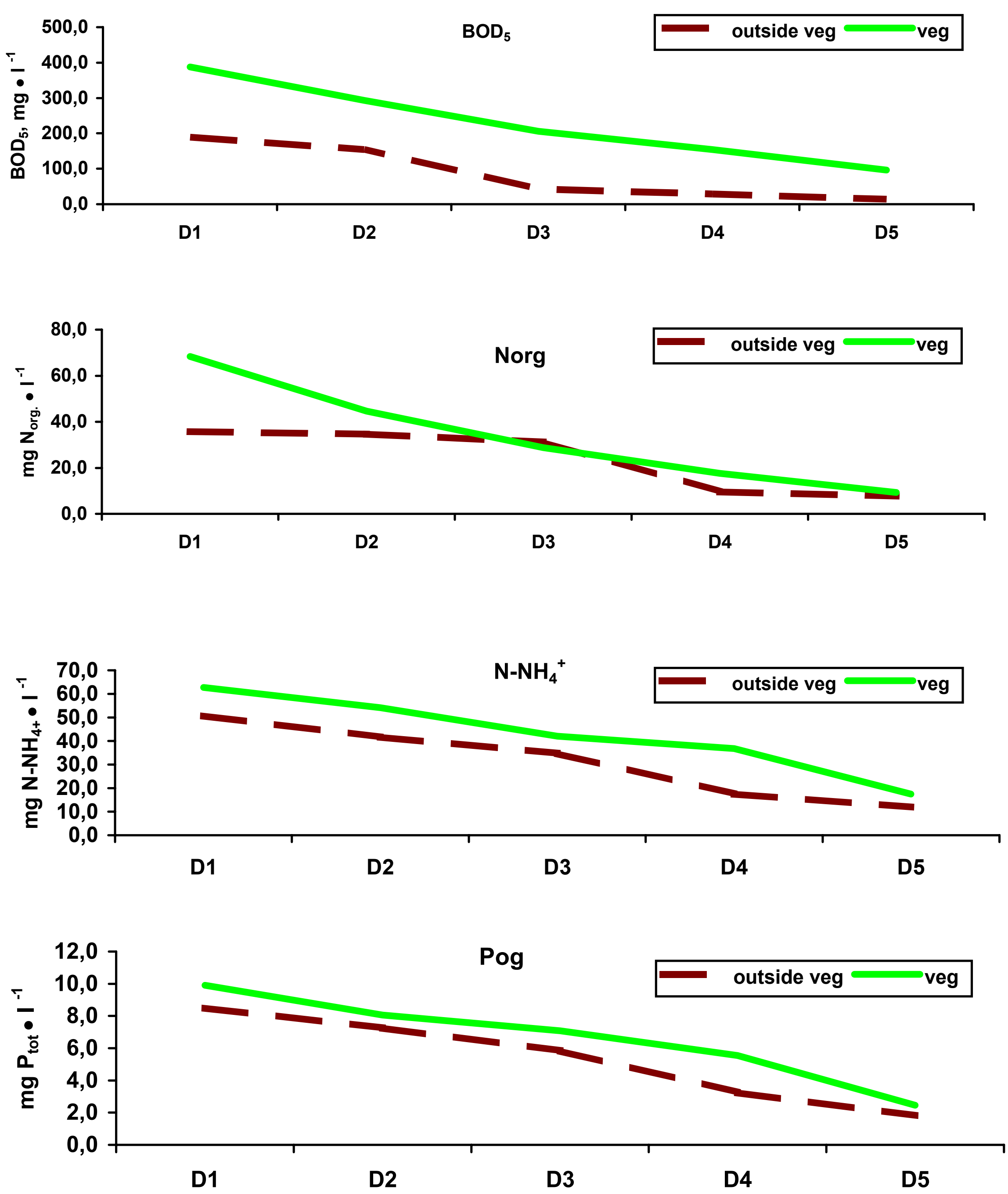

Fig. 4 The average concentration of organic matter (a), organic nitrogen (b), ammonium nitrogen (c) and total phosphorus (d) in sewage in Darżlubie in vegetation and outside vegetation season after subsequent stage of treatment (D1-D5)

The efficiency removal of chosen contaminations in vegetation season and in outside one is given in Fig. 5. During measurement period the efficiency removal of contamination was higher for outside vegetation period then in vegetation one. The biggest difference was observed for effectiveness of organic matter removal which was about 92.4 % for outside vegetation season and only 75.2 % for vegetation season. In case of ammonium nitrogen and total phosphorus the difference did not exceed 5%. It was probably caused by too high concentration of pollutants discharged during the period. High load of organic matter in colloidal form caused clogging of the HF-CW I filter and resulted in decrees of efficiency removal of all facility (Obarska-Pempkowiak and Ozimek 2000).

Only organic nitrogen was more effective removed during vegetation season (86.4%) than during outside vegetation one (78.5%).

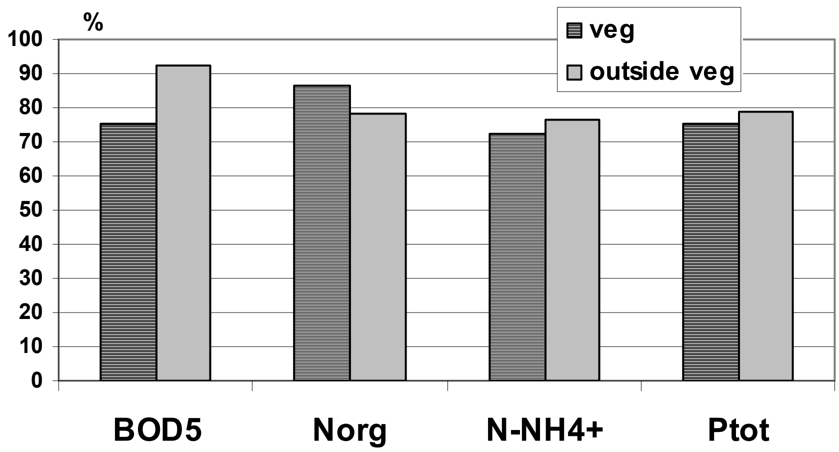

Fig. 5 Comparison of average efficiency removal of contaminations in vegetated and outside vegetation seasons in Darżlubie facilities

According to Billeter et all., (1998) efficiency of total nitrogen removal in Switzerland hybrid constructed wetland was equal to 86.0% and is very similar to those reported by Cooper (1998), Vymazal et all.(1998), Mæhlum'a et all., (1999), Kadlec (1995). According to Billeter et all., (1998) average efficiency of ammonium nitrogen removal varied from 13.0% to 96.0% for one stage CWs of HCWs, while organic matter removal varied from 78.1 to 96.8%. Hybrid constructed wetland in Oaklands Park described by Cooper (1998), ensured average efficiency of ammonium nitrogen equal to 78.1%. In Norwey average efficiency of ammonium nitrogen removal varied from 13.9% to 80.0% (Mæhlum et all.,1999).

According to Brix (1996) one stage constructed wetland are able to remove nitrogen with the rate in range 0.3 – 0.7 g N $\cdot m^{-2}$ $\cdot day^{-1}$. While according to Kowalik el all (1995) such constructed wetland are able to remove about 15 g COD $m^{-2}day^{-1}$ and total nitrogen in range 2 to 6 g Ntot $m^{-2}day^{-1}$.Hybrid constructed wetlands with HF-CW at the beginning of treatment are able to removed from 3.8 to 8.0 g COD $m^{-2}day^{-1}$ and total nitrogen from 0.27- 0.76 g N $\cdot m^{-2}$ $\cdot day^{-1}$ (Obarska-Pempkowiak et all., 2003, Gajewska et all, 2004). Effectives of total nitrogen and ammonium nitrogen removal was very similar and vearied in rang 85.6 to 95.9% for N_{Tot} and from 63.3 to 96.8% for $N\text{-}NH_4$+. While in HCW with VF-CW beds applied as a first (German objects) are able to remove from 15 to 23.5 g COD $m^{-2}day^{-1}$and from 0,6 to 2.05 g N $\cdot m^{-2}$ $\cdot day^{-1}$,and what characteristic ammonium nitrogen was remove with higher rate from 2.3 to 2.52 g $N\text{-}NH_4^+$ $\cdot m^{-2}$ $\cdot day^{-}1$ (Obarska-Pempkowiak and Gajewsaka, 2003, Gajewska et all, 2004). Configuration applied VF-CW at the beginning of biological treatment create favourable conditions for nitrification process and effective removal of organic compounds while total nitrogen was not so effective removed as it could be expected. Effectives of ammonium removal in HCW with this configuration was very high (over 90%) but total nitrogen was with lower efficiency from 33.4 to 64.6% (Gajewska et all, 2004).

4. Conclusions

1. Hybrid constructed wetlands are able to remove organic matter with daily rate from 3.8 g/m^2 (Wiklino) to 5.6 g/m^2 (Darżlubie) and for total nitrogen with adequate rate 0.8 g/m^2 and 0.43g/m^2.
2. Temperature of air and sewage did not substantially affect nitrogen removal. The average effectiveness of ammonia nitrogen removal in vegetation were about 85% for Wiklino facility and 72.2% for Darżlubie, while in outside vegetation were respectively 78.0 % and 76.4%
3. The decrease of effectiveness of nitrogen transformation process in outside vegetation season is confirmed by the values of reaction constant rates which in case of Wiklino facility were equal to 60% of the values reached in vegetation season.
4. In case of Darżlubie improper maintenance of the system caused clogging of same filters and resulted in decrees of efficiency removal of the whole facilities.
5. The outflow from Wiklino in both seasons did not exceed new polish standards while in Darżlubie concentration of organic matter (BOD_5)in vegetation season exceeded permissible value due to huge load of organic matter and nitrogen caused by inflow of pig manure in inflowing sewage.

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Modernization of Water Economy and Environmental Awareness

Marija Vukelic-Sutoska

Abstract: In this paper are presented issues related to the plan modernization of agro, food and water economy, to have development plan for the structures within the frames of agriculture, basic characteristics of agriculture, basic characteristics of the water economy, irrigation possibilities, technical basis of water economy structures according to the programme, protection against floods and erosion, environmental protection and improvement, water protection and the best management practices.
Keywords: Modernization of agro, food and water economy, best management practices, environmental awareness, and waste management

1. Introduction

Tremendous successes have been obtained in Macedonia, but in many areas water irrigation systems function below what could be expected. Having in mind the required increase in food production in the future with the need for sustainable agricultural development a wide range of issues is of major importance. One of them is the influence of the agriculture on water management in the conditions of agricultural pollution. Development of agriculture in Macedonia is followed by the water management plans with respect to water protection and influence of water management only. There were many factors which influenced the point abstraction of water and also the local pollution increased in much larger scale.

The development after 1990 has been characterised by the process of transformation of agriculture, privatisation and restitution of the property. The most important sources of pollution are high capacity pig farms with the sludge production, pollution by accident such as wash out of organic fertilizers., the disposal of sludge and farm manure on the frozen soil surface and others. The application of drip irrigation, water conveyance through pipe lines rather than through open and unlined canals, and other efficient water supply techniques are expected to lead to major changes in the design of agricultural projects. Utilization of aquifers continues to accelerate to meet the needs for irrigation, industrial, urban and suburban water supplies. Groundwater is generally a renewable resource, however, the natural supply of groundwater is limited as to time and space. One of the most important issues in water resources research is the management of groundwater systems in order to avoid or minimize bed effects on the environment and to maximize economic benefits. The accurate planning of water resources systems is a complex interdisciplinary problem which involves complicated environmental, ecological and economical aspects.

2. Water management

Arable agriculture land in the Republic of Macedonia is estimated at 612,000 ha. With development of more reservoirs, it is possible to secure irrigation of realistically 370,000 ha. To date, a total of 106 reservoirs of various size have been built, and they cover 163,693 ha of fertile arable land, i.e. 24.5% of the arable land. Out of the total land for irrigation (163,693 ha), 100,000 ha (61%) can be irrigated by sprinklers, and 63,300 ha (39%) simply by gravitation. Because of underdevelopment irrigation systems, land given for other purposes etc, it is realistic to expect irrigation of 126,617 ha (77.35% of the irrigation-covered land for which there exists a technical documentation), or 19% of total arable land due to under developed irrigation systems and other land given permanently for other purposes. Exploitation level varies greatly, and it ranges from 40 to 70%. By completing the rehabilitation project for the system HMS Bregalnica, Tikves, and Polog, a significant increase of exploitation level is expected. The largest irrigation systems are the following: Bregalnica (28,298 ha), Strezevo (20,200 ha), Tikves (19,225 ha), Strumica (16,717 ha) and Kumanovo (10,000 ha). Water supply for population and industrial facilities is usually performed by (separate) local water supply systems. Annual population demand for water is approximately 440 milion m^3 of water, and annual industrial demand is 534 million m^3 of water. Eightly four 84% of the whole water capacity in Macedonia originates domestically, while are, or flowing in from the neighboring countries. There are three natural lakes in Macedonia: Ohrid Lake-total area 348,8 m^2 , out of which 229,9 km^2 in the Republic of Macedonia, Prespa Lake-total area 274 km^2 , out of which 176.8 km^2 in the epublic of Macedonia, Dojran Lake – total area 43 km^2 , out of which 27.4 km^2 in the Republic of Macedonia. There are 19 big and over 100 small water dams built for utilization of the 4,414 registrated springs and the hydro potential of the rivers. Their total volume is 1,854 m^3 of water. Also there are facilities (green house, spas, pools) in the Republic of Macedonia where geothermal water is used for heating.

3. Structures within the frames of the Vardar river valley programme

3.1 Development plan

In its development plans, Republic of Macedonia has always shown a particular interest in the Vardar river valley area. The changed conditions in its economy and the possibilities offered by the new Law on investment of capital of joint ownership, the democratisation of the society and the introduction of the market economy have created conditions for a new approach to future strategic interest in this programme since the Vardar valley coincides with the European development corridors.

Energetics-from energetic aspects, tho construction of the hydro-electric power plants along the Vardar river valley within the period 2010-2015 represents a basis for long-term development of the electric power system of the Republic. The studies and projects that have been carried out so far point to the need of using the hydro-potential of the Vardar river and its tributaries. To that effect, 12 hydro-electric power plants are planned to be constructed along the Vardar river course, from Skopje to the border on Greece, 200 km length. Two hydro-electric power plants are envisaged to be constructed on each of the tributaries-Treska and Crna Reka, or a total of 16 hydro-electric power plants with a total installed power of aggregates of about 1000 MW and annual producrion of 2.2×10^9 kWh.

Irrigation-The total water potential of Republic of Macedonia in an average year amount to 7.8 millions m^3, out of which 5.6 millions^3 from the Vardar river valley, the Crn Drim river with 2.2×10^9 m^3. There is a possibility for irrigation of 370000 ha, or 55%, out of the total of 670000 ha arable land. So far, irrigation systems have been constructed for 173000 ha, out of which 40000 ha in the Vardar river valley. The construction of multi-purpose hydro-systems in this area shall enable irrigation of another 70000 ha, water supply for the population and the industry, electric power production, fishing, tourism, protection against floods and pollution, as well as amelioration of small waters.

Environment-The protection and improvement of human environment issues are covered by protection of water and land, seismic hazard and protection of cultural monuments. The Vardar river is the main recipient of the basin area, covering 80% of the territory of Republic of Macedonia. Therefore, from ecological aspects, the main emphasis is put on protection of the waters of the Vardar river and its tributaries. Protection is planned to be realized by construction of 135 stations for treatment of industrial waters and 9 complexes for treatment of waste waters from urban settlements.

3.2 Water economy, agriculture and irrigation possibilities

The initial waters of the Vardar river are used directly for irrigation of about 8000 ha, about 30 ha for carp fish ponds and 5.5 m^3/s for industry and for the Negotino steam power plant. The use of Vardar river waters is of particular significance for development of the economy. Considering the non-uniform distribution of precipitations and the annual flow, the regulation of the Vardar River water regime, its rational multi-purpose use, is an important factor for further development of the economy in the Vardar valley.

- The hydro-amelioration systems (HMS) in Macedonia, depending on the way of using waters for irrigation (running waters, reservoirs, lakes and groundwater) represents a technical,technological and economic entity.
- The existing potential of (HMS) covers 173000 ha, and it consists of: 16 irrigation systems encompassing 133000 ha by catching 731.5 x 10^6 m^3 of water from reservoirs, 42 irrigation systems covering about 20000 ha, pumping water from natural water courses, reservoirs and lakes, and 48 irrigation systems covering about 20000 ha, in which water is taken from natural water courses, by gravitation. During an average dry year, it is possible to provide irrigation from the existing hydro-amelioration systems of as much as 6)% of the irrigation areas. The water supplying system, depending on the way of water catchment and the available specific yield of the existing capacities, mostly, do not provide the demands of the population and the industry.
- The water potential represents a limiting factor in the Republic of Macedonia. To satisfy the needs of all users until 2025, it is necessary to elaborate a balance of the available waters, a balance of the user's needs and their coordination. The Vardar river with its tributaries, the rivers of Treska, Pcinja, Bregalnica and Crna Raka, are only partially used for irrigation. The total technical irrigation potential of the Vardar river basin with the tributaries is estimated at about 293587 ha, while the economic, used potential is about 173000 ha or 69%. On the basis of the development plans for irrigation from 2010 to 2015, and the existing constructed facilities, it has been estimated that there are realistic possibilities for covering of new areas for irrigation of about 300000 ha. The water economy study of the Republic of Macedonia, as well as other water economy studies, projects and development plans, point to the need for direct utilization of the remaining water potential of the river basin, respecting all the future users.
- The implementation of the water economy within the scope of the Vardar Valley Programme in the period 2010-2025, from the water economy aspect will represent the basis for a long term and well conceived development plan. The phase construction of water economy structures and facilities (irrigation systems, electric power production systems, protection against floods, water protection against pollution, anti-erosion measures, etc) and their continuous operationalization and exploitation, creates possibilities for opening of new capacities (electricity production, irrigation of new areas, etc). The implementation of this complex programme will be reflected in covering of new areas for irrigation and other water economy facilities for protection against flood.

3.3 Technical basis of water economy structures according to the Vardar valley programme

By the Vardar Valley programme it is foreseen utilization of the water potential of the Vardar river and its tributaries, the Treska, Pcinja and Crna rivers. For hydro-amelioration system are planned to be constructed along the Vardar river water course from Skopje to the Greek border, covering 54282 ha of arable land and water supply systems (Skopsko pole-19600, Lisice-3780, Pepeliste-1506, Desnica-8000, Gevgelisko pole-8030, and Kumanovsko pole-11000 ha). Here are also other smaller (HMS): Skopsko pole-1993 (small reservoir), and Kumanovsko pole (small systems)-2656 ha.

Water supply of population and industry in the Vardar valley is a problem, requiring manyfaceted investigations in future for the consumption centres of Skopje, Kumanovo, Veles, Negotino, Gevgelija and many villages settlements. Consedireing the limited possibilities of the natural sources and the underground waters, the water supply of population and industry in future can be resolved by water catchment from groundwater and reservoirs.

By the future development of water supply, it is planned to satisfy the needs of population and industry: Skopje-source of Rasce, Kozjak and Matka reservoirs on the Treska river and Paligrad on the Kadina reka, as well as groundwater, Kumanovo-Slupcanska reservoir and groundwater, Veles-Lisice reservoir on the Topolka river, Babuna and Basino on the Vardar river, Negotino-Dopsnica reservoir on the Dosnica river, Gevgelija-Konjsko reservoir on the Konjska river, and Valandovo-groundwater and natural wells.

3.4 Protection against flood and erosion

In future, it is necessary to increase the percentage of protection against floods, which will be provided by construction of protection embankments and by regulation of the water regime of the Vardar river by construction of reservoirs in the Vardar valley. The regulation of large waters and flood protection is also of interest for the Republic of Greece since part of the areas of the Vardar river basin in the territory of Greece are also susceptible. In addition of flood protection, in future, it is also necessary to undertake measures for preventing land erosion, such as special ways of land cultivation, afforesting of erosion-prone land, rehgulation of torrents, etc.

4. Environmental protection and improvement

4.1 Water Protection

The Cadastar of Pollutants of Vardar River was elaborated in two phases: in the first phase, a poll was conducted through all the inhabited places and industrial facilities which are a constituent part of the urban areas along the Vardar river course, and in the second phase, based on the conducted poll, a selection of industrial capacities classified as pollutants and potential pollutants was made and complete technical documentation on the state of their waste waters was prepared. According to the knowledge acquired during the visit to the populated area and industrial capacities presented in the Cadastre of Pollutants of Vardar River it may be concluded that the state regarding the waste waters related in this recipient is critical to the extent of a catastrophe due to the following: central waste waters filtering stations do not exist in any of mentioned populated areas, the industrial capacities release their unfiltered waste waters partially in the urban sewerage systems and partially drectly into the recipient, there are systems for filtering of waste waters at the level pre-treatment or complete treatment in a small number of industrial capacities (however, these systems are pretty old and non-efficient and some of them are even non-usable so that the waste waters are mainly released through the existing by-passes), in the inhabited places and industrial capacities there is a partially constructed sewerage infrastructure mainly of a mixed type through which the complete amount of fecal. Technological and atmospheric waters are released directly into the river, and the depositing of the communal and industrial waste materials in the towns is inappropriate and not well organized (it is performed in places which do not satisfy the health department regulations so that they add to the pollution of the total environment, especially air, soil and ground water).

After the accomplishment of the Cadaster of Pollutants of the Vardar River, there exist possibilities for realization of the second phase of the Programme of Measures and Activities for Protection of the Waters of Vardar River and its ttributaries Against Pollution which involves elaboration of conceptual solutions for filtering of the total amount of waste waters from the inhabited places. For purpose of obtaining of the most favourable economic-technical solutions, it is necessary to elaborate seceral variant solutions of central filtering stations with an emphasis on measures to be taken for industrial waters.

4.2 Best management practices for nutrient and irrigation management in the Vardar river valley

Best Management Practices (BMP) are recommended methods, structures, and/or activities designed to prevent or reduce water pollution while maintaining producer profits. The goal of BMPs is to protect Vardar river valley water resources from degradation, while maintaining the economic viability of agriculture and related industries. Ideally, these practices will improve producer profitability at the same time soil and water is protected from contamination. Success with voluntary BMPs will depend upon how many farmers and agricultural chemical applicators actually use and promote them.

Having in mind these recommendations of BMPs and Figures 4.1 and 4.2 one may say the following about The Best Management Practices:

INTEGRATED CROPLAND CULTURAL PRACTICE (Crop Relation BMPs, Soil Management BMPs, General Nutrient Management BMPs), NITROGEN FERTILIZER MANAGEMENT, BEST MANAGEMENT PRACTICES FOR NITROGEN FERTILIZATION (Nitrogen Application BMPs, Fertilizer Handing and Storage BMPs), PHOSPHORUS MANAGEMENT (BMPs for Phosphorus Fertilization), MANURE AND ORGANIC WASTE UTILIZATION (BMPs for Manure Utilization,Manure Storage BMPs), IRRIGATION MANAGEMENT (BMPs for Irrigation Management, Flood or Furrow Irrigation BMPs, Sprinkler Irrigation BMPs, Chemigation BMPs).

5. The main strategic and reform objectives

It is necessary to design and create a modern, stable and dynamic agricultural sector, integration of the Macedonian agriculture into the global development processes, development of the rural areas and establishment of favourable economic conditions for the Macedonian farmers, improve the marketing of the agricultural commodities, development of the regional cooperation, and implementation of the action plan for execution of the Agreement for Stabilization and Association with the Eurepean Union. Significant improvement is achieved by carrying out the activities for reform and harmonization of the legislative to the one of the EU, as follows: private farmers are allowed to have access to public land by distributing 27,5000 ha of arable land to new users, denationalization of land previously, natinalized from original owners, privatization of companies and transformation of agricultural cooperatives are in their phases, newly passed regulations now regulate usage of goods/assets of public ineterst, such as water, forests, pastures, agricultural land, fish farms and wildlife, veterinary and plant protection regulations are getting approximated with those of the EU. Border cross veterinary and sanitary inspection teams are trained and appropriately equipped. Institutional and financial support has been obtained for opening of 37 new veterinary facilities, with an aim to improve the quality of service and enhancing the competitiveness, a national system for identification and registration of livestock is getting introduced, structural changes have been made in the National Extension service, after the technical equipment process will be finished, permanent training of 78 agronomists will continue, as well as implementation of the state project for their (Extention service) subsequent privatization and self-financing by providing services on farms, support and encouragement to the formation of farmer associations, a total of 178 local associations have been formed by 2002, 14 regional unions and 9 national unions, all of these are united into a Federation of Macedonian Farmers, representing approximately 25, 000 farmers, the process of liberalization of the agricultural commodities market goes on, in accordance to the Agreement for Stabilization and Association with the WTO, free trade agreements have been signed with several countries in the region, the system for preferred crediting has been abandoned, and all subsidies, as well as stimulations that have been in place since 1996 have been terminated, after the subsidies have been abandoned, financial support to the agriculture continued through programs by which, during the period 1997-2001, a cumulative amount of 25 million EUR has been invested, in order to support the agricultural development, a Fund for agriculture has been founded, and a special Fund for water resources for the water management.

In Figure (1) is shown Framework for analysis in water resources planning studies

Conclusion

People in the Vardar Valley have a strong interest and extraordinary knowledge of their surface and groundwater resource. Agricultural producers are leading the effort to protect and wisely utilize the water that makes life possible for people, crops and wildlife ecosystems within the Valley. The goal of this paper is also to prepare a review of Best Management Practices (BMPs) containing nutrient and irrigation guidelines and recommendations.

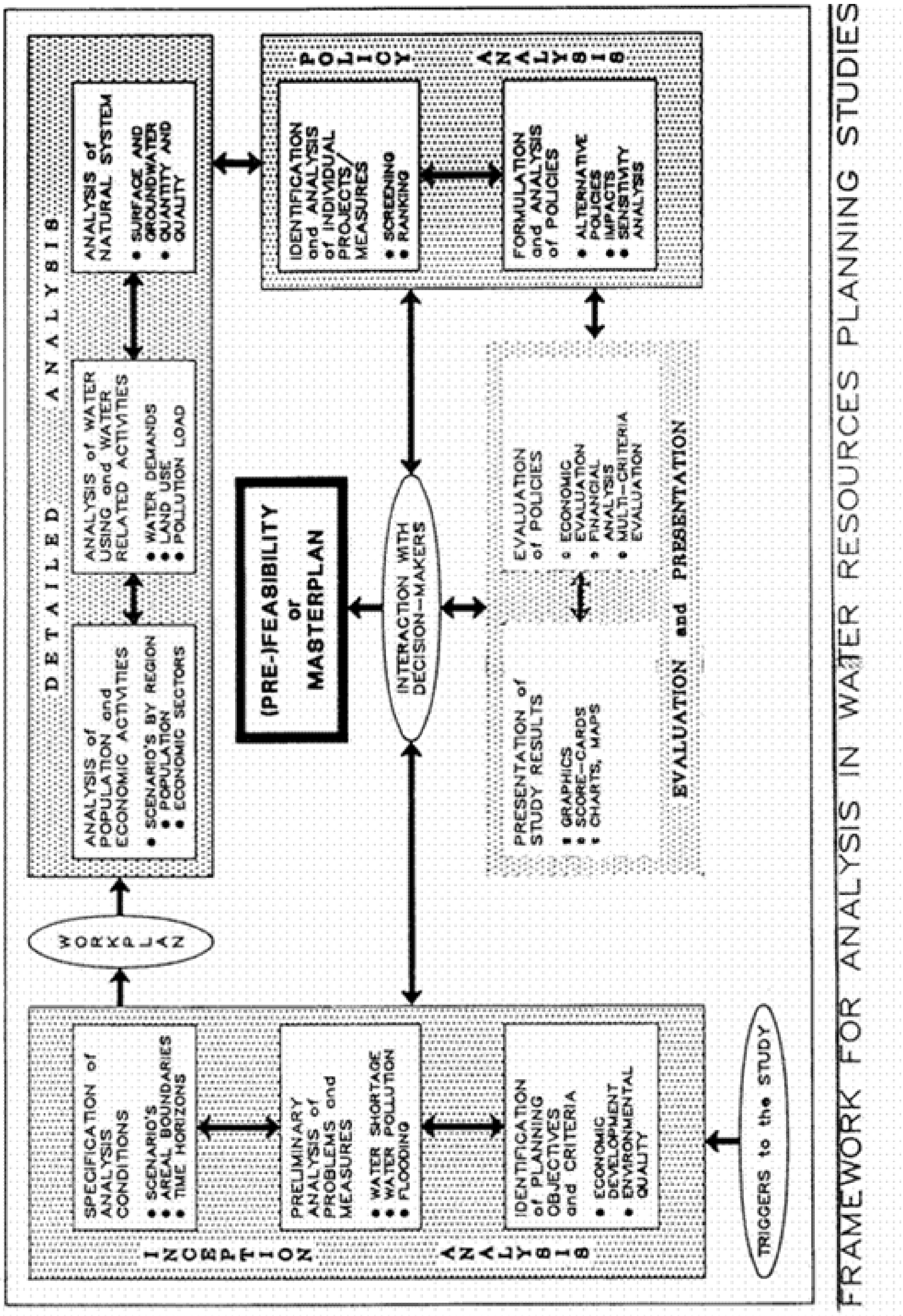

Figure 1 Framework for analysis in water resources planning studies

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The Remediation of the Adverse Effects of the Gravel Extraction in the Mostarsko Blato Catchment Area

Mladen Zelenika, Božo Soldo, Damir Štuhec

Abstract: Extraction of alluvial deposits at the weastern part of Mostarsko Blato, the gravel and sand from the Listica and the Mokasnica riverbed and surrounding area have adverse environmental impacts on quality and quantity of groundwater in use of the rural population. There are already very bad consequences for the private wells and the understandable complains to the authorities by fisherman, hunters, villagers and evironmentalists. The sink river water level and groundwater level in soil, dry water supply wells and reduce the productivities of gardens, and the compains are related to devastation of aquatic species, endemic plants and animals there, unpleasant noise from excavators, lorries etc. The field reconnaissance and complex analyzes of present environmental conditions at the devastated and surrounding unharmed areas is recommended including the documents related to pre-existing environmental conditions and establishe the systematic measurement of surface water level, groundwater level, level of sediments in the Mokasnica and the Listica, and changes in quality of water and biological productivity of soil at Project area. The mining operations should be reduced and stopped there and introduce the relevant remediation actions.
Keywords: M. Blato, gravel, hydrological - hydrogeological data, object, environmental.

1. Introduction

Dinaric karst in Republic of Croatia and Bosnia and Herzegovina are characterized by a great variety of natural environments and hence by an equally great variety of environmental problems. Catchment area Mostarsko Blato extends over more than 530 square kilometers, and approximately 100000 people reside in this area. The most important element of life in catchment area Mostarsko Blato is unquestionably water, due to its abounded quantity during winter and scarcity during summer.

Besides many periodical streams, the perennial streams: Listica, Crnasnica, Orovnik and Zvatic are flowing in this karstic catchment area to a naturally closed valley (polje) Mostarsko Blato. All area is scarce of water during the summer, but in the rainy season (winter), there is a flooded (lake) area of more than 25 square kilometers. Natural sink-holes (ponor) and for the last 50 years an artificial tunnel, evacuate all water to 200 m lower Jasenica river in the vicinity of only 5 km (see attached map).

Exhaustion of gravel and sand from the Listica and the Mokasnica riverbed sources have adverse environmental impacts on quality and quantity of groundwater in use for rural water supply. This is one of very bad consequences of three years of war and conflicts in the region. Mayor routeways, activities, towns and villages are sided close to the Listica and the Mokasnica streams, where from come understandable complains to the authorities about devastation of aquatic species, including endemic plants and animals. More and more people demand the stark complex natural beauty and the peace and tranquility afforded by rivers. It is unfortunately replaced by lowered water level in the rivers and soil, dry water supply wells and land, unpleasant noise from excavators, lorries etc.

Only few local contractors have short-time license to mine gravel and sand there, but the mining continues almost 10 years, even it should be stopped, and demand of the aggregate should be met using the crushed rock (limestones) at an adequate site. The present devastation should be urgently surveyed and analyzed, designed the most effective remedial measures ad recommendation. Implementation may take place at the only those sites where the methods of work and process are acceptable for all users of water, land, forest etc.

2. Hydrometeorological and climatological data

The climate in the area of Mostarsko Blato has mixed mediterranean and continental characteristics. It is generally humid, with moderately dry summers. The average yearly precipitation is 1.594 mm/year. The minimum, during the analyzed period between 1989 and 1961, was 910 mm (1938), and the maximum 2748 mm (1900) (Zavod za hidrotehniku, 1963). The average number of days with precipitation in a single year is 140. The precipitation in December is four times bigger than in July and the total average precipitation for the period between November and January (693 mm) is three times bigger than in the period from June to August (227 mm) (Energoinvest, 1981). When these data are combined with the effects of increased evaporation and evapotranspiration in the summer months it is clear why problem of seasonal water shortage exists in this area.

The average air temperature in the area of Mostarsko Blato is 14 C^0, the minimum is –15 C^0 and maximum 42 C^0. Within each year there are approximately 40 to 50 hot days with temperatures above 30^0 C and only 1 or 2 days in winter with maximal daily temperatures bellow 0^0C. As per meteorological observatory Mostar, the yearly aerodynamic conditions in the Mostarsko Blato are as follows: There are only 60 quite days/year, approximately 300 windy days/year (bove 4 m/s), 160 days/year (above 12 m/s),) and 20 days/year (above 20 m/s), strong wind blows more than half time and it is mostly “bura”, the cold northern wind typical for this region.

3. Hydrological data

The estimated orographic drainage area of the Mostarsko Blato is around 530 km$^{2.}$ However due to the possible underground connections, orographic boundaries may not coincide with the actual limits of the Mostarsko Blato catchment area. The inflow of four rivers Listica, Mokasnica, Crnasnica, and Zvatic in to the Mostarsko Blato, (including the water from irrigation channels), has been estimated as 534 x 106 m^3 or approximately 16,6 m^3/s. However, this value must be increased for the substantial amount of water brought in by numerous streams that flow directly into the Mostarsko Blato, and drain approximately 180 km^2 of the drainage basin (Orovnik, Blaz, Gromolj, Kacun, Zelenikove Babe, Đolina Draga, Zvec, Dragana, Zukanovina, Arine and other).

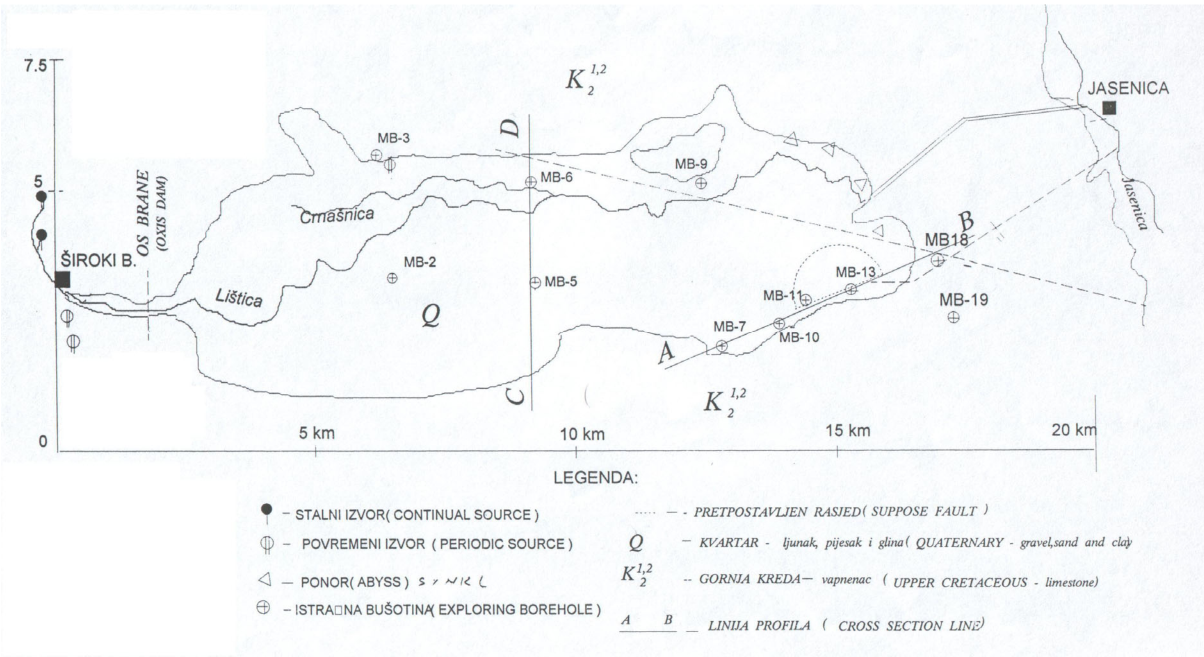

Figure 1 Sketch of exploring works and hydrogeological appearances

The water that leaves the polje through swallow holes (ponor) and the evacuation tunnel with a capacity of 40 m^3/s is diverted to the river Jasenica. The value of 16,6 m^3/s for the average flow of Mostarsko Blato, that was determined on the basis of existing hydrological data is substantially higher than the flow of the river Jasenica, 10,92 m^3/s, and the difference between these two values is retained in the polje in a form of floods. Flood waters occur yearly in the Mostarsko Blato area, in registered amount of HQ/20=258 m^3/s.

Four main monitored watercourses: Listica, Crnasnica, Mokasnica and Zvatic provide bulk of the water in the polje, and their total Q_{max} equals 339,16 m^3/s, (Energoinvest, 1981). Ther monthly average flow rates are shown in Table 2. The table also contains information about periods of observation for a characteristic entrance profile and position and elevation of the respective gauging station.

Valuable hydrological data for the area can be obtained from six gauging stations on the streams Ugrovaca (1), Listica (2), Crnasnica (1), Mokasnica (1) and Zvatic (1) and four gauging stations on the points of outflow from the polje: Krusevo swallow hole, and Jasenica (3) (Energoinvest, 1981). However, the data from the existing measurement stick in the Mokasnica and Mokro polje are not available and information similar to those presented in table 1 are nonexistent for the numerous smaller streams, which strongly influence Mostarsko Blato.

Table 1 Mean monthly and yearly inflows in Mostarsko Blato

Flow rate (m^3/s)	LISTICA	CRNASNICA	MOKASNICA	ZVATIC	PAHE M.B.
Profile	Uzar. Bridge	Knespolje	Jare	Zvatic	Σ
Elevation of “0” profile	240,56	236,35	232,52	232,74	(¨>222,20)
Period of observation	1965. –1979.	1967.-1979	1926-1979	1926-1979	1026-1965
January	27,82	0,928	1,790	1,170	18,40
February	26,79	0,947	1,800	1,240	16,40
March	17,40	0,840	1,290	0,966	21,10
April	16,40	0,850	0,936	1,030	25,60
May	13,46	0,748	0,731	0,695	23,30
June	4,86	0,540	0,173	0,371	11,00
July	1,35	0,348	0,038	0,190	4,00
August	1,46	0,274	0,042	0,139	1,30
September	4,00	0,391	0,152	0,337	2,70
October	8,44	0,579	0,457	0,419	12,00
November	21,60	0,815	0,828	0,965	27,00
December	28,08	0,950	1,410	1,080	26,50
Q_{year}	14,30	0,684	0,766	0,716	15,76
Q_{max}	319,46	3,050	12,580	4,070	339,16
Q_{min}	Dry	0,03	dry	0,00	0,03*

*Approximately 1m^3/s of water is taken through irrigation channels away from river Listica and they are not included in the values from the table

Values of the basic hydrological parameters have not yet been determined for the Mostarsko Blato area. Presented values are from literature and pertain to Adriatic part of Croatia and Bosnia and Herzegovina (Sekulic and Vertacnik):

Average precipitation: 1,385 mm/year - Evapotranspiration 545 mm/year - Average inflow by surface streams: 554 mm/year - Direct flow in to the see 23 mm/year - Subsurface drainage 225 mm/year - Submarine springs 41 mm/year.

4. Hydrogeological data

There are two collectors of underground water in the Mostarsko Blato area: **Cretaceous limestones** (Mesozoic) and **Quaternary deposits of gravel, sand** and limestone debris. Smaller amounts of underground water can be captated also in the Tertiary limestones and marls (Cenozoic) and these deposits have a significant meaning for rural water supply (i.e. Dobrinjska Draga at Crnac or excavated wells near Privalj).

In the periods of heavy precipitation, intermittent springs and rain water flow in the streams of: Ugrovača; Mostina, Dragana, Zukovina, Zvec, Zelenikove Babe, Kacun, Gromolj, Blaz, Mokasnica etc. Thes intermittent springs are related to the cretaceous (upper) aquifer. The perannual springs of Listica, Crnasnica, Zvatić and Orovnik are also related to the cretaceous aquifer

It has been noted that despite aerial distance of only 10 km between bore holes PB-5 in Dobric and PB-10 in Ljuti Dolac, difference in the water levels between the two bore holes reaches in dry season up to 87 meters (Sliskovic & Zelenika, 1998). This phenomenon indicates that, western part of the polje is completely isolated from the eastern part by the deposits with smaller permeability. These sediments include 70 m thick Qaternary impermeable clay deposits, some possible marl deposits of Neogene age and probably clayey limestones of Cretaceous or sometimes Tertiary age. Existence of such layers is the main cause for the formation of the springs of Crnasica, Zvatic and Orovnik and for the permanent flow of Mokasnica in the lower part.

The deposits of the Quaternary age have thickness between 10 and 70 m. and even 150 m. They are discordantly deposited over the paleorelief formed on the Mesozoic carbonates.

Quaternary deposits consist mainly of gravels and sands, brought by the rivers in the western part of the polje and by the longer nonpermanent flows along the edges of the polje. Clays and other lake sediments occur in the eastern and central parts of the Mostarsko Blato, and carbonate debris mixed with clay is present along its edges at the various depths.

Vast areas covered with fine sand occur on the southern side of the polje in the area of Podgorje, Ljuti Dolac, Jare, and Biograci, (Zukanovina, Kraljevine, Odanci, Dubovi, Lokve, Njivice etc). The sand was brought in to the polje by rivers and streams and deposited during flood periods through the wave activity induced by the “bura” wind. In the dry periods force of wind a main agent controlling sand distribution.

Wells with depths from 1.5 to 5 m have been excavated along the edges of the polje and used for the water supply of local rural population. Tens of such private wells existed in Knespolje, Uzarici, Jare, Biograci, Ljuti Dolac, Polog, Dobric and Gradac. Earlier, the wells have been dug manually but now they are mostly excavated with mechanical equipment, usually on the approximate elevation between 225 and 235 m. Groundwater is captated in the alluvial aquifer, consists of gravels and sandy gravels, rarely in debris, at the depths of 2 or more meters. It is used for the irrigation of the agricultural surfaces (vegetable) and for the supply of the households in the periods when “cisterns” (reservoirs for collected rain water) go dry. This thin aquifer extends from the central to the southern, western and northern edges of the polje. However, it thins gradually, and disappears around Kaluza in the eastern direction. Gravels and sands used to be a natural source of water for the springs of the important peranual tributaries to lištica and Mokašnica rivers were flowing through the central areas of the polje. Zones of swallow holes is in the eastern part of the Polje, near Polog and in the larger part of the area near Krusevo, carry the water from the Mostarsko Blato towards to lover parts of the river Jasenica/Neretva.

5. Hydrotechnical objects in the MostarsKO BLAto

During the dry summer season hydrotechnical measures (irrigation) are necessary in the area of Mostarsko Blato to ensure water for agricultural production. One of the first such projects in the area, was construction of the channel that takes water from the lower part of the river Mokasnica, near permanent spring Bilila, towards southeast, and irrigates some plots in Biograci. An earthen dam, approximately one meter high, was built for this purpose in Mokosnica river upstream from the spring. At the time of its construction, the spring has provided minimal amounts of water to the lower part of the river and the overall effects were relatively favourable. During 1950 earthen dam was repleaced with a concrete one which collapsed 15 years later due the weak footing.

However, remaining water was later diverted from Mokosnica and its tributary at locality Kaluža to irrigate downstream fields in Ljuti Dolac. The consequence of this measure was drying out of the lower flat part of the river with the devastating effects on the aquatic life and fish population.

The tunnel with a length of approximately 1527,7 m and a connecting channel with a trapezoid cross-section (196 m long) take flood waters from Mostarsko Blato at the elevation 222 m near Krusevo to river Jasenica at elevation 48,30 m.

The new 9488-m long drain passes trough the center of the Mostarsko Blato and it has numerous harmful effects for the environment in the polje. The lower parts of river Moksnica become dry, and generally, the level of groundwater has been lowered for 1,5 m to 2,5 m causing destruction of numerous unique marsh habitats at Mostarsko Blato.

6. Problem analyzes

Since the rivers Listica and Mokasnica have many bars and low water beds, the extraction of gravel and sand for construction of civil structure is convenient and low cost. It is estimated that more than 5 million cubic meters of gravel and sand have been extracted in 6 km long stream of Listica from Uzarica to Polog and approximately 3 km long stream of Mokasnica in Biograci. After the hydrologic reconnaissance of the mentioned devastated parts of rivers, the following impacts could bee stated.

6.1 Environmental impacts

6.1.1 Suspension of sediments

The mechanical disturbance created by the dredging process stirs up clouds of fine silt, and wastewater entering the river from gravel washing plant contains heavy concentration of fine particles. Sediments from these sources reduce light penetration, and settle, blanketing the riverbed. These impacts impair the respiration and photosynthesis of submerged aquatic plants, and reduce growth rates or cause death. The reduction in the extent of these vital habitats deprives river fauna of refuge from predators, and eliminates feeding and breeding areas. Fine settled particles make unsuitable spawning grounds for fish in Listica and Mokasnica, and thus sedimentation affect fish breeding.

6.1.2 Reduction in the natural capacity to take up sewage nutrients

Vegetated, shallow water areas in fat-flowing rivers have substantial capacity to assimilate nutrients in domestic and wastewater from other sources. Gravel and sand-mining leads to the removal of large areas of shallows, and the lower water velocities in the dredge hole dramatically reduced the efficiency of nutrient removal. Increased nutrient availability stimulates algal blooms and the growth of surface weed mats. Dense plant growth reduces the appeal of the water body and shades out life beneath (Mokasnica). Algal blooms may taint and color the water and release toxic substances, which are potentially hazardous to livestock and public health. These changes in water quality are detrimental to fish. The decline in aesthetic quality of the riverscape depresses owners of surrounding land, fishing activists, tourists and many others.

Organic matter may decompose in the sluggish waters of the borrow pit. It should be investigated because the accompanying decline in water quality may have an adverse effect of fish and the aquatic life and on the potable groundwater in water supply wells in Biograci, Jare, Polog, Dobric, Knespolje and Uzarici. These reasons justify a ban on extraction of gravel in zones where other activities must not be put in jeopardy. The critical zones should be determined by adequate program of investigation.

6.1.3 Erosion of the river bed and banks

The extraction of material from riverbed deepened the channels and enlarged the cross sectional areas of the rivers Listica (1 to 3,5 m) and Mokasnica (0,5 to 2 m). These resulted in more extracted quantity of material than the natural supply of sediments, lowering of the water level, and adverse erosion upstream and downstream for a distance from 1 to 5 km from the pits. Intensification of erosion on some places has destabilized the river banks. It may lead to collapse of some wells, pump-stations and even a bridge.

6.1.4 Clogging of the bed of the borrow pits

The slower rate of water flow in the borrow pits causes the deposition of fine sediments and organic matter held in suspension by the rivers Mokasnica and Listica. The sediments clog the bed of the borrow pit, so prevent filtration of river water into the aquifer. The cumulative impact of the 15 years of aggregate extractions has reduced groundwater replenishment and level for more than 2 m. Therefore groundwater became critical in the western part of Mostarsko Blato, where crop failure occurred, grazing pastures were lost, many water supply wells run dry as well as springs of smaller water flows.

6.1.5 Aesthetic quality

Extraction machinery and piles of sand reduced the aesthetic quality of the riverscape. Consideration should be given to the research of feasibility of planting desirable natural barriers to screen the extraction area.

6.1.6 Noise

Local people already reported that the increased mechanical noise at the extraction site discouraged migratory birds. Majority of water birds already moved to other more suitable areas. Research needs to determine the importance of Mostarsko Blato, Listica and Mokasnica rivers as a migratory birds stop-over.

Road haulage of gravel and sand involved higher noise and vibration levels. Transport of gravel, encouraged the passing of more private vehicles through these illegal roads and increased the risk of road accidents and of pouring out the fuels and lubricant in that fragile terrain.

7. Recommendation of the project monitoring and remediation needs

In order to remedy the consequences the mining at Mostarsko Blato, a field reconnaissance should be done, and complex analyzes of present environmental conditions at the devastated and surrounding unharmed areas. The documents related to pre-existing environmental conditions, should be studied. Approximately 10 bench-marks should be be installed on the best suitable sites in the project area.

In the first phase of this project the baseline data should be established through the systematic measuring at bench-marks and other places in the area of gravel extraction and its influence. The measurement of surface water level, groundwater level, level of sediments in the Mokasnica and the Listica, and changes in quality of water and biological productivity of soil at Project area will cover all seasons of the year. The baseline data enable comparisons of the existing conditions in the area of mining operations influence and pre-existing environmental conditions, Pre-existing data could be imagined from earlier documents and through observations at unharmed areas out of effect of mining operations.

A monitoring program will be initiated to continue throughout time after the project. The data collected should be analyzed and findings fed back into restoration works. The monitoring may include the following:

- changes in water quality, including nutrient status, salinity, turbidity and sedimentation patterns;
- changes in the hydrological regime, including fluctuations of flood, river flows and the level of the water table;

- changes in faunal diversity and abundance;
- alternation of landforms, e.g. river bed, stability of banks;
- interference with significant archeological site - Sarampovo.

The potential environmental hazards may be avaluated and then the adequate measures could be proposed to avoid further hazards. The collected data may be the support to take proper action to remedy the area.

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At the last minute …

Physical modeling in water management

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Abstract: The flow regulation and irrigation, the utilization of water for power, the construction of weirs and dams and the water management, has long tradition in Europe. The Laboratory of Water Management Research has a long time experience and the tradition on research in the branch water structures and water management. Some results of work are shown in this paper.

Keywords: physical and mathematical modeling, research

1. Introduction

Physical modeling is the specific branch of the scientific research allowing the observation of complex phenomena and processes taking place at the real constructions on their, usually down scaled, models, including interdependences. The Laboratory of Water Management Research at the Department of Water Structures of the Faculty of Civil Engineering, Brno University of Technology has long time experience and continues the tradition in this branch of the research. Selected examples of the research carried out in the last years are described. Most of them solve concrete requirements asked by practice. For instance, the research that determines capacity and designs steps to send water stream from the radial gate during observing of air flow in individual variants of designed bottom outlet of the right by-pass tunnel of the hydraulic structure Les Království at the Elbe River, or the research dealing with the sediment transport in streams documented by the physical model of the weir at the Morávka River in Vyšší Lhoty. Many of works carried out within the research became the methodic rules of the given problematic. For example, the design of the object called „the pear", initiated by the need of kinetic energy absorption of the stream by nature pleasant means, solved within the research of the B/h characteristic of the riverbed getting wide and narrow. The structure, at present built on the Emme River, Switzerland, was solved for the concrete locality of the Morávka River. The importance of the physical modeling is not only based on the solution of concrete requests. At present, the rapid progress of electronic measurement instruments and computing technique causes the application of the physical modeling in the mathematical models (Fluent 6, HEC-RAS, MIKE Zero-11 and 21, ANSYS 8, HYDRO 11, and demo version of Aquadyne, SMS 8.1, CCHE2D 2.0) calibration becomes integral part of tasks connecting with the physical modeling.

2. History and function

The Laboratory of Water Management Research has more than 85-year tradition and it represents the oldest laboratory in the Austria-Hungary Monarchy and the third oldest experimental laboratory in Europe. The founder of the laboratory Professor Antonín Smrček Dr.hon.causa (10.12.1859-2.1.1951) professor of Civil engineering used it for student practical education and scientific work. The laboratory was constructed between 1914 and 1916. During the World war I. the Czech Technical University including laboratory served as a field hospital. Thereupon the beginning of the first experimental work has started again in 1917. The research activity has followed the practical needs. The earth dikes and their destruction were studied in context of disaster on Bílá Desná, where the earth dike broke on 8.9.1916. There are theoretical studies known to date concerning the influence of laminar and turbulent flow as well as surface tension on weir discharge measuring. Similarly, questions of model similarity of physical weir models were also experimentally solved at the laboratory. The results of these experiments are well known not only in Europe, but in the USA and Brazil too.

Fig.1 The founder of the laboratory – Prof. Smrček and view to the laboratory galleries

Recent scientific research projects are oriented on:

- Flow in a reservoir (temperature stratification and water pollution).
- Monitoring of earth dike and the process of their breakage and destruction using new method and electronic devices.
- Research of riverbed forming processes and stability of watercourses.
- Research of intake and outlet structure flowing e.g. in small hydraulic power plants.
- Research of suspended matter and sediments load of rivers.
- Study of effect of the stream to floating ship and measurements of the acting pressures.

- Measuring of turbulent flow, point velocity of flow and experimental calibration of current meter.
- Authorized activities at discharge measurements.

3. Research activity

The Laboratory of Water Management Research disposes of modern measurement techniques (**L**aser **D**oppler **A**nemometry, **P**article **I**mage **V**elocimetry, **U**ltrasonic **V**elocity **P**rofiling, **THERM**al scalar field, **E**lectrical **I**mpedance **S**pectrometry and so on). With the expansion of computer techniques and digital data transfer and treatment physical and mathematical modeling can join together. The main activity domain is in young people education for all time. Some results of research activity are shown in the following parts of this capture.

3.1 Flow in a reservoirs (temperature stratification and water pollution)

The method of the electrical impedance spectrometry – EIS [1, 2] has been used for the measurements of the water pollution transport (Fig.2). The two-terminal method of impedance measurement has been used. The stainless steel electrode system consisting of several pairs of electrodes has been build into the river. This configuration is simple and enables good manipulation with electrodes and the instrumentation.

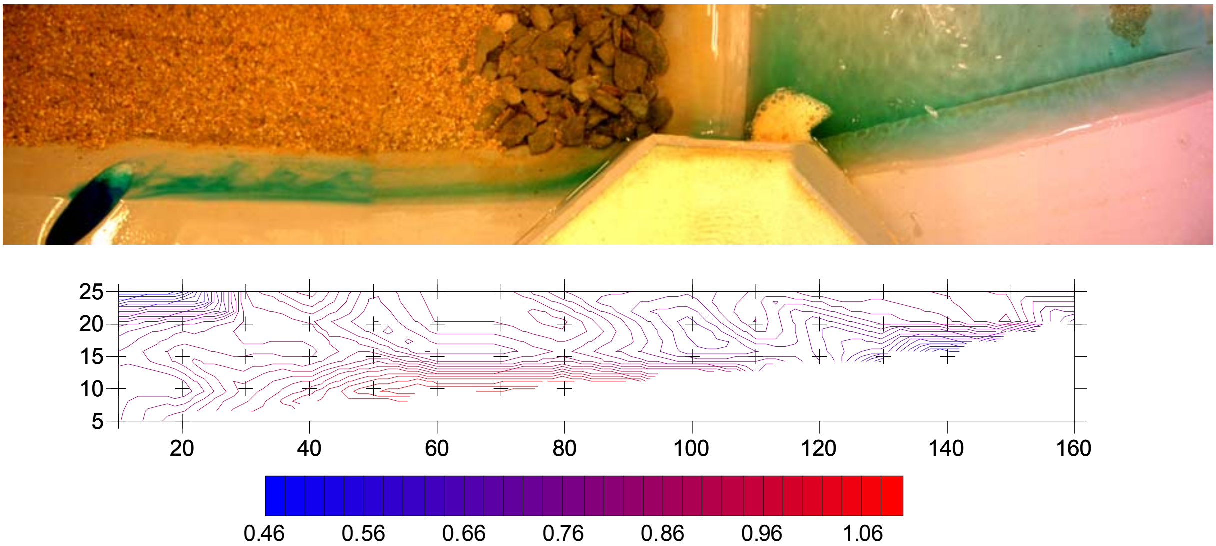


Fig.2 Changes of electrical conductivity (from *0.45 mS* to *1.05 mS*) in the layer of *0.01 m* under the water level; area of interest was *1.60 m* x *0.25 m*, the cross marks denote the measurement places

3.2 Monitoring of earth dike and the process of their breakage

Two electrical methods [3] of dikes monitoring - temperature scalar field and electrical impedance spectrometry were used. Using these methods, the non-stationary movement of the free water level in the dike can be indicated. The methods also enable to detect the piping in the dike due to the activity of animals, the destruction of the dike by overtopping and surface erosion on both slopes of the dike and also of the stream bank and the riverbed. Some experiments are shown in Fig.3.

Fig.3 Physical Model of the Dike

3.3 River training and river revitalization

In the long term development of riverbed is systematically monitored for selected streams by samples collection of bed load sediments. Their summary processing serves as basic data for management, maintenance and designing of streams. Physical modeling is useful in the case of problematic and mathematically difficult to model sections of watercourses, structures etc.

For example, using the physical model of the section of the Morávka River to scale *1:50*, the use of the pear-shaped structure for the dissipation of energy was tested (according to Swiss sources, used on the Emme River, etc.). In order to avoid the laborious testing of many variants and to avoid performing too many time-consuming measurements, the measurement is combined with mathematical modeling where the physical model serves for the calibration and the subsequent verification of the mathematical model.

Weir - the construction of barriers across rivers has had negative effects on natural fish populations contributing to the diminished abundance, disappearance and even extinction of species. The Hrabovský weir is situated in the territory that had been experiencing extensive mining operations. Therefore the land has declined and the riverbed erosion increases. During its existence the weir has been reconstructed several times. The experimental model (to scale *1:50*) research of some variants of the weir-reconstruction was realized. The structure consists of two-weir construction joined with the rock-ramp fishway. Two different variants of fishway have been built there. In the first case, the fishway was situated in the middle of the stream and in the second one it was along the left riverbank.

Mathematical modeling is, in comparison with physical modeling, a method which is much more acceptable from the viewpoint of obtaining information about the watercourse as regards time and funds. But the problem of fixing the coefficients that strongly affects the flow is subject to human factors.

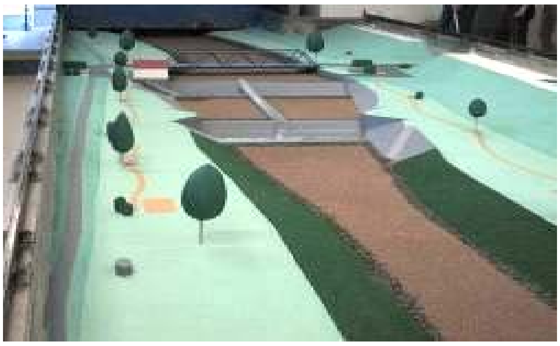


Fig.4 The “piriform object” on Morávka River   The Hrabovsky weir on Ostravice River with fishway (M_l = 50)

Ill-considered submission of coefficients often leads to very different results, which causes financial and sometimes even loss of human life.

The assessment of correct coefficients is made in the modeling stage we call model calibration. This stated part must not be underestimated as regards both quality and time, as it happens that the time of successive approximation to the correct coefficient values may amount to as much as half of the whole process of mathematical model construction and use. However, the calibration and the subsequent model verification require data from real watercourses under all flow rates in sufficient quality and quantity. This requirement often poses the greatest problem, and, for this reason, data from physical models are used in some cases.

The water flow in the channel causes the movement of particles from the environment of which it is composed and on which the water flows. Thus the modeling of the sediment and suspended solids with the bottom deformation has become ever more important in the forecasts of the future watercourse channel development. In some cases, as is the case in the “piriform object”, it even plays a pivotal role. (Fig.5).

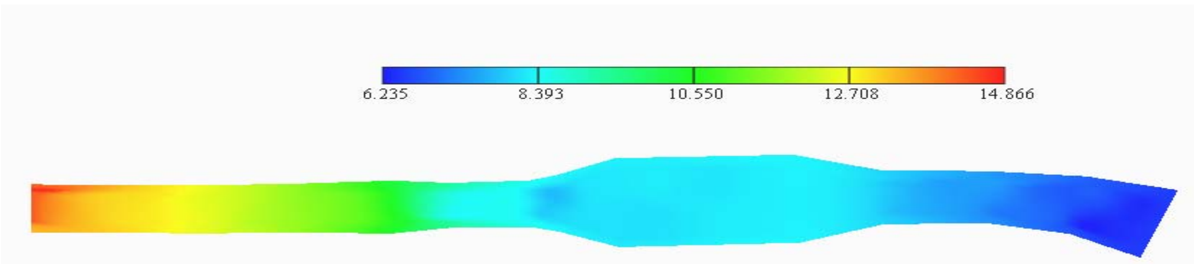


Fig.5 Water level in the watercourse section with the piriform object (in meters) – a 2D model, software CCHE2D

3.4 Research of flow structure of water power plants

Research of flow structure of water power plants is the specific area of activities of Hydraulic Laboratory. The research determines capacity and designs steps to send water stream from the radial gate during observing of air flow in individual variants of designed bottom outlet of the right by-pass tunnel of the hydraulic structure Les Království at the Elbe River (it was built in 1919) or of the hydraulic structure Souš at the Černá Desná River (it was built in 1915).

Hydraulic Structure Souš, Černá Desná River M_l = 12,5

Hydraulic Structure Znojmo, Dyje River M_l = 35

Hydraulic Structure Les Království, Labe River M_l = 20

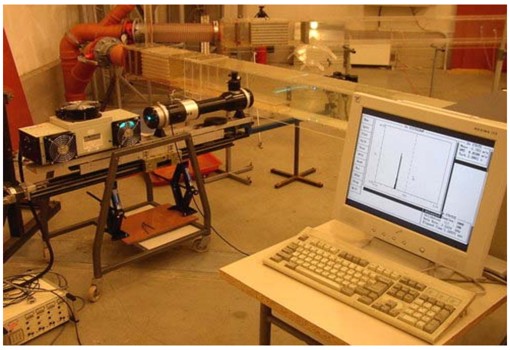
Little water power plant Libčice, Vltava River M_l = 25

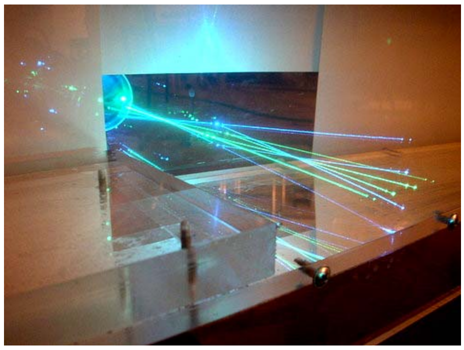
Little water power plant Gries, Salzach River M_l = 20

Fig.6 Research of flow structure of water power plants

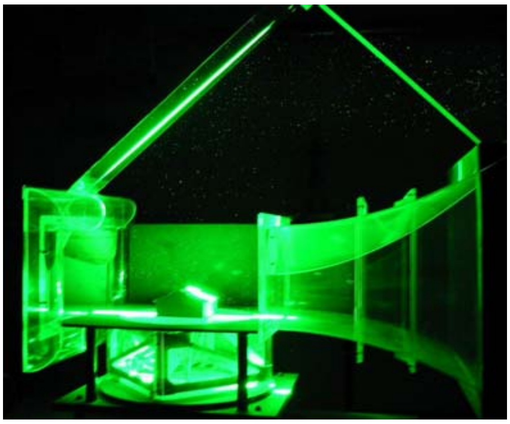
3.5 Measuring of turbulent flow and point velocity of flow

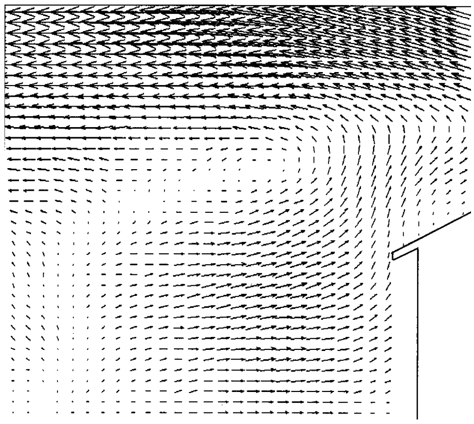
The measurement of flow parameters in defined profiles of square constant cross section and it's linking upstream and downstream straight axis sections offers important information for others applications. For the measurement of mean and pulsation velocity components the contact-free measuring technique was used, such as laser Doppler anemometer (LDA) and integral laser anemometer (PIV). Complementary the values of pressure in selected piezometric points on the canal walls were measured. The measurement was followed out on hydraulic or aerodynamic models. The results of this detailed measurement in selected relatively simple element are used for subsequent calibration and verification of mathematical models with appropriate type of turbulence.





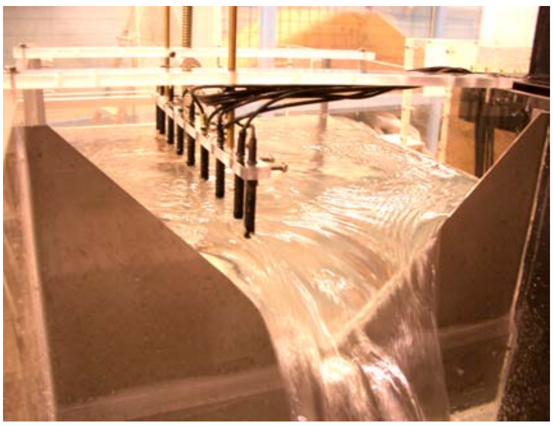
LDA System for turbulent flow measuring (Hydrodynamic tunnel)

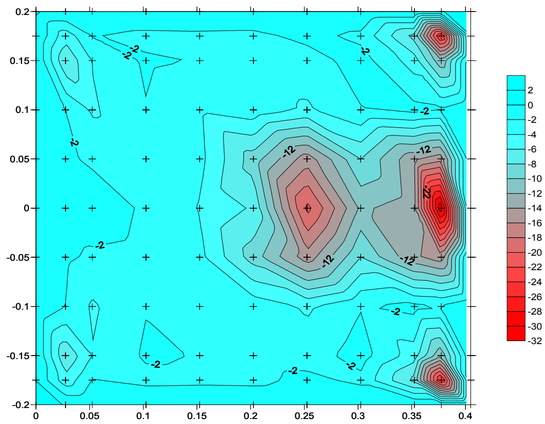




PIV System for turbulent flow measuring (Aerodynamic tunnel)

For point velocity of flow measuring is also used UVP Monitor with *4 MHz* probe. UVP Monitor disposes of integrated switcher (20 places of measurement can be switched) and enables to use also *2 MHz* and *8 MHz* probes (different velocity range). The software Met-Flow UVP version 3.0 enables experiment proceedings and data processing. For other application the date format *.dat, *.txt, *.csv is possible used.





UVP Monitor for flow measuring near spillway crest (4 MHz probes, vertical velocity field)

Fig.7 LDA, PIV, UVP measuring systems

3.6 Activities at others measurements

Fig.8 Authorized activities at discharge measurements

Fig.9 New test line of abrasion measurement in sewer and one of the results

Summary

Results of some laboratory works are shown in this paper. The results gained by model experiments show the possibility of the application in practices, some physical or mathematical models were realized as answers the theoretical or practices questions, some given important theoretical conclusions. Laboratory of Water Management Research is one of most interesting workplaces of its kind.

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Two Non-invasive Methods Application in the Dike Monitoring

Jana Pařílková, PhD.

Abstract: Two original electrical methods of dikes monitoring are described in detail. Using these methods, the non-stationary movement of the free water level in the dike can be indicated. The methods also enable to detect the piping in the dike due to the activity of animals. Some results are shown and discussed.

Keywords: dike, thermistor, electrical impedance, monitoring

1. Introduction

Frequent floods initiated intensive research oriented to the solution of stability and reliability of dikes. The knowledge of the physical properties of the soil materials of the dike and the mechanism of its destruction by infiltrating water help to solve the problem of the protection of the environment. For this purpose the physical and mathematical modeling is applied. The soil medium involves all three phases – solid, liquid and gas. The processes occurring in it are very difficult to be described mathematically. The non-stationary free surface flow of water in un-saturated soil media is a complicated problem. Therefore, the method of the monitoring on physical models of dikes is the one of the effective methods enabling to solve the problem. In the article the method of the physical modeling is described and the results are presented and compared with the numerical solution.

The possibility of the non-traditional use of two non-invasive methods for the soil status and its change monitoring during underground water flow is discussed. The methods are the measurement of the temperature scalar field and its progress in time in the observed water structure space (the thermistor probes are used for the temperature to electrical resistance conversion), and the electrical impedance spectrometry (EIS), which monitors electrical impedance and its changes caused by the load of the construction by water. How it was said, the complexity of the monitored phenomenon is in non-stationary water flow through indefinite soil saturation. In view to these facts, the applied measurement systems were tested, especially from the point of view of their time constants and parasitic influences elimination (transient resistance, cables length, probe and electrode construction – e.g. their shape and durability under rough operational conditions – they are localized in wet soil). Methods, monitoring apparatus, and handling and evaluating software packages are built and tested within the grant projects GACR, 103/01/0057 and 103/04/0741. The experiments have been carried out in laboratory conditions in the Laboratory of Water Management at the Institute of Water Structures of the Faculty of the Civil Engineering of Brno University of Technology. The experimental results, carried on physical models of protective dikes and regular bodies, were used for calibration of mathematical models. At present, owing to the optimistic conclusions, the new EIS apparatus operating in real environment conditions is built.

2. Physical modeling

2.1 Physical Model

The models were built of unscreened sand from Bratčice with the hydraulic conductivity $k = 1.78 \cdot 10^{-4}$ *m/s* and with the effective grain size $d_{ef} = 1.57$ *mm*. The material inequality grading number $U = 7.42$, the initial relative humidity of the material (prior to the first loading) varied between (*0.78* and *0.89*). The temperature of the material varied between (*13.3* and *21.5*)°C, the laboratory air temperature between (*13.2* and *21.9*)°C, the relative air humidity in the laboratory between (*0.38* and *0.58*) and the water temperature in the storage tank varied between (*12.7* and *18.7*)°C. The material was compacted by a plate vibrator.

2.1.1 Geometrical dimensions of physical models

Geometrical dimensions of physical models of the dike:

Dike height	*0.8 m*
Dike width at the crest	*0.4 m*
Dike length in the center of the crest	*1.0 m*
Slope of up-and down-stream faces	*1 : 2*
Dike width at the foundation with the given slope 1 : 2	*3.6 m*.

The dikes (Fig.2a) were built in the flume – *6.0 x 1.0 x 1.5 m* with the steel bottom simulating impermeable foundation. The flume structure was made of steel plate *0,004 m* thick, with piezometers installed; the opposite wall was of organic glass *0.025 m* thick. A transparent film from the inside and a raster field *0.1x0.1 m* from the outside protect the glass wall against wear. The inflow section was equally of steel plate *0.004 m* thick, *1x1x2 m,* with four handling gates allowing control of the level of the upstream face of the physical model of the earth fill dike. Induction flow indicator measures the discharge. The measuring channel itself ends in a sand trap made of steel plate *0.004 m* thick, *3x1x1 m*. The end of the measuring channel was provided with a collecting main to measure the discharge of infiltrated water. Water was pumped into the system from a *35* m^3 storage tank.

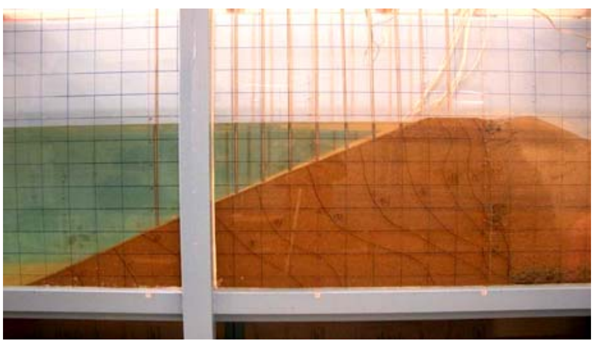
In the same flume was built also the body of cube-form with proportion *1x1 m*. The material – sand from Bratčice was not compacted by a plate vibrator. The model and movement of water level on the upstream face is shown in Fig. 1.

Fig.1 The model of cube-form in the flume

Because obtained results were very interesting from view of calibration of the measuring devices, was built other model of the cube-form. The tank of cube-form structure *0.725x0.495x0.610 m* was made of organic glass *0.025 m* thick. The inflow section was equally of organic glass, *0.160x0.495x0.610 m,* with piezometer installed. The proportion of the measuring tank is *0.495x0.495x0.610 m.* The model itself ends in a sand trap made of same material, *0.017x0.495x0.610 m*. The wall between inflow section and measuring tank was perforated; diameter of the holes was *0.006 m* and their distance *0.020 m*. The material – sand from Bratčice in measuring tank was compacted by a plate vibrator. The vertical distance of compaction was *0.100 m*. This model of the cube-form is shown in Fig.3.

2.2 The movement of the water level on the upstream face

During the experiments, the models were loaded by water with constant level *0.789 m* above the channel bottom. The process of the water infiltration is illustrated in Fig. 2a. In Fig.2b the movement of the water level on the upstream face is shown. Measurements of the free water level of the infiltrating water were stopped after *1 hour*.



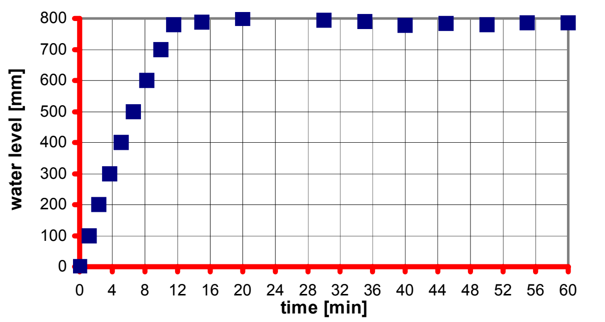


Fig.2a) The process of infiltration in the dike b) Movement of the water level on the upstream face of the dike

The typical movement of water level on the upstream face of cube-form models is shown in Fig. 4.

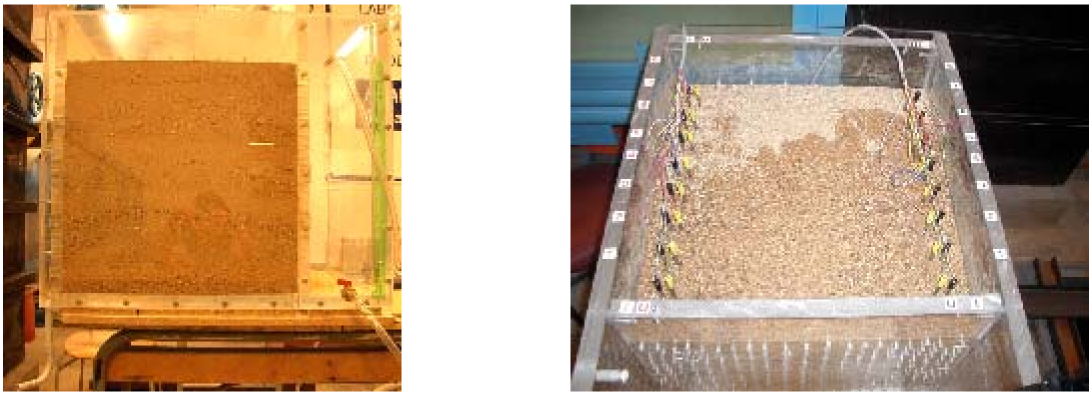


Fig.3 The model of cube-form

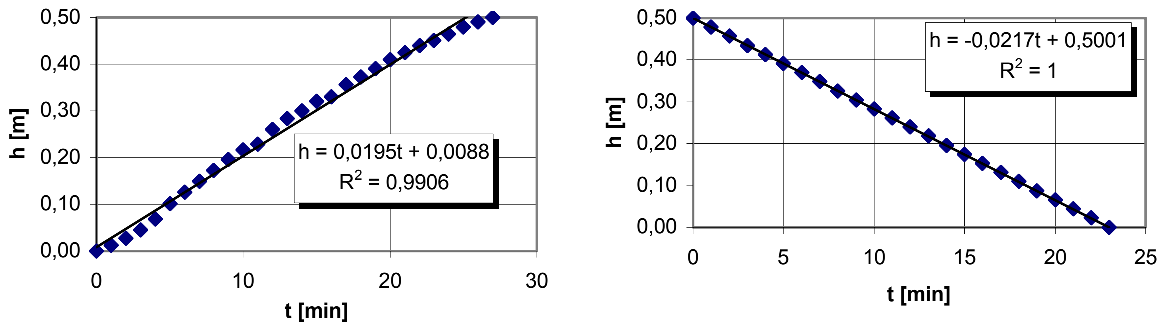


Fig.4 The movement of water level on the upstream face of cube-form body

2.2 Realized experiments

Experiments carried out on physical models are divided into four groups:

- measurements of the free water level movement in the dike;
- measurements of the breaching process during the overtopping;
- monitoring of the structural changes due to internal erosion (piping), activity of animals and surface erosion on both slopes of the dike and also of the stream bank and the river bed;
- monitoring of the water pollution.

2.3 Factors affecting resistivity of soil media

To describe water - the soil media interaction is complicated. The soil medium involves all three phases (solid, liquid and gas) and therefore the non–stationary movement of free water level in that soil medium is not easy to detect. The weather is a big problem, too. That is why the soil medium can be saturated in one part and together un-saturated in other part.

The electrical resistivity of the soil media depends on the following factors:

- Porosity and the pore structure of the soil media;
- Amount of water (degree of saturation);
- Salinity of the water;
- Temperature;
- Pressure;
- Water – soil medium interaction and alteration;
- Steam content in the water;
- Steam content in the soil media;
- Season (influence of the weather).

3. Methods of measurement and their results

Two electronic methods have been proposed and applied (except visual and piezometric methods) for monitoring the non-stationary flow and the destruction of the dike by overtopping on physical models.

3.1 Temperature scalar field

The first method – is based on the monitoring of time dependence of the temperature scalar field. The thermistor sensors have been used for the conversion of the temperature *T [K]* to electrical resistance *R [Ω]* by the Wilson relationship:

$$R = Ae^{B/T}, \qquad 3.1$$

where *A* is a constant describing the material and the shape of the quasi-conductor and *B* is a constant describing the material properties of the thermistor probe.

The thermistors are supplied with constant current. The temperature properties of the dike material are computed from the changes of the electrical voltage drops during the infiltration. Thermistor sensors [2, 3] have been placed in prescribed positions to create a space grid in the dike. The aim of the measurements is to determine the point of the thermal jump caused by the contact of infiltrating water with the sensor. The prescribed sensor position and the prescribed discrete time enable to observe the process of infiltration in the space of the sensor grid and to determine the progress of the filtration in time and space. The applied thermistor sensors, with diameter smaller than *2 mm*, comparable with the size of effective grain of the material, did not affect the conductivity properties of the grain material. 128 thermistor sensors (full points) were placed in the grid as shown in Fig.5. The unique data logger supplies the sensors with constant current *10 μA*. The developed and applied device TERM provides switching of channels, and measures voltage drops in sensors. Measured voltage drops are digitized in a 12-bit ADC and processed in the embedded digital signal processor, which controls the data logger function and sends data to the PC.

The software TERM 1.5 enables to realize computer animation too. The software records the history of the process of infiltration with sampling period of *500 ms* in all 128 channels. The overall time of the measurement is limited only by the PC memory capacity. The comparison between results obtained by experiments (solid line represents at the same time the voltage drops ΔV between the thermistor sensors placed in the model of the dike and the corresponding free water level) and numerical modeling (broken line) is shown in Fig.5 too. The numerical model is described in [3].

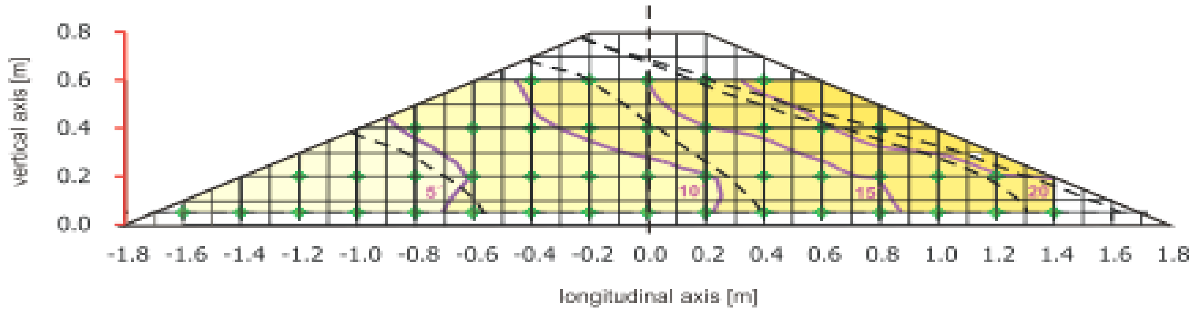


Fig.5 Results obtained by experimental measurements (solid line) and numerical solution (broken line)

3.2 Electrical impedance spectrometry

The method of the electrical impedance spectrometry [2, 3] has been used for the measurements of the prescribed physical properties (the changes of the electrical impedance caused by the water level and the structural changes of the soil medium, or by the pollution). This method (EIS) takes advantage of the impedance measurement in a complex form in different parts of the dike (on the surface and in the dike).

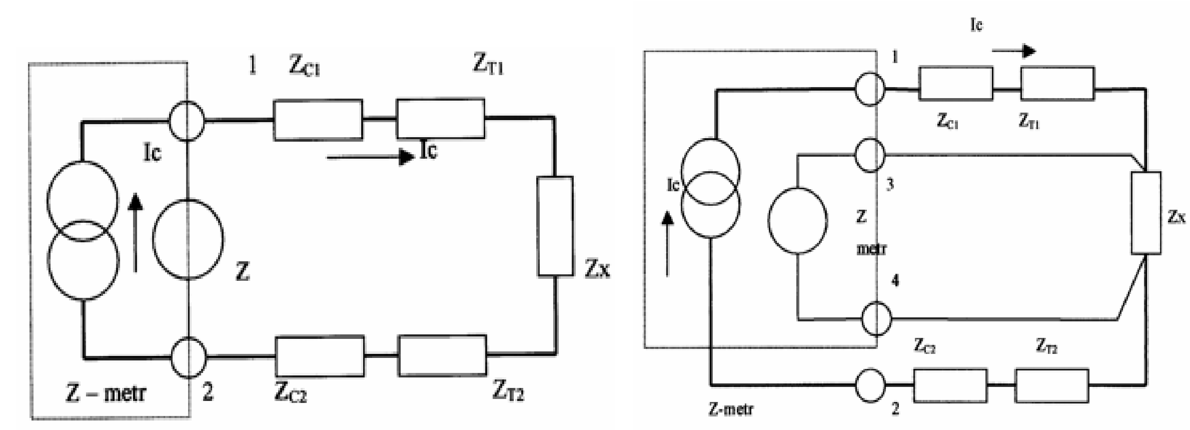
The electrical impedance Z (or admittance Y) of the soil between electrodes can be calculated in Carthesian (R and X) coordinates. These parameters are related as follows:

$$Z = R + j\,X \qquad 3.2$$

The complex impedance Z of the un-saturated porous material describes its properties:

- The solid part (grains) is formed by insulating materials characterised by their dielectric constants and represents the imaginary part of the measured impedance.
- Water containing mineral salts is a conductive material. The degree of saturation of the material strongly influences the real part of the measured impedance.

Therefore, the measuring equipment must enable to determine both parts of the impedance Z. The device includes current supply I_C, the electrical impedance Z meter and external electronic switcher. It enables the measurement in one frequency or in the prescribed frequency range. In both cases two or four terminal electrode system may be used for measurement. The system with four terminal electrodes eliminates the influence of the lead-in cable resistance Z_{C1}, Z_{C2} and the transition resistance between electrode and soil medium Z_{T1}, Z_{T2}. Differences in these systems are shown in Fig.6a,b.



a) Two terminal electrode system; b) Four terminal electrode system

Fig.6 Electrode system connection

In Fig. 6a) two terminal electrode system is shown. Unknown electrical impedance Z_x is measured between the point 1 and 2 and is calculated from the relationship $Z = Z_x + Z_{C1} + Z_{C2} + Z_{T1} + Z_{T2.}$ In Fig.6b) four terminal electrode system is shown. The points 1and 2 represent current electrodes and the points 3 and 4 potential electrodes. Unknown electrical impedance Z_x is measured between the point 3 and 4 - $Z = Z_x$.

The stainless steel electrode system consisting of several pairs of electrodes has been build into the dike. The stainless steel rod electrodes have the diameter of *0.015 m* (*0,004 m* in cube-form body) and the length of *1.5 m* (*0,45 m* in the second one). The arrangement of the electrode pairs (the two terminal method of impedance measurement) in the dike is shown in Fig.2a, 7 (four terminal method in Fig.3). Electrodes create the electrode pair, which is supplied with the a.c. signal of prescribed amplitude and frequency generated in the programmable digital synthesizer. This configuration is simple and enables good manipulation with electrodes and the instrumentation. Nevertheless, this simplicity contributes to the transient resistance error and, therefore, its use is restricted to cases of monitoring the water infiltration process or to the water pollution transport.

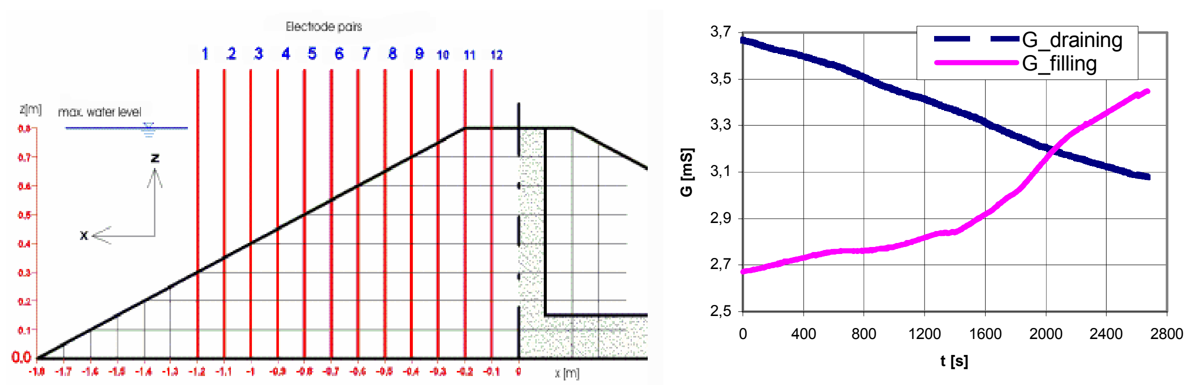


Fig.7 Installation of pair electrodes in the dike, continuous measurement - one electrode pair (f=8kHz, U=400 mV)

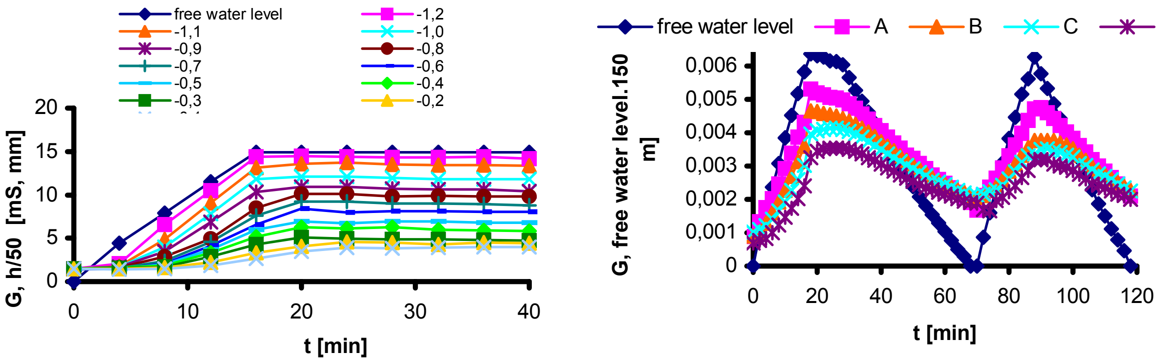


Fig.8 Impedance reaction on the rush water level measured by two terminals in the dike and in the cube-form

Summary

The experiments proved the applicability of both methods for identification the dynamic effects of the infiltrating water and the volume changes in the dike. Comparison with the frequently used visual method proved the capability of discovering material inhomogeneities, together with the identification of the state of the dike. The results gained by model experiments show the possibility of the application in practices. Simultaneously they indicate the necessity of further improvements of their sensitivity and information value. As to the thermistor application, their use in a form of anemometer is planned. The use of the anemometric method promises improvement of the sensitivity in cases, when the material of the dike and the infiltrating water has the same temperature. Further development of special divided electrode system would enable to observe surface changes of the dike, as well as structural deformation. Four terminal electrode system can detect the surface changes as well as the changes inside the monitored soil medium. But it does not permit to localize vertical position of these changes (the system measures the total profiles between electrodes). Therefore four terminal electrode with different construction were developed and tested. In this system narrow electrical field of current electrodes in prescribed vertical position is desirables. So the second potential electrode in the same place enables to monitor the changes of the soil medium. Measurement is realized as relative.

The research goes on, and is carried out within the grant projects No 103/01/0057 and 103/04/0741 (Grant Agency of Czech Republic).

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