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FACULTY OF CIVIL ENGINEERING

GDAŃSK UNIVERSITY OF TECHNOLOGY
FACULTY OF THE HYDRO AND ENVIRONMENTAL ENGINEERING

UNIVERSITY OF ZAGREB
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WATER MANAGEMENT
AND
HYDRAULIC ENGINEERING

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Preface

The history of the International Symposium "Water Management and Hydraulic Engineering" has been initiated and based on a bilateral co-operation between the Faculties of Technical Universities in Gdansk and Zagreb since 1976 and organised in three year period. Since 1998 when this Symposium was held in Dubrovnik, Slovak participants from the Slovak University of Technology in Bratislava and from the Institute of Hydrology, Slovak Academy of Sciences started to attend this Symposium.
During the last Symposium held in Miedzybrodzie Zywieckie was decided to organise the symposia in other countries, as well. Faculty of Civil Engineering of the Slovak University of Technology was appointed to organise the VIII. International Symposium on "Water Management and Hydraulic Engineering" in the year 2003 in Slovakia. The scientific and organising committees have decided to realise this event in mountainous region of High Tatras from October 5th – 9th, 2003 .
Topics of the symposium have been chosen to cover main aspects of hydrology, hydraulics, geotechnics, sanitary engineering, environmental protection in all branches of the water management practice. The main goal which is foreseen to be achieved is to present current research results at different research working places in Poland, Croatia, Slovakia, Czech Republic, Macedonia and Austria. The evidence of interest from mentioned countries is that this proceedings consists of more than 50 scientific contributions. In the proceedings they are disposed in alphabetical order and during the Symposium they will be divided to scientific session on:

- **Hydraulics and Hydro-engineering**
- **Water Management and Hydrology**
- **Sanitary Engineering**
- **Geotechnical Engineering**
- **Environmental and Flood Protection.**

The Organising Committee has prepared for the participants a number of attractions in adjacent region of the Permon Hotel, i.e. excursion to Čierny Váh pumping storage hydro-power plant and Liptovská Mara water reservoir as well as the excursion to the Demänová cave, Museum of Liptov Village and a beautiful rafting on wooden rafts on the Dunajec river.

Faculty of Civil Engineering as well as Slovak University of Technology celebrate in this year the 65th Anniversary of their existence. It is a honour and pleasure, as well, to welcome all participants of the International Symposium "Water Management and Hydraulic Engineering" for the first time in Slovak Republic, especially in picturesque region of High Tatras. I wish all of them interesting presentation, fruitful discussion and exchange of ideas and experiences about new research results and their realisation in water management practice.

Assoc. Prof. Andrej Šoltész, PhD.
Vice-Dean of the Faculty of Civil Engineering
responsible for Water Management Study

High Tatras, Slovakia, October 2003

The Ground Water Level Control in the Area of the Influence of Žilina Water Structure

Dana Baroková, Pavol Frankovský*, Andrej Šoltész

Slovak University of Technology, Department of Hydraulic Engineering, Radlinského 11, 813 68 Bratislava, Slovakia, barokova@svf.stuba.sk , soltesza@svf.stuba.sk,
*Hydroconsult Bratislava, Exnárova 59, 826 13 Bratislava hts@hydroconsult.sk

Abstract

Žilina Water structure is the first construction in Slovakia where the environmental impact assessment (EIA) of this structure was reviewed comprehensively. On the right side of the water reservoir a small village Mojš was after constructing a sealing wall along the levees attacked by ground water surplus, especially in extreme hydrologic situations. Goal of the presented paper is, on base of numerical modeling of ground water flow, to show a possibility of technical measures for decreasing the unfavorable ground water table in the village. The finite element method model for ground water flow was calibrated for two hydrologic situations and afterwards appropriate technical measures were introduced into simulation. Results of this study are very actual and were completely accepted by the project and investment managers.

1 Introduction

The hydraulic structure Žilina is Slovakia's first structure whose influence on the environment has been assessed comprehensively by means of EIA method (Environmental Impact Assessment). The passing of the amendment by the NC SR 127/1994 on the assessment of the environmental issues has been confirmed by the hydraulic structure Žilina. The proposals to abate the influence of the structure on the environment have been included into object composition of the structure (1994 – 1998) and the co-operation with the environment experts was also maintained at the realisation and operation of the structure. A part of building process of Žilina WS was setting up the village of Nová Mojšová Lúčka and a part of the village of Rosinky on the left bank of the Váh river.
On the right bank of Váh river, in the very surroundings of the newly- built WS reservoir there was a village of Mojš that remained untouched. One of the measures taken when building the hydraulic structure was setting up a sealing wall in order to prevent the reservoir water from flowing into the aquifer. Although the wall fulfilled its purpose, at the same time it eliminated the draining effect of the Váh river. This accounted for an increase of the ground water level in Mojš (in extreme hydrological situations – precipitation, thawing of snow), accompanied by flooding the cellars in Mojš.
The objective of the research was to prove the causes of damping the cellars in Mojš and, based on mathematical modelling of the ground water flow, to propose a certain technical measures eliminating the negative situation in the real conditions of the area.
The presented study evaluates the hydrogeological conditions of the area, which were the basis for constructing a mathematical model of the ground water flow using the finite element method. This model was calibrated for dry seasons (November 2001, May 2002) as well as extreme seasons of precipitation (July 2001) and used for the groundwater level forecast by taking particular technical measures. The missing data, such as the discharge and water level

regime in the bio-corridor were measured directly in situ by the researchers. (June, July 2002).

2 Description of the investigated area

The investigated area is located in the alluvium on the right side of the Váh river , in the very surroundings of the waterworks. It is marked in the east by its tributary–Varínka and in the north by the foot of the Kysuce mountain line. This area is marked by another peculiarity, there is an alternate corridor transporting water and serving as a fish ladder alongside the dam on the right side of the hydraulic structure (filled from the reservoir at the confluence of Váh and Varínka rivers. The alternate bio-corridor due to its level and discharge regime actively affects the water level regime of the ground water in the given area. (Kadlec - Kvál - Bukvová, 2001). Due to the construction of the Žilina WS, a complex geological and hydrogeological research was carried out in this area. The research was done at various times and by various organisations. We focused on the geological boreholes drilled, assessed and monitored on the right side of the Váh river with detailed focus on the village of Mojš.

3 Geological and hydrogeological conditions

For the solution of the ground water level mode it was necessary to possess sufficient knowledge about the geological status of the examined area. The assessment of the geological status suggests that the cover layers of soil in the whole area are made up by fine-grained soil of alluvial facies, whose thickness ranges from 0,5 – 2,0 m in the flat areas (Mojš), and 2,0 – 4,5 m in the sub-mountain areas (Gbeľany) with alternating sandy clay, clay soil, and sand containing a mixture of fine soil.
Žilina basin is characterised by the absence of neogenic sediments. The quaternary sediments are mostly represented by fluvial sediments. These include predominantly gravel, creating in the sub-base of fine–grained cover layers a continuous, but unequally thick layer. The sediments are characterised as gravel with admixture of fine–grained soil, or unequally gritty gravel. The crucial importance in the assessment of Žilina WS is held by fluvial sediments. They form the base soil in all objects and to a great degree they form the groundwater. The unequal potency mentioned above grows from 6,5m up to 18 m from the Váh river northward to the sub-mountain areas (Teplička, Gbeľany). Another outstanding feature we devoted our attention to, was determining the permeability of the aquifer layers. Here again, we based our work on geological research carried out in this area in the past years. Fig.1 shows the samplers and bore-holes, where the hydraulic conductivities were determined. Stating the hydraulic conductivity in over 50 samplers was done at various times, using various methods and having different calculated value. In accordance with this data we assembled a map of hydraulic conductivity isolines in $m.s^{-1}$ with a detailed focus on the village of Mojš.
We used the data from 53 bore-holes as well as literary resources (Šofko, 1992, Šalaga, 1995). The latter suggest that the aquifers are represented by gravel-sand levees of alluvial plain and stream terraces. These determine the ground water level, which is, in case of settled gravel, slightly pressured. Groundwater level creates coherent horizon with the depth of approx. 2,5–16,5 m beneath the terrain. The hydraulic conductivity oscillates (with certain exceptions) between $1.10^{-3} – 1.10^{-4}$ $m.s^{-1}$. The lowest figures were found in the mountainside (Teplička, Varín). According to these figures it is possible to assess the aquifer of the right side alluvium of Váh river as a porous medium with high permeability, suitable as a source for water supply (Štofko, 1992).
Another factor influencing the groundwater flow is the sealing wall built south of the water source Teplička. In the eastern part of the given area this wall partially splits the stream of groundwater which infiltrates from the northern mountains and Varínka stream. One portion

of the groundwater flows down over the wall toward the Teplička water source and another portion flows towards the village of Mojš. While the Váh river showed the drainage function this flow did not necessarily have to cause difficulties in form of high groundwater level in the village. In case of building the sealing wall in the dam on the right side of the water reservoir this becomes a substantial problem, requiring an immediate treatment.

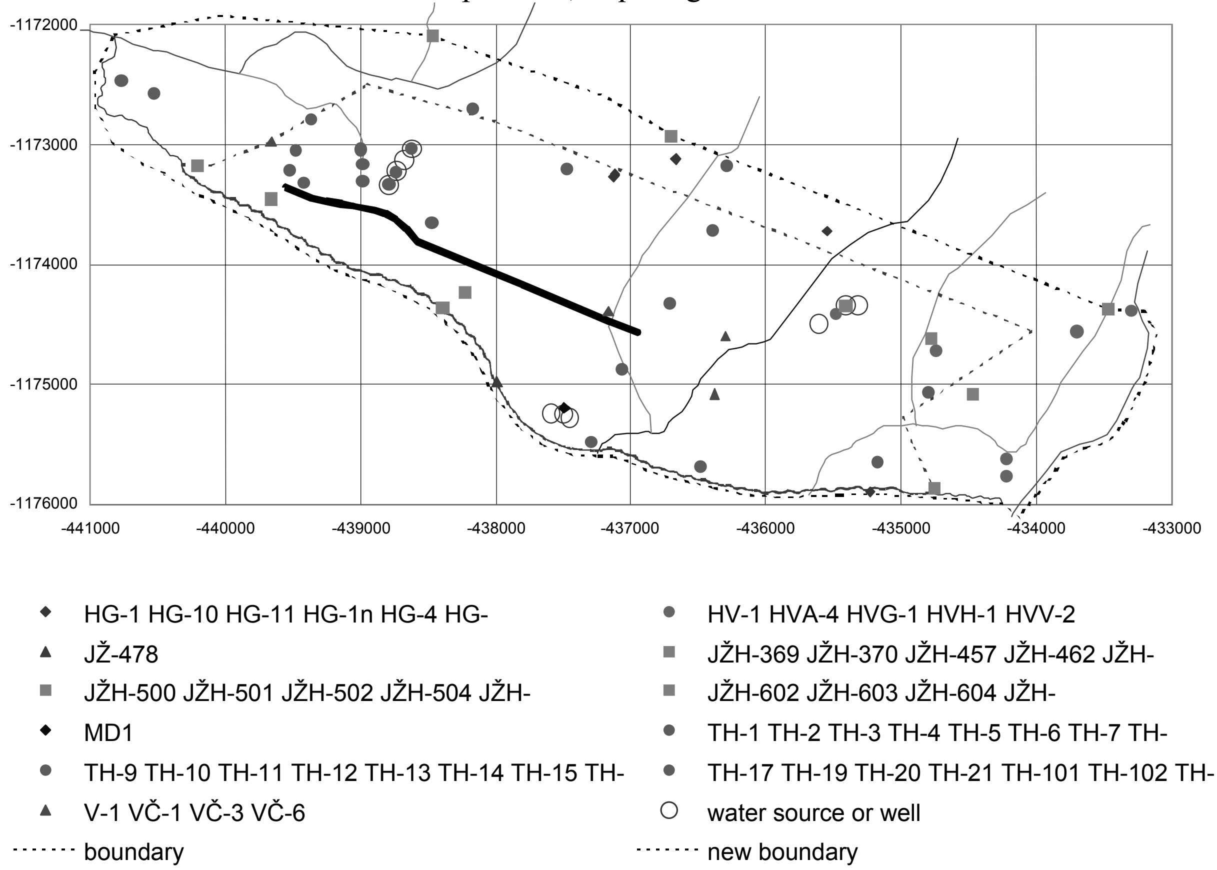


Fig. 1 Map of observation bore-holes where the hydraulic conductivity was determined.

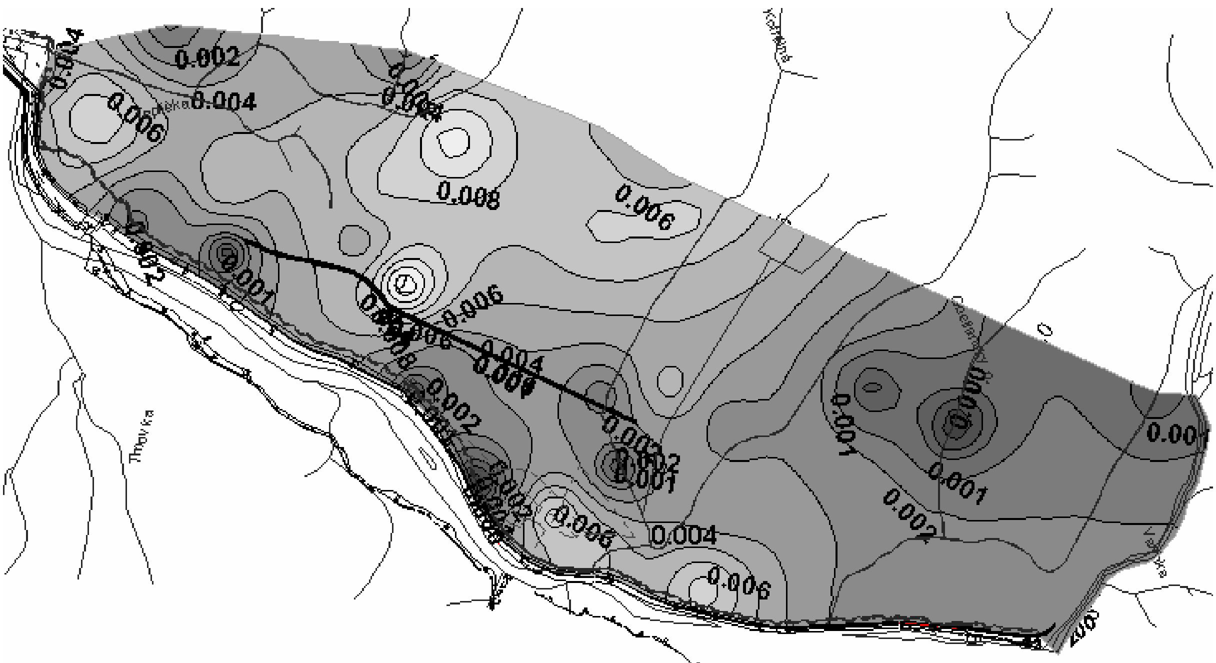


Fig. 2a Map of isolines of hydraulic conductivity (m.s^{-1}).

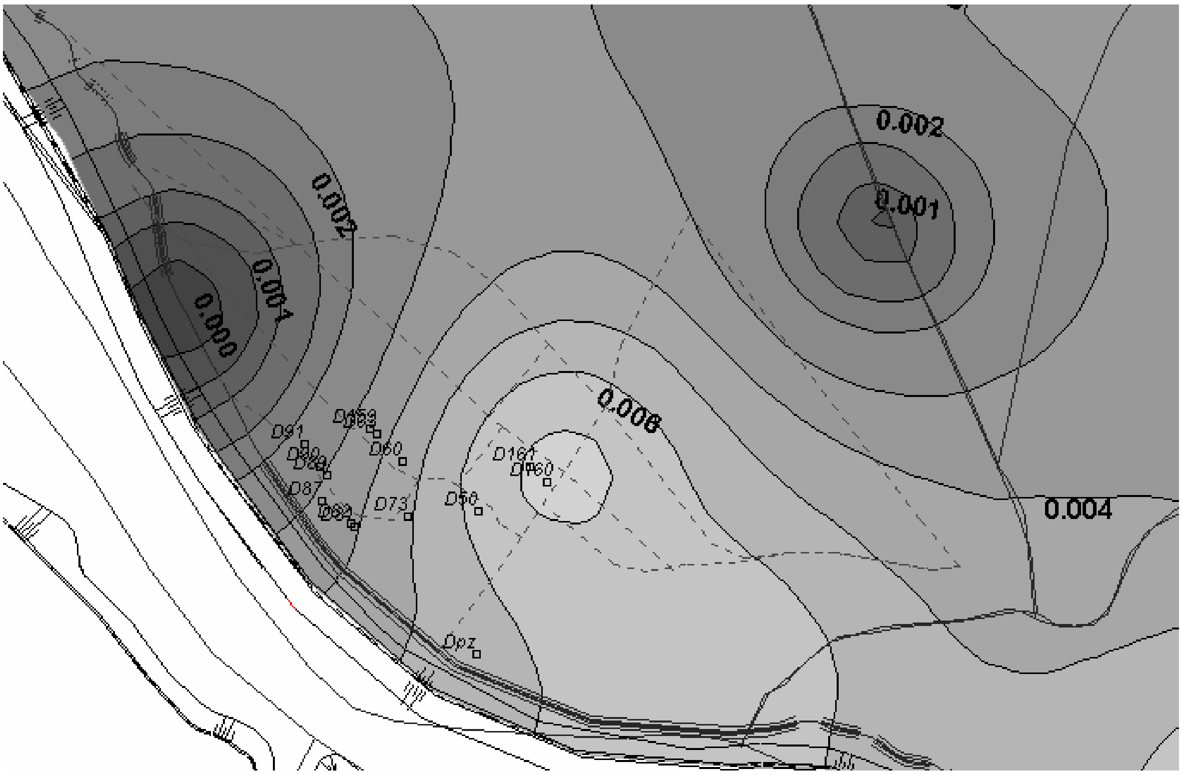


Fig. 2b Map of isolines of hydraulic conductivity (m.s^{-1}) – detail in Mojš village.

As the Fig.3a and Fig.3b suggest, the thickness of the cover layers in the whole area (especially in the village of Mojš) is relatively low. The minimum value measured 0,5 m beneath the terrain proves that the cover layers cannot fulfil the function of an isolator of the aquifer, not only quantitatively but also qualitatively. It means that Mojš tends to acquire a high amount of water at intensive precipitation events thus causing in a short time period excessive increase in the ground water level.

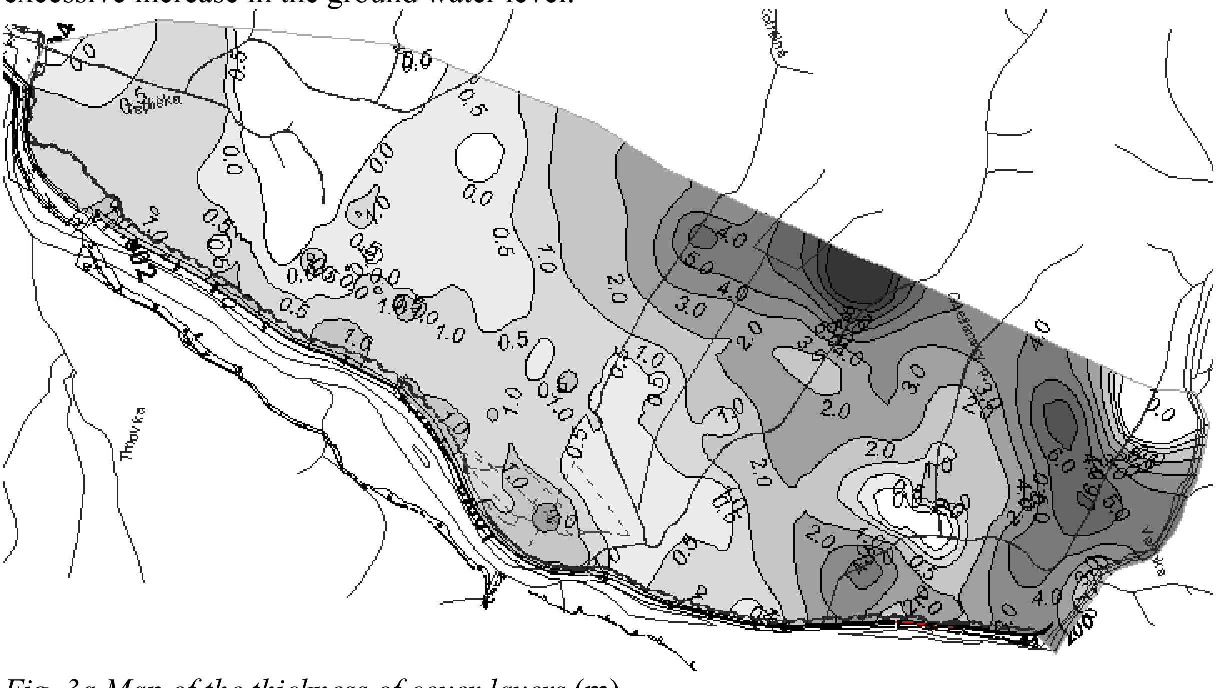


Fig. 3a Map of the thickness of cover layers (m).

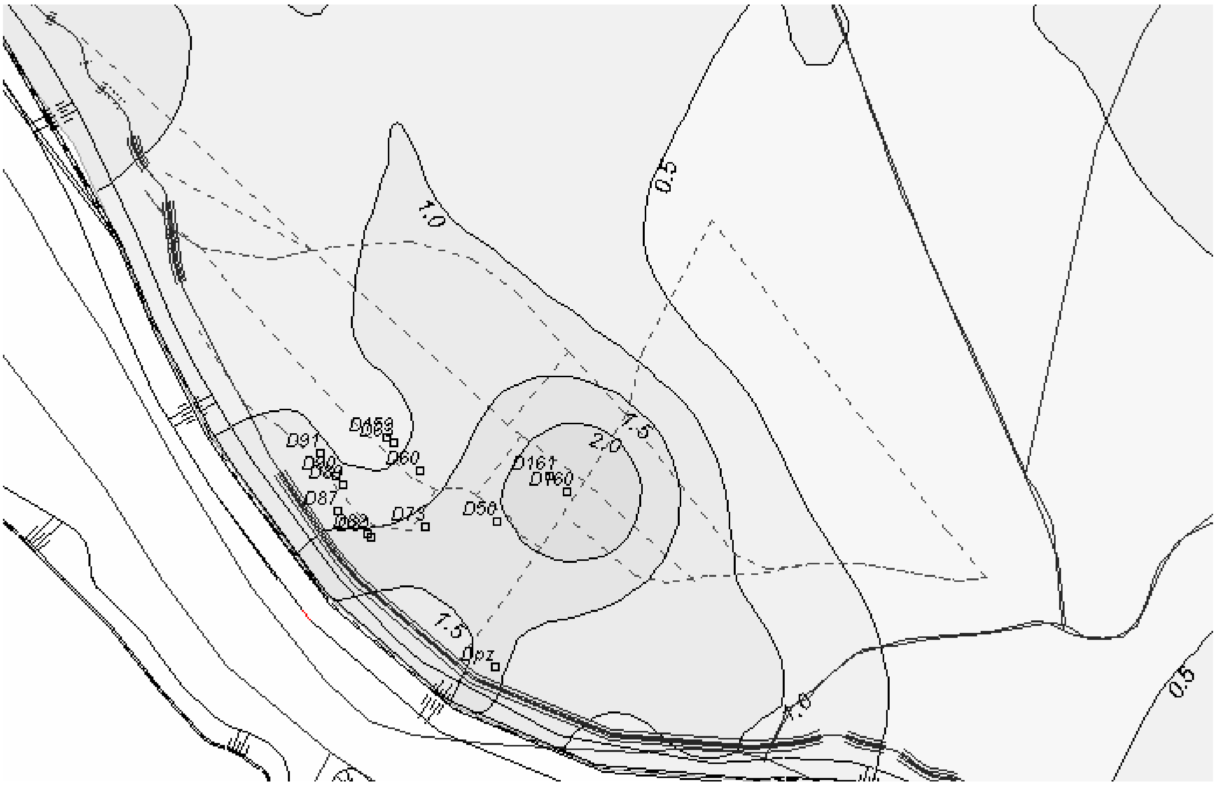


Fig. 3b Map of the thickness of cover layers (m) - detail in Mojš village.

4 Determination of the boundaries of the filtration area

Firstly, it becomes necessary from hydrological viewpoint to mark correctly the examined area. Fig. 4 clearly shows the selected edges of the investigated area. The area is marked in the south by the alternate bio-corridor, where the boundary conditions were represented by the water level of the bio-corridor. The water level is monitored alongside the entire bio-corridor in three points - B1, B2, and B3. The eastern part offered an opportunity to mark the area by the Varínka river. Lacking the information about the water level regime of this river and having studied the groundwater level measurements, we decided to narrow the investigated area and as the boundary conditions we chose the water levels in the bore-holes of SHMI (L-320, L-322, L–328), alternatively the regular monitoring bore-holes. The most difficult task was to determine the boundary of the area from the north. On the basis of hydrological foundations and measurements of the mode we tried to demarcate the inflow from the Kysuce mountains as the boundary condition. From the western side we interconnected the monitoring sampler VČ–6 and L–318 with the confluence of the alternate bio-corridor and the Váh river below the hydraulic structure. Fig.4 shows the samplers, which could be used by the calibration of the mathematical model. There were not many of these calibration points, however, they helped us set the model to selected hydrological conditions.

5 Selection of calibration periods

Based on the results of the complex monitoring of groundwater level measurements in the years 1999, 2000, 2001 and part of 2002 (Kadlec et al., 2001), the table of the extreme figures of groundwater level measurements in the years 1999 and 2000 (Kadlec et al., 2001), groundwater level regime measurements in the bore-holes of the monitoring network of

SHMI and precipitation rate, we selected extreme operation periods, selected as being the calibration ones. At the same time we paid attention to the fact that the selected periods must be from recent years and thus well – documented. The final selection is shown in the Fig. 5.

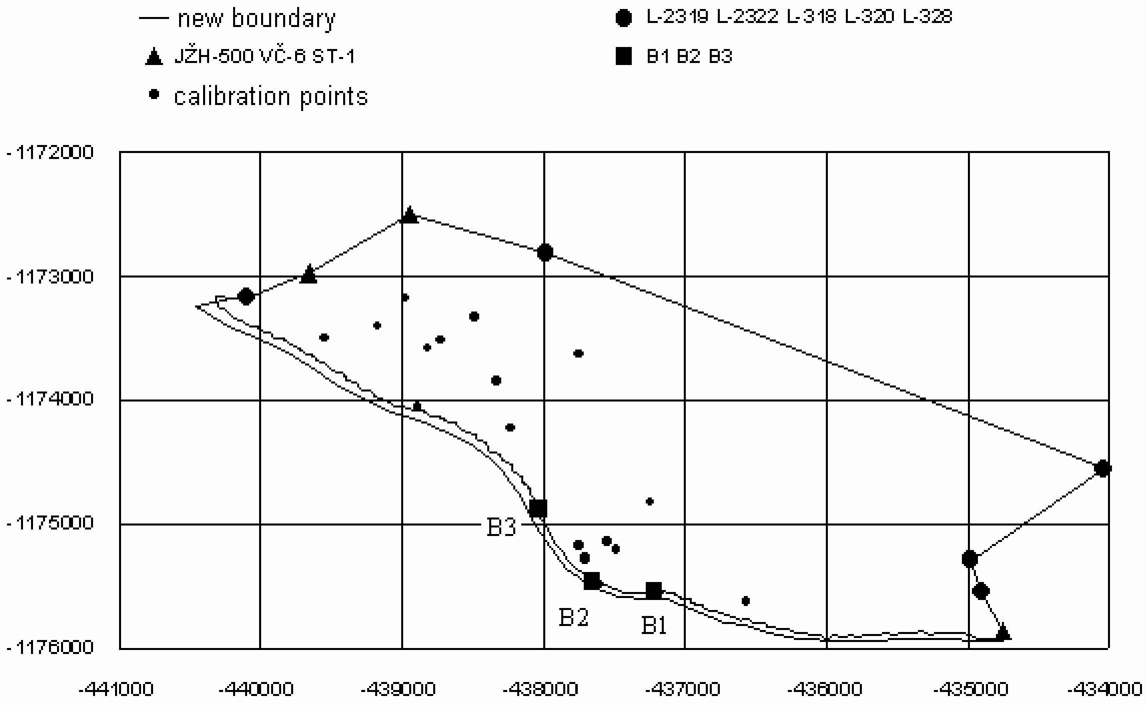


Fig. 4 Investigated area - illustration of calibration boreholes and sonds used for determination of boundary condition

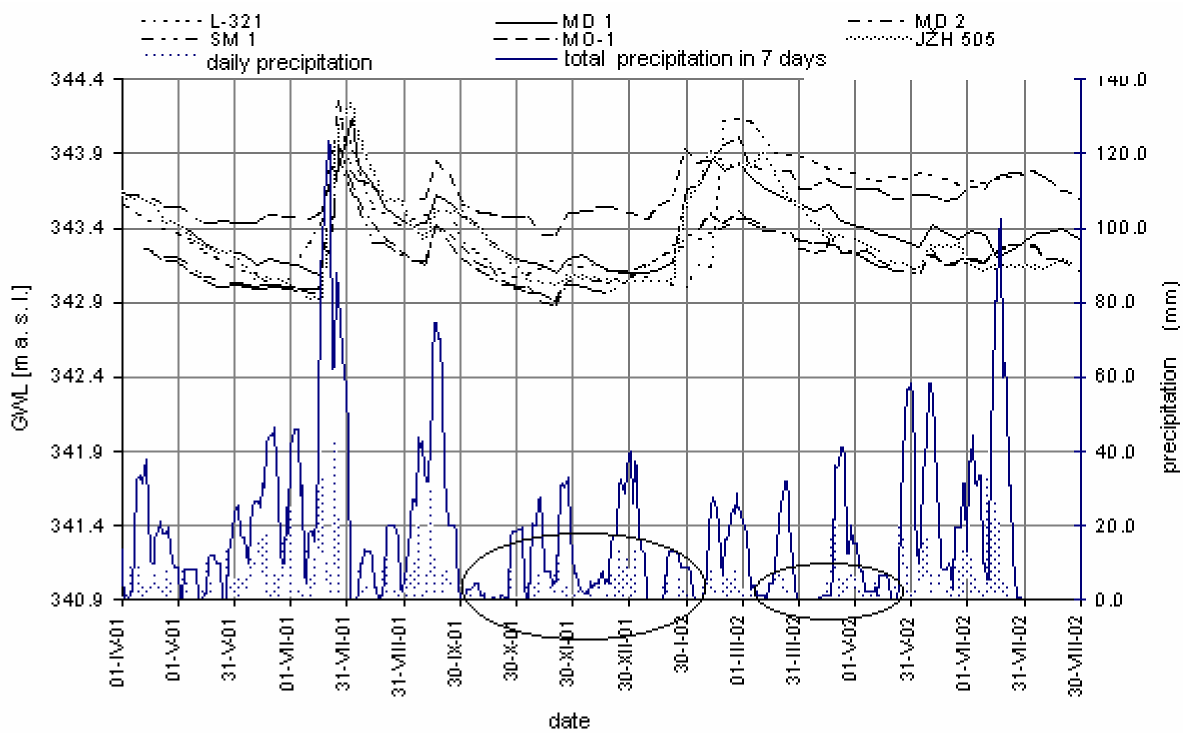


Fig. 5 Course of groundwater level in bore-holes and precipitation in Mojš village. Illustration of the calibration periods.

Primarily, we have selected the calibration period showing the most intensive amount of precipitation. It is the period of the last decade in July 2001, producing 123 mm of rainfall in 7 days. The groundwater level in the bore-holes reached its maximum, predominantly in the village of Mojš (Fig.5).
The second calibration period is represented by a stable period with relatively low groundwater level – the other extreme, intended to mark the potential order of magnitude of the groundwater level regime in the right alluvium of Žilina WS. (May 2002). We selected the boundary conditions of two-dimensional groundwater flow in the calibration periods as the conditions of the 1-st order, i.e. the water level in the bore-holes, or possibly in the river-bed of the alternate bio-corridor. These were determined as a mean value in the calibration period. The results of the model calibration are elaborated more detailed in the study. (Šoltész et al., 2002).

6 Proposal of technical measures for groundwater decrease in the village of Mojš

Based on the given geological, hydrogeological, and morphological properties of the investigated area, and hydrological conditions for extreme cases we were able to propose technical measures for decreasing the groundwater level regime in Mojš. We consider various alternatives of measures, verified their efficiency until we reached the final proposal.
We were aware of the fact that the groundwater flow into the area of Mojš from the north-east, and at the same time the area is exposed to massive rainfall and snow melting. Due to thin cover layers in the alluvium of the Váh river, groundwater is constantly infiltrated by the rainwater. This fact greatly influenced possible technical measures proposed in the numerical model.
The technical measures used in investigation can be divided as follows (Fig. 6):

- Pumping from wells built in Mojš,
- Construction of drains in various modifications (draining in existing, or planned communications)
- Deepening the river-bed of Kotrčina and Gbely streams (numbered 3 and 4)
- The water level control in the alternate bio-corridor (numbered 9)

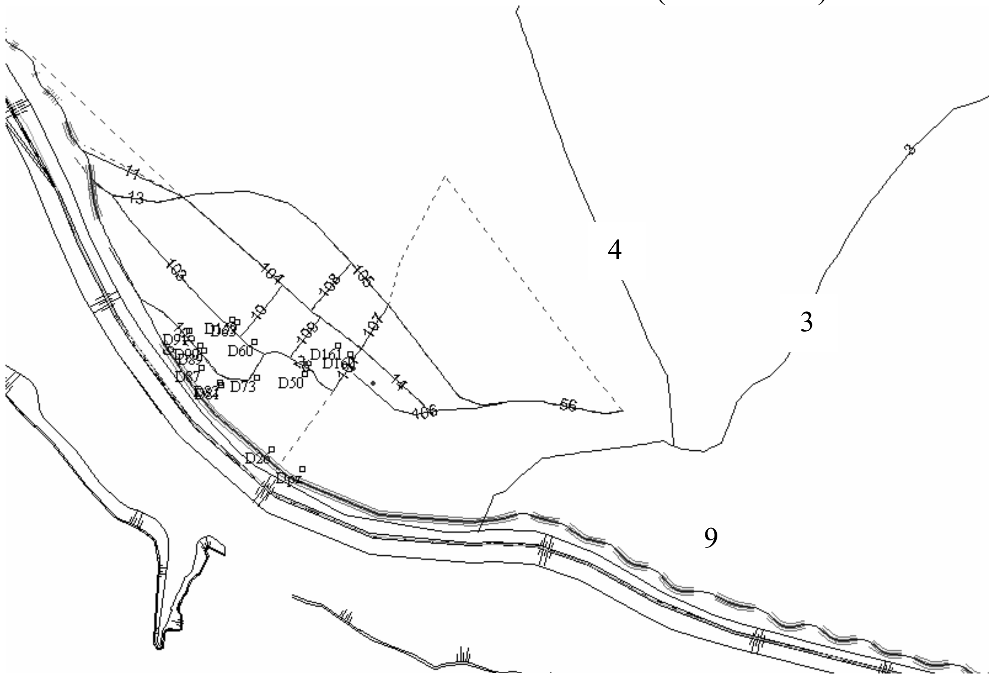


Fig. 6 Proposal of all technical measures in the village of Mojš.

The first issue we have dealt with was varied pumping quantities from the private-owned wells used to monitor the groundwater level regime. We used this procedure in case of the most unfavourable conditions. The outcome of this scenario pointed out to the local efficiency of drainage by means of wells. Apart from this fact, we have to realise that this precaution requires the use of electricity and the drainage may cause the instability of soil in porous medium with consequent static's failure of the houses.

Another possibility to treat the unfavourable water level range was the proposal of draining in various modifications. We thought of this option as the most economical solution, sufficiently ensuring the drainage of redundant water from the area of Mojš. The proposed solutions were always designed for the extreme hydrological conditions. With respect to the fact that the drainage sluices the redundant water due to gravity, the fundamental premise of this procedure is that the infiltrating water is safely drained away. A suitable recipient for the drained water proved to be the alternate bio-corridor. We verified this option and came to a positive conclusion and hence we proceeded modelling these variants.

Fig. 7 shows only four of all scenarios of drains locations. The first one requires building the drainage system No.4 and, in a lower-placed street the drainage system No.2. (Fig. 7a) The other variant shown in the Fig. 7 requires building of the drainage system No. 5 located above the inhabited part of Mojš in the direction of the planned road (Fig. 7b). Scenario shown in Fig. 7c consists of the main downward drainage, which is interconnected with the bio-corridor in the direction of the planned road communication. It requires the construction of another secondary collecting drains built on both sides of the main collecting drain. The final variant, shown in the Fig. 7d, only differs from the previous one in the fact that the collecting drain below the village is headed directly into the lower-placed reach of the bio-corridor stream channel.

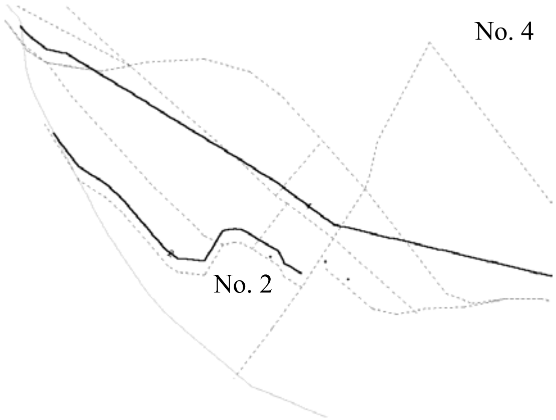
The results of the modelling showed that integrating these drainage elements into the filtration area caused the improvement of the situation, especially in the northern part, where the effect of the measures was clearly observable. The weakest solution is certainly the scenario of drainage system No.5, which is placed too much up to the north and its effect is subtle. The results of the first model (Fig. 7a) show that this variant might meet the requirements of an efficient draining of the redundant water, however, there is a problem with the construction of the draining system itself (excessively narrow street, engineering system). The results of the modelling shown in the Fig. 7d point out to the fact that this variant would reliably solve the situation in the northern part of the village even in the case of extreme hydrological conditions. However, it would fail to resolve the negative situation in the houses near the bio-corridor.

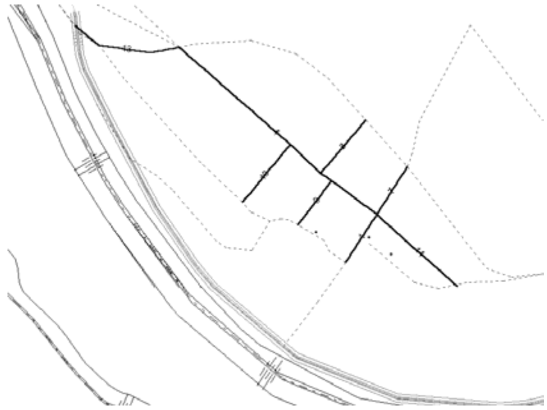
As we later implied by the model calculation of proposed solutions, the individual technical measures on their own can hardly function in dewatering of the redundant groundwater. However, if we were to meet the owner's requirements we had to proceed to resolving another scenarios, which came to existence as a combination of particular technical measures.

The first scenario examined was pumping from three wells in the village of Mojš at current water level decrease in the bio-corridor to the minimum level. The results of fitting to the most unfavourable hydrological status proves that this variant is likely to suit the majority of houses in the village, predominantly the areas where the effect of the pumping well is detected. We have previously discussed the inconvenience of the pumping in municipal areas and this variant confirmed our theory again (Šoltész et al., 2002).

Another combined scenario was the possibility of deepening the river-beds of Kotrčina and Gbely stream (in the Fig. 6 theses are numbered as 3 and 4) with simultaneous lowering the water level in the bio-corridor to the minimum operational level. This variant would cause all of the houses to be at dry places. We only have to take into account that this model calculation merely applied to the maximum operational time periods. To see the whole situation at the model calculation applied to the most extreme condition, we also did this calculation the results of which are elaborated in the research report (Šoltész et al., 2002). The calculation suggests that this variant at extreme rainfall activity in the area of Mojš would be inefficient predominantly in the northern and western part of the village. Beside that, it is worth emphasising that deepening of the river bed would require enormous groundwork (deepening by three meters), which makes the procedure financially restricted.

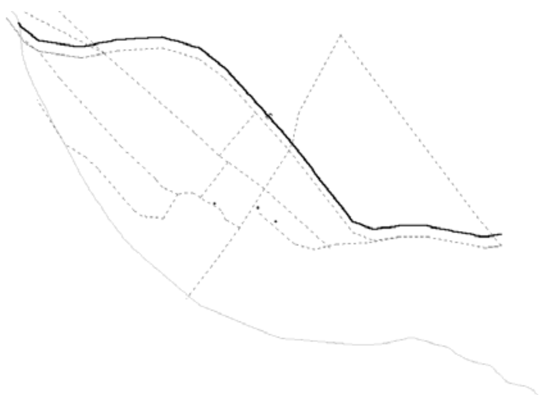
No. 4

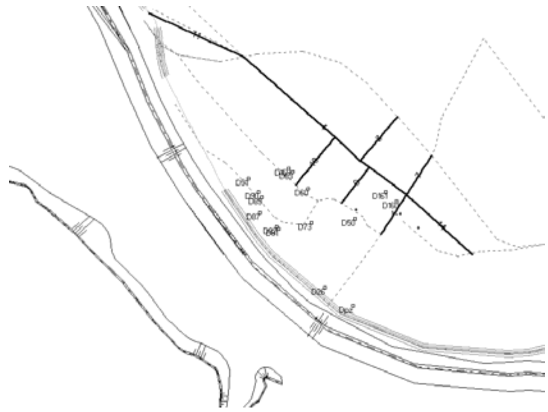




(7a) *(7c)*

No. 5





(7b) *(7d)*

Fig. 7 Proposal of location scenarios of dewatering drains.

In the next step we dealt with the combination of the main drainage system design (No.4, Fig. 7c, 7d) with lowering the water level in the bio-corridor to the minimum level. This variant would meet the objectives even at the extreme hydrological condition. A similar situation also arises when considering that the last part of the drain (no.11) is not active, i.e. it fails to collect the redundant water and serves merely as a transport pipeline. This variant also enables the house cellars to be dry.

7 Conclusions

In accordance with the results of the modelling calculations during the solution we have come to the conclusion which we consider the most realistic. It is based on the possibility to control the water level regime in the bio-corridor in the river section from B2 (below the footbridge) down to the profile B3 (the bridge below the village) (Fig. 4).

To the lowered water level in the section of the bio-corridor from B2 to B3 (which we ourselves proposed) we applied the scenario of the main drainage system with two additional collecting drains. The results of modelling this type of solution in the extremely negative hydrological conditions prove that this variant with simultaneous lowering the water level in the bio-corridor by 20 cm is sufficient and by lowering the level by 40cm absolutely convenient. The house cellars in the whole village are in this case in the dry. If we exclude from this model the secondary collecting drainage system n.9, we shall obtain similar results, which are only a little less unfavourable than in the previous scenario, but still suitable. The modelling results are shown in the Fig. 8.

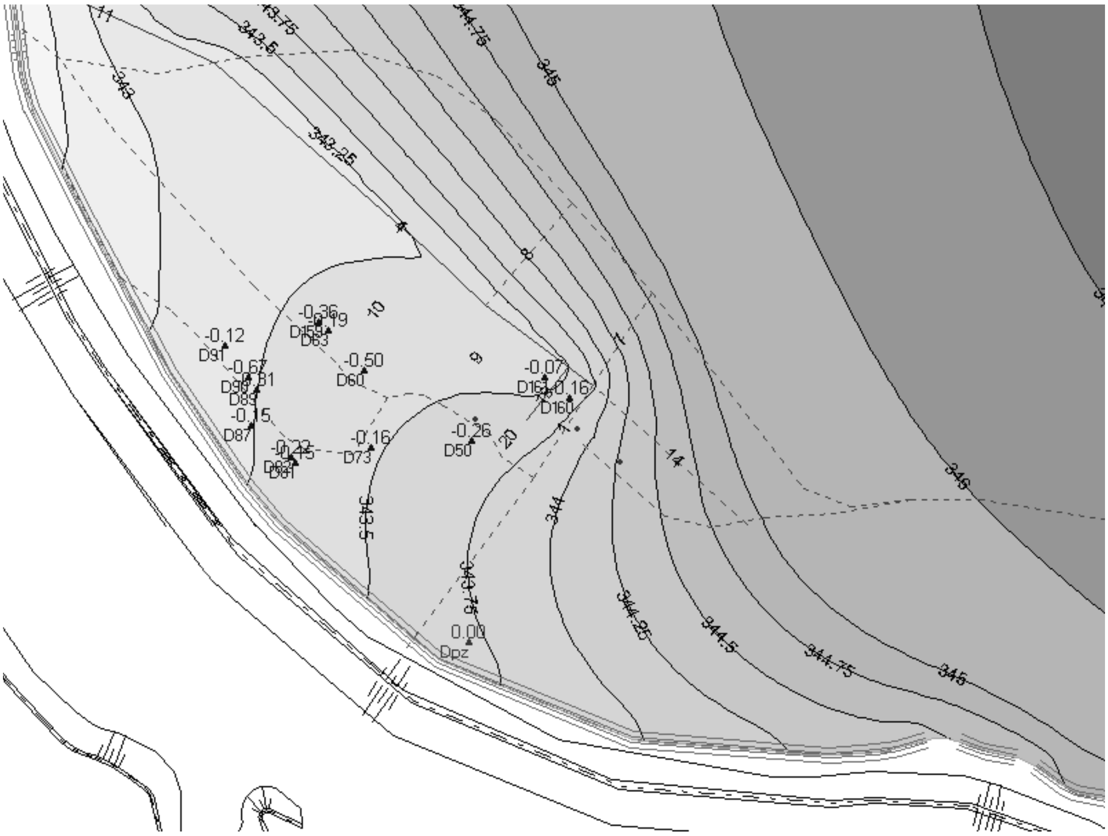


Fig. 8 Modelling of the most unfavourable hydrological situation for scenario: drainage element 11, 4 19 and 40 cm decrease of water level in bio-corridor in the reach from "B2" to "B3".

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VIII. INTERNATIONAL SYMPOSIUM ON
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Degradation Process of Asphalt Concrete Lining of Upper Reservoir in Zarnowiec Pumped Storage Power Plant

Stefan Bednarczyk, Remigiusz Duszyński[1], Piotr Książek[2]

Abstract: The paper deals with some faults and deficiencies of bituminous linings of the upper reservoir of the pumped storage power plant Zarnowiec. The heaves in the bottom of reservoir and the breaking of asphalt concrete lining upon the line of drains were described. The reasons of these phenomena were analysed and methods of renovation were characterised.

1. Introduction

The power plant, commissioned in 1983 and situated in Zarnowieckie lake close to Gdańsk, is the biggest pumped storage power plant in Poland. It makes use of the Zarnowieckie lake as a lower reservoir. The artificial upper reservoir of useful capacity of 13,8 million cubic meters was constructed on a hill distant about 1500 meter from the power plant. Zarnowiec power plant is equipped with four equal Francis reversible pump turbine generator motor units - each of capacity 179 MW. The main technical data of Żarnowiec power plant were presented in table 1.

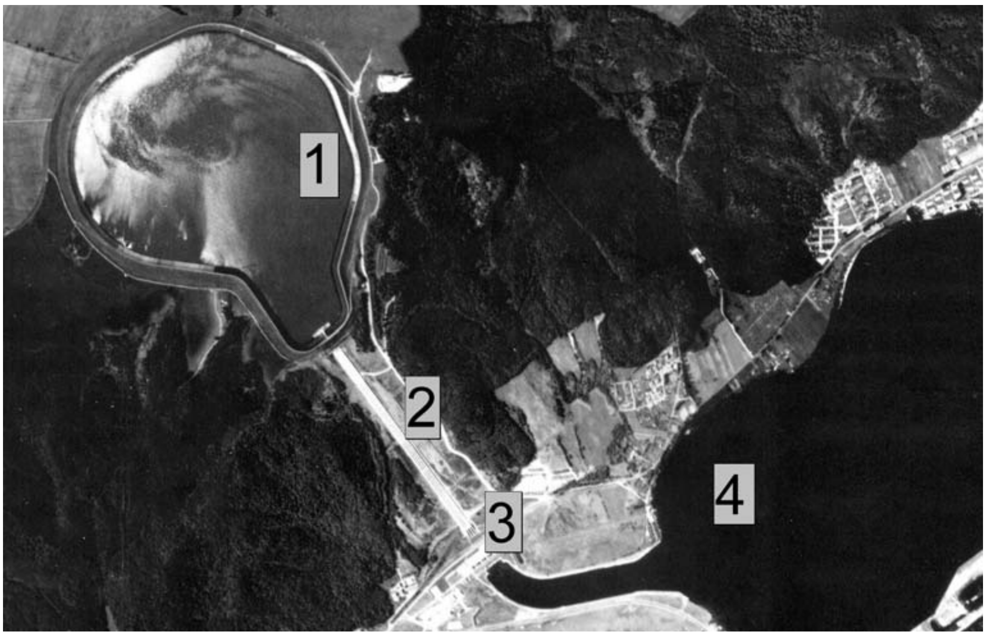

Fig. 1. Aerial view on the pumped storage power plant Zarnowiec
(1-upper reservoir, 2- pipelines, 3-power plant building, 4-Zarnowieckie Lake).

[1] Faculty of Hydro and Environmental Engineering, Gdańsk University of Technology, ul. Narutowicza 11/12, 80-952 Gdańsk, Poland
[2] Żarnowiec Pumped Storage Power Plant, Poland

The average total starting time for full operational power for generating mode is 180 sec and 400 sec for pumping mode. The process is fully automatic remotely controlled by Domestic Dispatching Centre in Warsaw. Short starting time allows not only to cover the demand for the peak power load but also to fulfil its interventional task for Domestic Energy System to quickly cover the power deficit. Zarnowiec power plant can also work for reactive compensation mode.

Table 1. Technical data of Żarnowiec pumped storage power plant:

Installed capacity for generating mode pumping mode	 716 MW (4 x 179 MW) 800 MW (4 x 200 MW)
Lower reservoir (Zarnowieckie lake) lake surface water surface level	 1470 ha 1,0÷2,0 m a.s.l.
Upper reservoir (artificial) reservoir surface useful capacity water surface level	 135 ha 13,8 million m^3 126 m a.s.l.

2. Technical data of the upper reservoir

The upper reservoir was built on the moraine hill located over 2 km far from Zarnowieckie Lake. Embankments of upper reservoir were made of soil excavated from the hill. The main part of these soils is clay. The typical cross section of embankments is presented at figure 2, and the parameters of upper reservoir are collected in table 1.

The water from the reservoir is conducted on turbines by four steel pipelines of total length about 1100 m each, and diameter varied from 7000 mm to 5400 mm. The maximum fluctuation of the water surface level is 16 m high and occurs in 5 hours of turbine work and in 7 hours of pumped work.

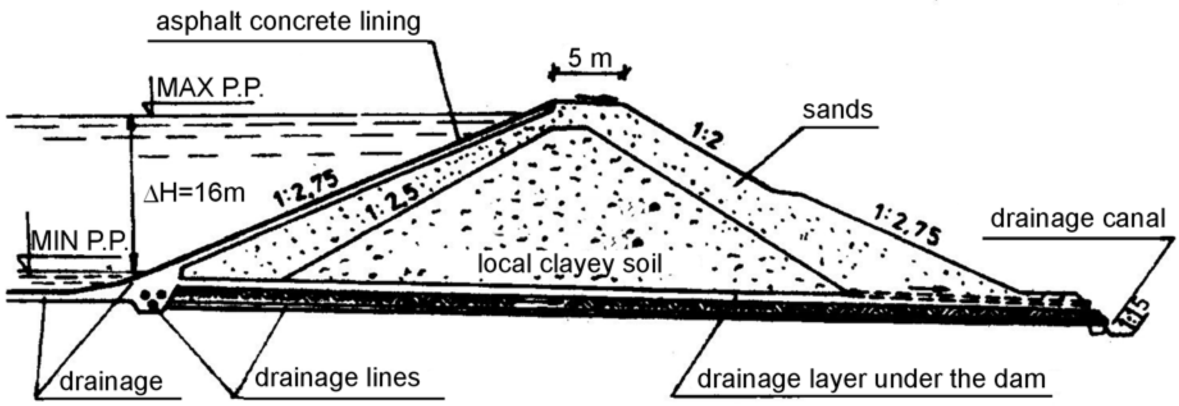

Fig. 2. Typical cross section of upper reservoir embankments

Water side slopes of the dam and the bottom part of the reservoir are covered with the asphalt concrete lining places on the gravel drainage layer, which thickness is 24 cm on the slopes and 40 cm on the bottom. The filtration coefficient of drainage layer is 500m/day.

The asphalt concrete lining is composed of two different parts. Each of them is about 7 cm thick. The lower part of asphalt concrete has an open structure and was placed on the drainage layer covered by the cationic emulsion (5 kg/m^2). The upper part of asphalt concrete layer, placed on the bottom part, is watertight. The asphalt concrete was placed directly on the soil surface mainly because of technical and financial reasons.

3. The characteristics of upper reservoir asphalt concrete linings

Each layer of asphalt concrete lining was prepared of component mixed in accordance with recipe presented in table 2.

Table 2. Asphalt concrete components (%)

Material	base course layer	watertight layer	sandy asphalt concrete
basaltic grit 12/16	26,7	-	-
basaltic grit 8/12	19,2	18,6	-
basaltic grit 5/8	14,3	18,6-	-
basaltic grit 2/5	9,6	18,6	-
basaltic fines 0/2	-	28,9	36,8
sand	20,0	18,6	41,4
calcareous meal	5,7	8,5	13,8
road asphalt D70	4,5	6,8	8
Summary %	100	100	100

The asphalt concrete was prepared, conveyed, placed and compacted in accordance with all generally accepted rules and standard recommendations. Designed thickness of asphalt concrete lining on slopes was maintained. On the bottom of the reservoir, due to the insufficient equality of surface, asphalt concrete lining thickness varied between 2 and 10 cm. Check tests confirmed good quality of asphalt concrete lining, which has fulfilled all physical and mechanical parameters requirements specified by design engineers. The asphalt concrete had a good watertightness ($k \leq 10^{-9}$ cm/sec) and stability, was fully resistant on flow of and had correct flexibility. Except joints, the whole surface of the asphalt concrete lining was even and smooth.

The original project demand to cover all asphalt concrete by mastic and bituminous emulsion. However the results of check tests were good and engineers abandoned those layers. After failure of asphalt concrete lining in 1982, engineers returned to original project and between 1984 and 1985, near 60% of upper reservoir slopes have been covered with protection layer made of asphalt D200, PS-175 and 145. All repairs were made manually. The hot liquid asphalt were placed on the cold inadequate clean surface, so the connection between new and old layers was improper and asphalt has flowed to the bottom of slope.

4. Damage of bottom lining due to compressed air present in soil subsurface

In June 1979, when near 70% of the reservoir bottom was covered with asphalt concrete, after sudden night rainfall many bottom heaves occurred (fig. 3). They have had maximum diameter of 150 cm and maximum height of 50 cm. Most of heaves have scratched or cracked on the top. The compressed air has flowed through those cracks.

The heaves were caused by two reasons. Firstly the atmospheric pressure had suddenly decreased for 12 hPa after 2 months of high barometric pressure. Secondly the drainage in bottom part of slopes has been completely silted-up by fine parts of soil flushed with rainfalls. When the rain was falling the upper part of the slope was uncovered so the rainwater has flowed into drainage layer and toward into the soil under the bottom of the reservoir. Migration of water caused high pressure of air present in soil pores.

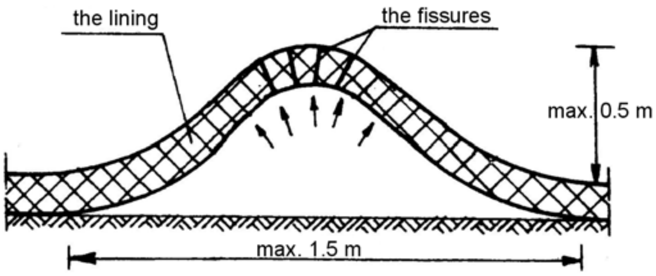

Fig. 3. The bottom air heave

The high difference between atmospheric and under bottom air pressure was the main reason of the damages. All heaves have appeared in those places, where the lining was thinner than 5 cm. Repairs were made by cutting of the heaves and making cork of the asphalt concrete (fig. 4b).

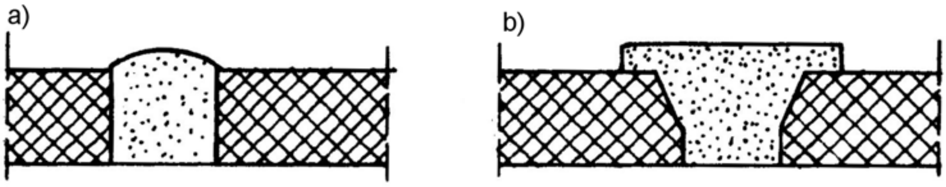

Fig. 4. Asphalt concrete cores in bottom lining a) incorrectly arranged, b) correctly arranged

5. Breakdown of asphalt concrete lining upon slope toe drainage

Few months after first start-up, 27th of July 1982, sudden breakdown of 250 m asphalt concrete lining situated along the toe of the slope took place. Damage was very dangerous because the water has flowed through the whole cross section of drainage washing out fines particles of drainage aggregate. Probably the improper rinsing of silted-up drainage has caused the failure. Too high water flow has washed out not only soil particles causing silting but also smaller particles of drainage aggregate. The every 10 m bored holes in bottom lining were corked with asphalt concrete, but unfortunately at least one was made inadequate (fig. 4a), so the water pressure has pushed them through the drainage layer making inlet into the bottom subsoil.

Damaged asphalt concrete was removed, drainage was perfectly repaired and the new asphalt concrete lining were places at the area of 5 m width and 250 m long.

6. Puncture and collapses of bottom asphalt concrete lining

During the next start-up tankage of upper reservoir to the 120 m a.s.l. many punctures and collapses of asphalt concrete lining has occurred.

Some hard objects left in the subsoil of the bottom lining have caused punctures. There were some steel, concrete or wooden elements, which act as a knife on the bottom asphalt concrete lining loaded by high water level pressure. Prior to the renovation of these punctures all hard objects were removed from subsoil. Then the bottom was corked with the new asphalt concrete.

Insufficient compacted and stabilised subsoil has caused collapses of bottom lining. In this cases the lining has not been cracked but each and every collapse was filled with the asphalt concrete.

7. The collapse of bottom asphalt concrete lining upon the shaft well

The upper reservoir was located on the former farm terrain. There was a shaft well localised directly in the bottom part of the reservoir. The well was filled with sand and covered with asphalt concrete lining as the rest of the bottom. In 1988 the big collapse of asphalt concrete lining was discovered. The asphalt concrete lining was pushed 2 meters deep into the shaft well by the water pressure. Close to the shaft well, quite a few failures of bottom lining were noticed. There were collapses and heaves width 3 m diameter and height (depth) of 50 cm. There were also furrows 20 cm deep and 20 to 50 cm width with 20 m overall length.

All those failures were caused due to the water flow under the bottom lining of reservoir. The collapsed well was the inlet.

8. The smallpox of the slope asphalt concrete lining

The large quantities of the smallpox (fig. 5) on the slope lining of the upper reservoir were confirmed after 10 years of Zarnowiec pumped storage power plant working. The smallpox was situated mainly in the area of the highest water level variation, nearby the joints, where the closing bituminous layer had not properly done.

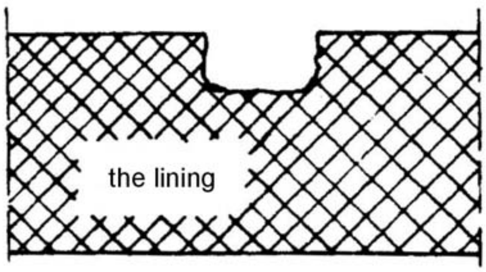

Fig. 5. The smallpox

Till now loosing some particles of basaltic grit causes the smallpox. The grits are washed-out by the water or ice.

There are many smallpox blisters in the same area where the smallpox is situated. The gravel grits moving to the top surface of lining (fig. 6) cause the smallpox blisters. When the grip is out the big smallpox hole appears.

Inadequate compaction of asphalt concrete mixture in the area of joints is the main reason of that kind of failures.

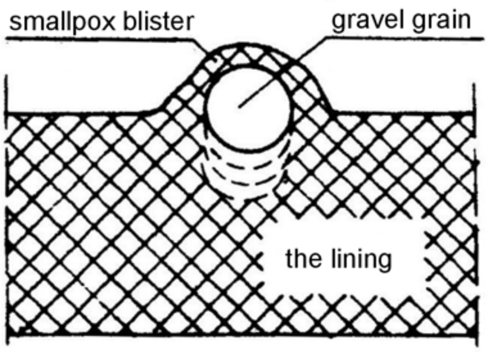

Fig. 6. The smallpox blister

9. Breaking of asphalt concrete lining upon drainage layer

In July 2001 at the toe of slope the lining upon the drainage layer collapsed causing some outflow of water trough the drainage. Some small particles of drainage aggregate were washed off. The main reason of this breaking was insufficient elasticity of asphalt concrete lining. One year after renovation the same kind of failure was noticed close to the renewed area.

10. Conclusions

All presented faults and failures were quickly and well repaired. They do not threaten the safety of the whole reservoir structure. Well made drainage system is able to take over leakage caused by every type of asphalt concrete lining faults.

From the beginning bituminous layer has a limited tension resistance and strain capability. During 20 years of exploitation resistance parameters were weaken by the ageing process of bituminous. The robust repairs and proper maintenance guaranty appropriate exploitation of the upper reservoir for the next years.

Main reason of many fault was insufficient compaction of bituminous material and no thorough closing of joint in the upper layer of asphalt concrete lining. Also the improper content of bituminous material in asphalt concrete mixture, the lack of protecting layer of mastic had influences on occurred faults.

11. References

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Hydrodynamics of Water and Sediment Variability within the Different Gravel Bar Regions of the Mountain Stream

Tadeusz Bednarczyk, Artur Radecki-Pawlik

Department of Water Engineering, Agricultural University of Cracow, Poland
30-059 Cracow, Al. Mickiewicza 24-28, **e-mail:** rmradeck@cyf-kr.edu.pl

Abstract
The purpose of the paper is to identify differences in hydraulic conditions in the areas closest to already-developed gravel bars situated in a small, mountain stream in the area of the Polish part of the Carpathian Mountains. Basic hydraulic parameters of flowing water, including velocity, shear stress, Froude number, Reynolds number and flow resistance coefficient were examined within the region of two different gravel bars in a mountain stream. At the same time, sediment samples were drawn from the riverbed in the area in which the hydraulics measurements were taken. After analyzing the data, several conclusions were presented concerning sedimentation of gravel and hydraulics parameters within the cross section of a mountainous stream. The study was undertaken on the Skawica-Ja³owiecki Stream in the Polish part of the Carpathian Mountains.

Key words: gravel bar, shear stress, shear velocity, bedload, hydraulics parameters, mountain stream

“The River lifts itself from its long bed. Poised wholly on its dream“ (by Hart Crane, from *The River*.)

Introduction

River bars and similar features, are typical of all rivers and are mimicked in other linear shear flows [Church & Jones 1982]. So far, no formal criterion has been developed for the occurrence of bars in terms of flow and sediment characteristics. Bars, defined as accumulations of sediment grains, or sand and/or gravel deposits [Whittow 1984], cannot develop if the flow depth is approximately equal to the minimum grain size [Church and Jones 1982]. In alluvial channels, three types of bars are commonly recognized: alternating bars, point bars and braid bars [Selby 1985]. Alternating bars form in straight channel segments within curves of meandering thawleg. Point bars develop in areas of relatively low stream power at the inside of channel meander. Braid bars, mostly diamond-shaped, are often associated with coarse material. They are aligned to the flow and are called longitudinal bars [Selby 1985].
Although bar forms have been commonly described in sandy or gravelly meandering rivers, little attention has been given to the role of obstructions in controlling geomorphic forms in coarse-grained environments [Carling and Reader 1981, De Jong and Ergenzinger 1995]. As far as the geometry of mountain streams is concerned, bars start to develop in the middle and lower reaches where the channels reach high width-to-depth ratios [Chang 1980]. Deposition of a river bar is directly related to bend curvature, reflecting particularly the role of sharp bands in arresting sediment under lower energy conditions. Other reasons for bar depositions are obstructions or hindrances caused by large boulders or bedrocks (a common spot for the formation of re-attachment bars), or wooden logs behind which sediment is trapped.

There are only a few studies on the role played by bars in a gravel stream environment, particularly in shaping a river channel and stopping bank erosion. Research has suggested that bars should be treated as roughness elements, equally or perhaps more important than channel

bed roughness associated with the dimensions of grains that form the river bed [Radecki-Pawlik 2002b]. This finding is of great interest, particularly in relation to high floods.
The purpose of the current paper is to identify differences in hydraulic conditions in the areas closest to already-developed gravel bars situated in a small, mountain stream in the area of the Polish part of the Carpathian Mountains. The paper also describes a difference in bedload granulometry that was found in the area in which the hydraulics parameters were examined.

Study area

The upper part of the Skawica-Ja³owiecki Stream in the Polish part of the Carpathian Mountains (Figure 1) is flashy and experiences frequent bedload movement. It is situated in the Carpathian flysh, and its streambed consists mostly of sandstone and mudstone bedload pebbles and cobbles that form a framework, the interstices of which are filled by a matrix of finer sediment.

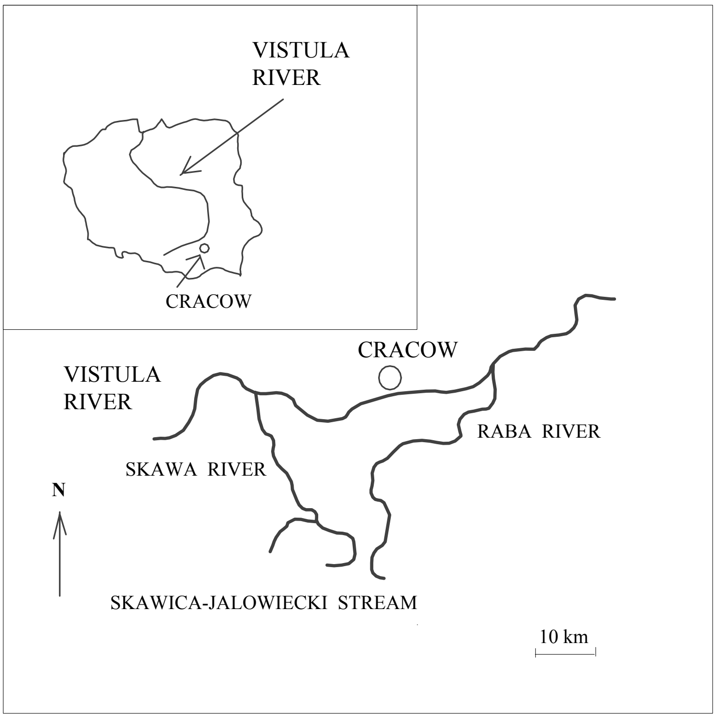

Figure 1. Catchment study region

Variables	Skawica-Jałowiecki Stream
Precipitation [mm]	1189
Catchment study area [km^2]	19,3
Maximum stream altitude [m a.s.l.]	1130
Minimum research point altitude [m a.s.l.]	594
Channel gradient (average within study area) [-]	0,085
Minimum annual discharge [$m^3 \cdot s^{-1}$]	0,020
Mean annual discharge [$m^3 \cdot s^{-1}$]	0,460

Table 1. Physical characteristics of the Skawica-Jalowiecki Stream catchment

Suspended sediment loads are small and contribute insignificantly to channel morphology. Within the 1109.5 m long study reach, the Skawica-Ja³owiecki cuts through an alluvial bed, mostly Quaternary Holocene river gravel, sand and mudstone [Ksi¹zkiewicz 1963]. The upstream portion of the study reach just borders upon a Tertiary Palaeogene reach where mica-sandstone, sandstone, mudstone and phyllite predominate.

Many gravel river bed-forms, such as point and middle bars, can be seen within the investigated Skawica-Jalowiecki reach. Most gravel bed-forms have developed behind and in front of obstacles, and those situated at riverbanks (meander-bars) are quite stable. The detailed geology and geomorphology of the region has been described in Radecki-Pawlik [2002a]. Some basic physical characteristics of the catchment study area are presented in Table 1.

Methods

For the purpose of the study, a 1109.5 meter-long research reach was selected within Zawoja municipality. Identification and field measurements of bed-forms were carried out during autumn 1999 and early spring 2000. The study was based on a hydraulic survey of water velocity close to the stream bed to calculate shear stress, drag coefficient and other hydraulic characteristics. Two well-developed stream bed features were recognised in the form of Point Bar A, situated downstream of a stream band at the upper part of the reach, and Alternation Bar B, an upstream-of-obstruction bar attached to the right bank of the creek (the obstruction in this case being a radiolarite megacluster). Research cross-sections were established within the regions of the two bars, and measuring points were fixed within them. Wading velocity measurements were performed at all of these points (Figure 2).

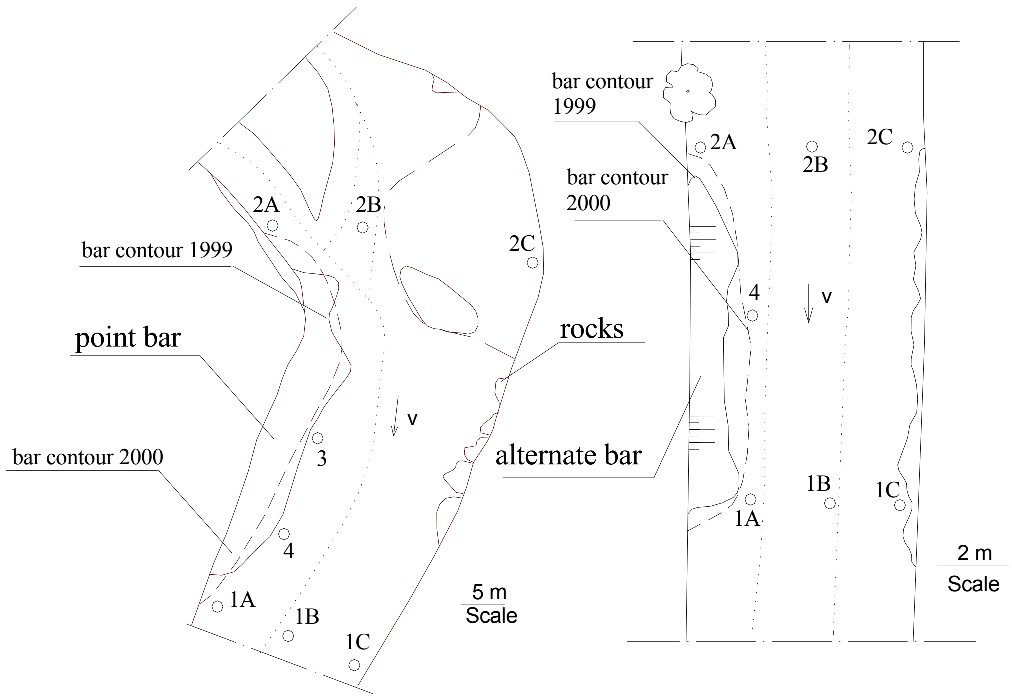

Figure 2. Investigated gravel bars and measuring points

Water velocity measurements were based on Jarrett's (1990) findings regarding the taking of velocity profiles in mountain stream cross-sections. Gordon & others (1992) and Bergeron & Abraham's (1992) methods were then applied to the field data, and shear velocity V_* values were calculated from the velocity profiles obtained near-to-river-bed. Finally, shear stress τ values were calculated from:

$$\tau = V_*^{\,2} \rho \quad [\mathrm{N\,m^{-2}}]$$

where: ρ - water density [kg m^{-3}] and V_* - shear velocity [m s^{-1}].

Shear stress value (V_*) was obtained just directly from the equation v = *f (D)* [Gordon et al. 1992]:

$$V_* = a / 5{,}75$$

where: a - slope coefficient v = *f (D)*,
according to the general line equation: $v = aD + b$,
where: D - water depth above the stream bed [m], b – free coefficient
To calculate the flow resistance coefficient (f), the conclusions drawn by Ven Te Chow [1967], Wijbenga [1990] and Przedwojski et al. [1995] were applied. Flow resistance coefficient was obtained from:

$$f = 8\, g\, J\, R / V_{mean}^{\;2} = 8\,(V_* / V_{mean})^{2}$$

were: R – hydraulic radius [m], J – slope [-], V_{mean} – mean velocity [m/s]
The detailed methods used to obtain values for all of the above-mentioned parameters using classic hydraulics equations are shown in Radecki-Pawlik [2002a].
Samples of bed-load sediment deposits were also collected in in the area in which the hydraulics data were gathered. The technique of sampling described by Wolman [1954] was applied. Later, grain size curves were plotted and characteristic grain dimensions were read. Additionally, for grain shape analysis, 339 single grains were chosen randomly from the riverbed and carefully measured. Grain shapes were described according to Zingg [Gradziñski et al. 1986, Gordon et al. 1992]

Results

For reasons of clarity, all results obtained are presented in tables. Tables 2 and 3 show all hydraulics parameters measured and calculated above the research points within the regions of the investigated bars (Figure 3). Wading measurements were taken under two discharges. The first was Q=0.34 m^3/s, which is close to the mean annual flow (Q=0.46 m^3/s) (Table 1). The second discharge was a spring flood, when Q reached 1.04 m^3/s. In the case of the second discharge, bedload movement under these conditions was observed. Sediment samples were taken from all the places in which the velocity measurements were done, right after the water dropped. The sediment data are presented in the form of characteristic grain dimensions (Tables 4 and 5), and as shape dominant grains (Table 6).

Table 2. Results of hydraulic measurements and calculations – point bar "a"

Point number	Water discharge Q [$m^3 \cdot s^{-1}$]	Froude number (Fr)	Shear velocity (V_*) [$cm \cdot s^{-1}$]	Reynolds number (Re)	Shear stress (τ) [$N \cdot m^{-2}$]	Flow resistance coefficient (f)
1a	0,34	0,20	1,95	7 571	0,38	0,1225
1b	0,34	0,30	8,59	488 851	7,39	0,4367
1c	0,34	0,20	4,11	16 886	1,69	0,3021
2a	0,34	0,30	8,39	10 6663	7,06	0,9053
2b	0,34	1,50	11,98	10 6555	14,36	0,0543
2c	0,34	0,08	1,50	15196	0,25	0,1466
3	0,34	0,44	4,48	38 862	2,01	0,0787
4	0,34	0,10	5,23	16 282	2,74	0,8835
1a	1,04	0,60	79,61	94 623	22,30	0,2823
1b	1,04	0,70	15,50	182 856	24,00	0,1414
1c	1,04	0,40	8,87	43 015	7,80	0,2885
2a	1,04	1,06	17,60	221 360	31,10	0,1015
2b	1,04	0,50	11,09	57 007	12,30	0,3239
2c	1,04	0,01	0,19	1 637	0,01	0,3136
3	1,04	0,80	31,77	211 493	100,93	0,0444
4	1,04	0,80	11,41	75 261	13,04	0,1555

Table 3. Results of hydraulic measurements and calculations: up-stream-of-obstruction bar "b"

Point number	Water discharge Q [$m^3 \cdot s^{-1}$]	Froude number *(Fr)*	Shear velocity (V_*) [$cm \cdot s^{-1}$]	Reynolds number (Re)	Shear stress (τ) [$N \cdot m^{-2}$]	Flow resistance coefficient (f)
1a	0,34	0,43	2,50	48 358	0,62	0,0205
1b	0,34	0,50	8,37	42 930	7,02	0,2255
1c	0,34	0,40	3,29	30 429	1,09	0,0505
2a	0,34	0,05	1,44	9 402	0,21	0,3765
2b	0,34	0,30	7,39	67 310	5,47	0,6687
2c	0,34	0,20	5,21	18 539	2,72	0,4038
4	0,34	0,04	0,56	2 341	0,04	0,2439
1a	1,04	0,53	6,49	53 784	4,21	0,0989
1b	1,04	0,60	17,27	214 292	29,80	0,1898
1c	1,04	0,60	19,29	136 091	37,20	0,3584
2a	1,04	0,08	2,69	9 056	0,87	0,9142
2b	1,04	0,90	13,38	195 812	17,90	0,0789
2c	1,04	0,54	5,63	159 169	3,10	0,0291
4	1,04	0,50	10,93	62 525	11,90	0,2891

Table 4,5. Characteristic grain size dimensions within the region of investigated point bars

alternate bar "b"										
	d_5	d_{10}	d_{16}	d_{25}	d_{50}	d_{60}	d_{75}	d_{84}	d_{90}	d_{95}
1A	77,6	95,4	115,7	146,2	230,8	264,6	315,4	345,9	366,2	383,1
1B	14,8	25	39,6	55,3	72,3	74,5	77,8	79,8	109,5	138,7
1C	6,1	10,8	15,3	20,6	36,2	42,9	52,1	56,4	59,2	76,1
2A	15	28,5	34,9	42,9	73,2	96,6	131,6	152,6	166,7	178,3
2B	21,1	32,6	43,8	54,4	72,8	78,6	133,3	171,7	197,3	218,7
2C	7,8	13,5	19,8	29,2	45,4	52	76,2	111,8	135,5	155,2
4	22,1	33	46,9	70,8	75,4	77,3	80,6	120	146,2	168,1
point bar „a"										
1A	18,5	34,1	54,4	70,4	77,5	82,2	111,4	128,9	140,6	150,3
1B	41,2	70,2	71,1	72,5	76,4	77,9	90,1	151,2	192	226
1C	20,3	32	43,2	56,5	74,4	77,6	107,8	141	163,1	181,6
2A	15,8	25,5	36,4	52,6	74,7	77,5	105,7	143,3	168,3	189,1
2B	14,3	17,9	21,7	27,4	45,2	51,3	56,8	61	98,1	129,1
2C	0,002	0,003	0,01	0,02	0,13	0,21	0,34	0,41	0,47	0,51
3	11,1	18,9	28,9	49,3	94,3	117,4	152,1	173	186,6	198,4
4	24,6	37,9	50,4	60,9	76,3	80	136,2	170	192,5	211,2

Table 6. Percentage of different grain shapes of bed load

Grain shape	Percentage
Spherical	4,43
Bladed	41
Disc-shaped and rod-like	54,57
100 %	

Discussion and Conclusions

The sediment deposited within the region of the investigated bars varied in diameter along the structure as well as across the cross-sections of the stream. In general, d_{50} was between 36.2 - 230.8 mm (a representative grain size), d_{16} between 15.3 - 125.7 mm, and d_{84} between 56.4 - 345.0 mm. The biggest differences in sediment diameters were observed along Alternate Bar B. Along Point Bar A, coarser sediment was deposited very close to the bar structure (Point 3) where the highest value of shear stress (τ=100.93 N/m^2) was noted, as well as the highest value for shear velocity (all calculated for Q=1.04 m^3/s flood discharge). Above Point 2B, the biggest shear stress value (τ=14.36 N/m^2) was found under mean annual flow conditions (Q=0.34 m^3/s). In this case, Fr was > 1, whereas at 2B under Q=1.04 m^3/s Fr was < 1 and shear stress and shear velocity were significantly smaller than under mean annual flow conditions. In the latter case, the water appeared to behave as it does above a typical riffle in a riffle-and-pool sequence when reversal velocity phenomena are observed. Point 2C is extraordinary in that it lies in the shadow of rock piled up on the streambed. A huge amount of fine sediment is deposited here, and Fr remains < 1, even under flood conditions.

Along Alternate Bar B, coarser sediment was deposited at Points 1A and 4, again close to the bar structure. The highest values of shear stress under flooding conditions were above Points 1B and 1C. The biggest shear stress value under annual discharge was again observed at Point

2B. The finest sediment was deposited along the left bank of the stream - opposite to the developed alternate bar structure. Point 2A was in the shadow of the bar, and shear stress and shear velocity were smallest here. The thawleg line, in the contest of hydraulics parameters, appeared to work like a vertical riffle within the reach. Along that line, Fr, shear stress and shear velocity were the highest under both run discharges.

The following conclusions were drawn from the analysis of hydraulic and sediment results within the regions of the investigated bars:

1. coarser sediment in a cross-section is deposited along the outer line of developed bars. Thus, bars are in a constant process of build-up. The distal part of the bars appears to be particularly stable.
2. fine sediment is deposited at the opposite bank to where the bars are formed. Shear stress, FR and shear velocity values are the smallest here.
3. fine sediment is deposited at spots within the stream reaches called "shadows", even under flood conditions. Such shadows may be found behind rocks and/or proximal part of bars.
4. in the case of Alternate Bars B, the highest values of shear stress, shear velocity and Froude number are found along the thawleg line lying approximately in the middle of the stream cross-sections within a bar region.
5. for Point Bar A, the highest Froude number is observed in the proximal part of the bar, at the entrance between the bar and the opposite bank, along the thawleg. Under annual flow conditions, it is possible to find places within a region of point bar that function similarly to riffle-and-pool sequences, where reversal velocity phenomena are observed.
6. with respect to the shapes of grains deposited in mountain, alluvial streams, the highest percentage are disc-shaped and rod-like. The next highest percentage are bladed. Spherical grains are a very small percentage less than 5%).

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Flood Wave Durations in the Danube River Basin in Croatia

Danko Biondić [1]
Darko Barbalić [2]

Croatian Waters, Ulica grada Vukovara 220, 10000 Zagreb, Croatia, dbiondic@voda.hr [1]
Croatian Waters, Ulica grada Vukovara 220, 10000 Zagreb, Croatia, darkob@voda.hr [2]

Abstract:
This paper presents envelope curves of main hydrological characteristics of flood wave durations over discharge thresholds ($Q_{5\%}$, $Q_{10\%}$, $Q_{15\%}$) in the Danube River basin in Croatia. Presented envelopes of average annual number of occurences, average duration and maximum observed duration of flood waves are calculated on the basis of 96 available sufficiently long homogenous time series of measured daily discharges on gauging stations at investigated area.

Key words:
Envelope curves, Flood wave durations, Discharge thresholds, The Danube River basin, Croatia

1 Introduction

The Danube River basin is after the Volga River basin the second biggest one in Europe with a size of about 817 000 km^2, with 18 riparian states and about 82 millions inhabitants.
In Croatia (Fig. 1) is covered approximately 34 000 km^2, roughly 60 % of the country's land area, where approximately 65 % of the total population live. Major Croatian rivers, such as the Danube, the Sava, the Drava, the Kupa, the Una and the Mura flow through this area. It is located in the Pannonian plain and its rims, with the water divide separating it from the Adriatic catchments running through the Dinaric karst.
A particular socio-economic significance of this area, not only for the Republic of Croatia but also for the greater region emphasizes importance of efficient flood control. Although certain flood control activities in this area date from 19th century, systematic development of flood control systems did not begin until catastrophic floods of the Sava, the Drava and their tributaries during the 1960s. Gradual development of flood control systems in the last four decades has significantly reduced potential damages, a fact proven by successful reduction of numerous recent floods. The most of constructed systems are partially completed, which results in a still existing significant risk of flooding in large areas. Further development of the flood and torrents control systems remains therefore one of the strategic tasks of Croatian water management.
Basic information for planning and designing of flood control systems are maximum discharges of required return periods, as well as maximum volumes and maximum durations of flood waves over certain discharge thresholds in adequate locations along the watercourses. Those can be estimated by applying stochastic methods for sufficiently long homogenous time series of measured discharges at gauged locations or by other methods at ungauged locations. Common hydrological practice also recognizes the use of regional

envelope curves for evaluation of reliability of such calculated values, as well as preliminary estimations of possible flood wave durations.

The aim of this paper is to present envelope curves of main hydrological characteristics of flood wave durations over discharge thresholds ($Q_{5\%}$, $Q_{10\%}$, $Q_{15\%}$) in the Danube River basin in Croatia, as a basis for further planning of flood control activities. These discharge thresholds are chosen as a representative with regard to overflowing from riverbeds to floodplains in the subject area. Similar investigations for maximum disharges in the Danube River basin in Croatia also were performed by Biondić & Barbalić, 2002 and 2003. [1,2]

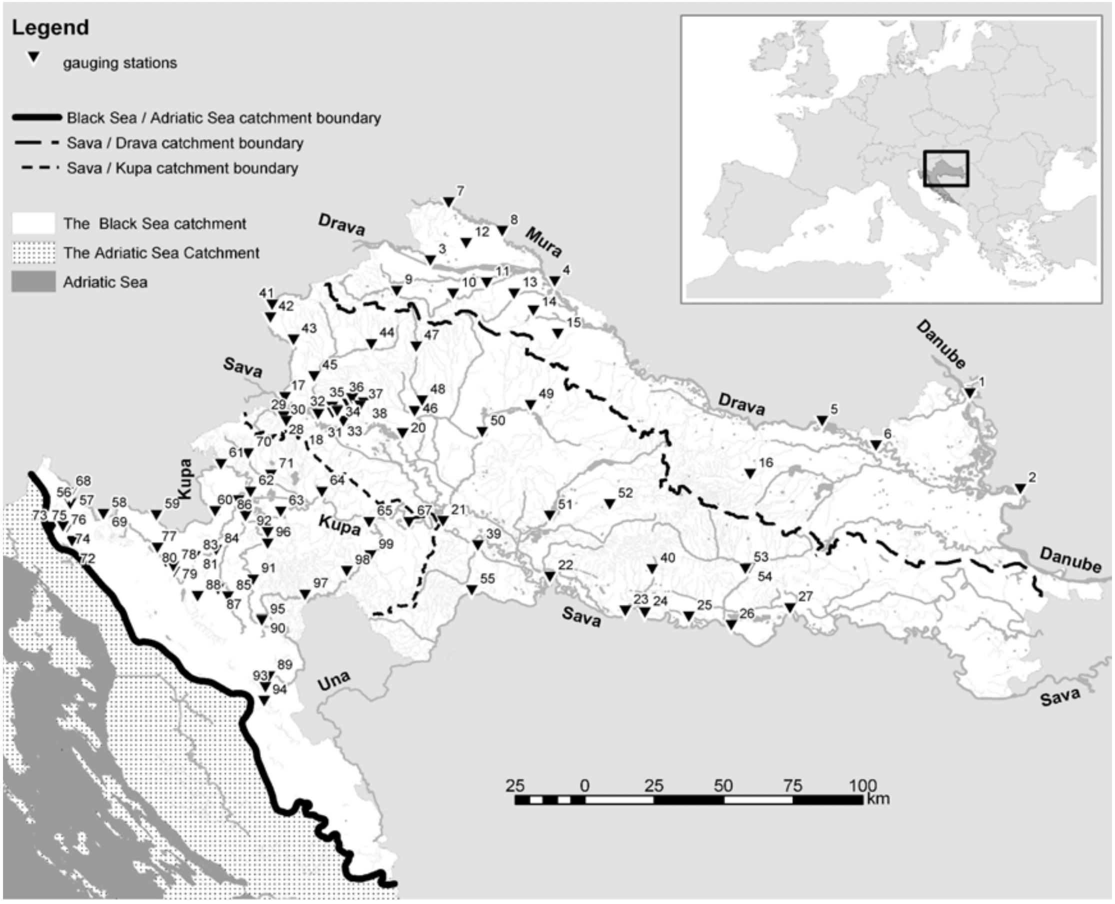

Fig. 1: Overview map of the locations of analysed gauging stations

2 Theoretical approach and investigation procedure

Envelope curves of main hydrological characteristics of flood wave durations over discharge thresholds (Fig. 2) - average annual numbers of occurences, average durations and maximum

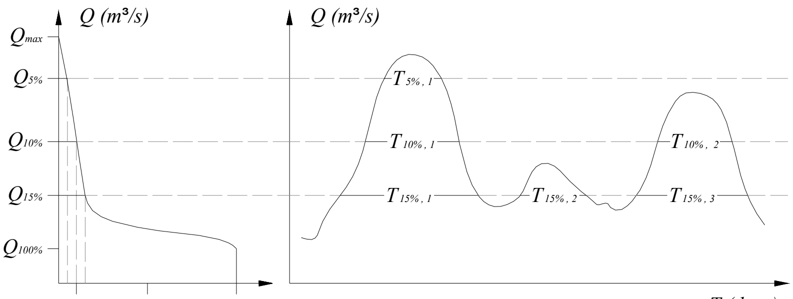

Fig. 2: Flood wave durations over discharge thresholds

observed durations are calculated on the basis of standard Creager's formula [3] as follows:

$$n = aA^{bA^{c}-1} \qquad (1)$$

$$T = aA^{bA^{c}-1} \qquad (2)$$

where:

n - average annual number of occurences of flood waves over discharge threshold
T - flood wave duration over discharge threshold (days)
A - catchment area (km^2)
a, b, c - regional parameters.

These curves for the Danube River basin in Croatia are defined on the basis of available time series of daily discharges which are stored in the Hydrological data base of Croatian Meteorological and Hydrological Service [4]. The previously described theoretical approach can be applied only when the series are sufficiently long, homogenous and when there are no significant trends. Analysis of time series homogeneity and trends were performed only for average annual discharge series at all gauging stations in the Danube river basin in Croatia which have observation periods longer or equal to 25 years (108 stations).
The analyses of homogeneity were performed by application of the Wilcoxon's test in such a way that the available series was split into two sub-series dependent on time of replacement of hydrometrical equipment at the stations (rods to limnigraphs) and dependent on time of constructions of main hydrotechnical structures with significant impacts on water regimes (reservoirs, main dykes and distribution structures). The presence of trend in time series of average annual discharges were tested by the Mann's test. On the basis of performed homogeneity and trends analysis, 99 time series of daily discharges were selected for further analysis (Table 1). Because of significant influence of large retentions in Sava flood control system on durations of flood waves, three gauging stations (No 23, 24, 25) downstream of the Lonjsko polje region were not used for calculations of envelope curves.
The last step was calculating of regional parameters "a", "b" and "c" of Creager's formula for each envelope curve by means of logaritmic and regresion analysis. For average annual number of occurences of flood waves over discharge thresholds, two different envelope curves were defined, one for catchments under and one for over 10.000 km^2.

No	RIVER	GAUGING STATION	CATCHMENT AREA (km^2)	OBSERVING PERIOD (NUMBER OF YEARS)	DISCHARGES (m^3/s)		AVERAGE ANNUAL NUMBER OF OCCURENCES OF FLOOD WAVES	AVERAGE DURATION OF FLOOD WAVES (days)	MAXIMUM OBSERVED DURATION OF FLOOD WAVE (days) (YEAR OF OCCURENCE)
					MEAN	$Q_{10\%}$			
1	Dunav	Batina	210250	1951-1989 (39)	2313	3680	2.7	13.3	118.8 (1965)
2	Dunav	Erdut	251593	1950-1989 (40)	2852	4370	2.2	16.3	110.5 (1965)
3	Drava	Varaždin	15616	1951-1981 (31)	341	588	7.4	4.9	91.1 (1951)
4	Drava	Botovo	31038	1961-1998 (38)	514	834	7.0	5.2	64.1 (1965)
5	Drava	Donji Miholjac	37142	1971-1998 (28)	514	831	5.6	6.5	61.1 (1979)
6	Drava	Belišće	38500	1962-1993 (31)	556	896	5.2	7.0	68.6 (1965)
7	Mura	Mursko Središće	10891	1926-1998 (67)	171	289	6.9	5.3	64.7 (1965)
8	Mura	Goričan	13148	1926-1978 (50)	157	265	7.0	5.2	65.8 (1965)
9	Bednja	Željeznica	308	1959-1998 (40)	4.04	9.27	11.6	3.2	28.3 (1986)
10	Bednja	Tuhovec	470	1958-1998 (36)	6.46	14.1	9.1	4.0	25.3 (1986)

11	Bednja	Ludbreg	547	1947-1998 (52)	7.29	16.9	8.1	4.5	23.9	(1986)
12	Trnava	Jendrašiček	148	1956-1998 (42)	0.405	0.726	5.3	7.0	77.0	(1974)
13	Gliboki Potok	Mlačine	84	1970-1998 (29)	0.731	1.49	10.6	3.6	42.9	(1979)
14	Koprivnica	Koprivnica	122	1951-1998 (46)	0.645	1.14	12.5	3.1	56.3	(1963)
15	Komarnica	Novigrad Podravski	48	1970-1998 (28)	0.242	0.578	12.5	2.8	45.3	(1986)
16	Vojlovica	Čačinci	150	1968-1998 (27)	1.77	3.92	8.0	4.3	48.6	(1970)
17	Sava	Jesenice	10750	1964-1995 (32)	276	542	9.8	3.7	25.0	(1970)
18	Sava	Podsused	12316	1949-1995 (47)	306	612	9.7	3.7	26.1	(1970)
19	Sava	Zagreb	12450	1926-1995 (70)	314	618	9.2	4.0	33.5	(1937)
20	Sava	Rugvica	12712	1926-1995 (67)	312	640	8.6	4.2	26.7	(1926)
21	Sava	Crnac	22852	1955-1992 (38)	529	1138	6.9	5.2	41.6	(1970)
22	Sava	Jasenovac	38953	1926-1991 (64)	784	1562	3.5	10.2	69.3	(1937)
23	Sava	Stara Gradiška	40100	1937-1991 (54)	788	1551	2.6	14.2	84.3	(1937)
24	Sava	Mačkovac	40838	1951-1990 (40)	823	1633	2.7	13.5	61.7	(1951)
25	Sava	Slavonski Kobaš	49031	1926-1976 (48)	1012	1951	2.3	15.4	83.1	(1937)
26	Sava	Slavonski Brod	50858	1945-1993 (49)	944	1882	3.0	12.3	125.2	(1970)
27	Sava	Županja	62891	1929-1998 (65)	1159	2260	2.8	12.8	124.8	(1970)
28	Bregana	Bregana Remont	88.5	1970-1998 (29)	1.38	2.34	10.5	3.6	24.0	(1984)
29	Lipovačka Gradna	Hamor	19.1	1948-1998 (50)	0.381	0.647	10.9	3.3	61.9	(1965)
30	Rudarska Gradna	Rudarska Draga	15.6	1957-1994 (38)	0.255	0.516	12.1	3.1	23.5	(1966)
31	Vrapčak	Gornje Vrapče	11.7	1970-1998 (29)	0.12	0.274	9.0	3.9	66.3	(1970)
32	Vrapčak	Zagreb	15	1961-1998 (38)	0.168	0.37	10.4	3.7	34.0	(1970)
33	Črnomerec	Fraterščica	7.37	1953-1984 (32)	0.09	0.192	9.7	3.8	47.5	(1970)
34	Kustošak	Kustošija	6.08	1956-1998 (32)	0.054	0.1	11.7	3.0	25.3	(1965)
35	Medveščak	Mihaljevac	14.2	1971-1998 (28)	0.135	0.267	7.1	5.2	31.1	(1993)
36	Bliznec	Markuševac	4.97	1969-1998 (30)	0.076	0.163	6.1	5.8	56.7	(1993)
37	Štefanovec	Dubrava	8.03	1961-1998 (34)	0.131	0.272	10.0	3.7	45.4	(1977)
38	Trnava	Granešina	29	1954-1984 (31)	0.338	0.701	9.6	3.6	37.8	(1970)
39	Sunja	Sunja	225	1965-1998 (31)	2.84	6.59	10.8	3.5	21.6	(1970)
40	Šumetlica	Cernik	33.5	1972-1998 (27)	0.291	0.599	7.1	5.1	58.8	(1980)
41	Sutla	Brezno	109	1946-1975 (30)	1.34	2.55	12.6	3.1	18.1	(1955)
42	Sutla	Miljana	263	1947-1976 (30)	4.19	8.44	14.2	2.8	18.5	(1970)
43	Sutla	Zelenjak	455	1958-1998 (41)	7.27	16.7	12.4	3.0	23.0	(1986)
44	Krapina	Zlatar Bistrica	228	1968-1993 (26)	2.11	4.1	11.1	3.3	22.3	(1993)
45	Krapina	Kupljenovo	1150	1964-1998 (35)	11.8	28	11.0	3.4	23.1	(1986)
46	Zelina	Božjakovina	186	1957-1998 (38)	1.64	3.82	11.7	3.3	21.7	(1993)
47	Lonja	Bisag	88.8	1952-1982 (31)	0.768	1.57	13.0	3.0	40.5	(1966)
48	Lonja	Lonjica	326	1972-1998 (27)	1.88	4.36	11.4	3.2	25.1	(1993)
49	Česma	Narta	881	1958-1998 (41)	5.43	15.2	8.1	4.5	36.5	(1993)
50	Česma	Čazma	2877	1963-1998 (36)	15.3	47.5	4.7	7.2	39.5	(1993)
51	Ilova	Veliko Vukovje	995	1945-1998 (52)	7.36	18.9	7.4	4.9	36.1	(1993)
52	Bijela	Badljevina	170	1969-1998 (30)	1.61	3.08	9.7	3.9	41.8	(1974)
53	Orljava	Pleternica	745	1970-1998 (29)	5.25	12.4	7.7	4.7	78.8	(1970)
54	Londža	Pleternica	483	1973-1998 (25)	1.87	4.47	6.1	6.1	31.6	(1981)
55	Una	Hrvatska Kostajnica	8876	1926-1978 (53)	234	471	7.6	4.9	26.8	(1947)
56	Kupa	Kupari	208	1951-1998 (48)	13.5	34.3	14.3	2.6	15.0	(1979)
57	Kupa	Hrvatsko	370	1957-1998 (40)	20.5	48.9	13.1	2.8	15.8	(1972)
58	Kupa	Petrina	528	1951-1992 (42)	26.6	58.4	13.3	2.8	14.1	(1972)
59	Kupa	Radenci	1304	1951-1992 (42)	53.7	127	13.2	2.8	15.4	(1984)
60	Kupa	Ladešić Draga	1590	1956-1998 (42)	58.4	142	13.0	2.8	14.1	(1974)
61	Kupa	Kamanje	2192	1957-1998 (40)	73.3	173	12.3	3.0	14.8	(1974)

62	Kupa	Brodarci	3405	1957-1982 (26)	118	264	12.7	2.9	11.5	(1981)
63	Kupa	Rečica	5923	1948-1982 (35)	171	375	10.4	3.5	17.5	(1974)
64	Kupa	Jamnička Kiselica	6805	1948-1978 (31)	180	411	9.6	3.7	17.4	(1950)
65	Kupa	Šišinec	7274	1950-1991 (41)	182	431	8.4	4.3	19.6	(1974)
66	Kupa	Farkašić	8902	1965-1992 (26)	196	465	7.7	4.7	32.8	(1974)
67	Kupa	Brest	9021	1926-1974 (49)	206	487	8.8	4.1	32.4	(1974)
68	Čabranka	Zamost	103	1950-1998 (49)	3.72	7.73	13.4	2.9	23.4	(1984)
69	Kupica	Brod na Kupi	291	1974-1998 (25)	12.8	32.3	11.6	3.1	13.4	(1976)
70	Kupčina	Strmac	125	1959-1998 (40)	2.18	4.28	10.2	3.7	30.1	(1959)
71	Kupčina	Lazina Brana	169	1973-1998 (26)	2.07	4.34	10.5	3.5	25.4	(1984)
72	Križ Potok	CP Križ	5.4	1963-1998 (36)	0.312	0.741	14.6	2.6	11.8	(1972)
73	Vela Voda	Crni Lug	3.7	1963-1998 (36)	0.208	0.481	13.8	2.7	17.0	(1964)
74	Bela Voda	Crni Lug	1.8	1963-1998 (36)	0.106	0.241	16.7	2.4	24.1	(1970)
75	Leska	Leska	0.25	1963-1998 (26)	0.015	0.03	16.0	2.5	11.0	(1986)
76	Klada	Klada	0.33	1963-1998 (26)	0.018	0.041	16.1	2.3	12.9	(1993)
77	Gornja Dobra	Luke	175	1947-1998 (51)	7.04	15.7	13.5	2.8	16.7	(1984)
78	Gornja Dobra	Ogulinski Hreljin	236	1948-1975 (28)	4.65	12.8	14.2	2.6	24.6	(1970)
79	Gornja Dobra	Turkovići	296	1963-1998 (33)	10	25.1	13.2	2.7	18.2	(1970)
80	Vitunjčica	Brestovac	34	1967-1998 (31)	3.28	8.47	14.2	2.6	10.8	(1972)
81	Donja Dobra	Trošmarija	821	1960-1998 (39)	27.9	63.2	9.4	3.9	40.5	(1964)
82	Donja Dobra	Donje Stative	1049	1960-1998 (39)	34.9	75.4	10.5	3.5	20.1	(1993)
83	Ribnjak	Lučanjek	50	1948-1975 (27)	3.01	6.81	10.2	3.4	17.8	(1974)
84	Globornica	Generalski Stol	32	1949-1975 (27)	1.02	2.87	11.8	3.2	13.2	(1950)
85	Mrežnica	Juzbašići	683	1947-1998 (46)	12.4	26.2	8.5	4.4	28.4	(1962)
86	Mrežnica	Mrzlo Polje	975	1960-1998 (39)	26.4	60	8.7	4.2	26.2	(1984)
87	Tounjčica	Ožanići	118	1948-1975 (28)	10.4	37	6.6	4.9	97.0	(1951)
88	Munjavčica	Josipdol	44.1	1948-1975 (27)	0.431	0.817	9.0	4.4	60.8	(1962)
89	Korana	Korana	232	1952-1986 (35)	3.09	6.56	7.1	5.2	38.6	(1970)
90	Korana	Vaganac	286	1949-1975 (24)	2.07	5.61	6.8	5.4	36.4	(1950)
91	Korana	Slunj	614	1964-1998 (29)	11.1	29.1	9.5	3.9	20.7	(1970)
92	Korana	Veljun	943	1949-1998 (44)	23	52.2	9.5	3.9	19.4	(1974)
93	Korana	Velemerić	1297	1946-1997 (46)	28.8	67.8	9.3	3.9	22.2	(1955)
94	Plitvička Jezera – Matica	Plitvički Ljeskovac	20.6	1952-1991 (37)	2.42	4.76	4.1	8.7	53.4	(1974)
95	Slunjčica	Slunj	220	1949-1983 (35)	9.48	19.9	9.0	4.2	30.9	(1974)
96	Radonja	Tušilović	224	1967-1998 (26)	3.32	7.93	8.1	4.6	21.8	(1970)
97	Glina	Maljevac	183	1953-1986 (34)	3.37	7.25	9.6	4.0	26.6	(1974)
98	Glina	Vranovina	889	1947-1998 (45)	14.1	30.8	9.1	4.0	23.2	(1955)
99	Glina	Glina	1145	1952-1998 (40)	18.3	42.6	8.7	4.2	22.6	(1955)

Table 1. *Basic characteristics of analysed gauging stations (data for flood waves over $Q_{10\%}$ threshold)*

3 Results

By applying the described methodology, Creager's envelope curves of average annual number of occurences, average durations and maximum observed durations of flood waves over discharge thresholds of $Q_{5\%}$, $Q_{10\%}$ and $Q_{15\%}$ were defined. The obtained results are shown in figures 3, 4, 5, 6, 7 and 8.

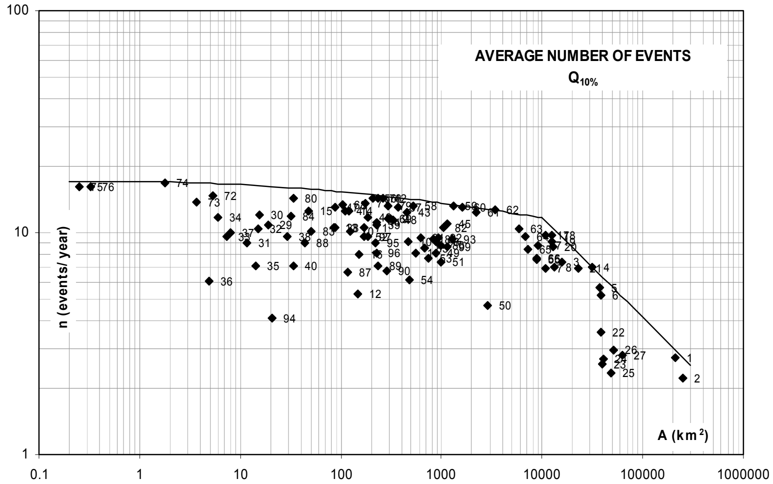

Fig. 3: Envelope curves of average annual number of occurences of flood waves over $Q_{10\%}$ discharge threshold in the Danube River basin in Croatia

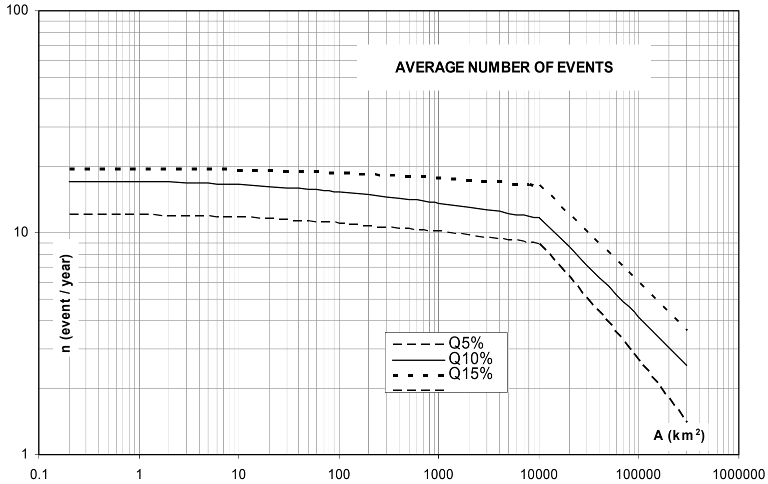

Fig. 4: Comparison of envelope curves of average annual number of occurences of flood waves over $Q_{5\%}$, $Q_{10\%}$ and $Q_{15\%}$ discharge thresholds in the Danube River basin in Croatia

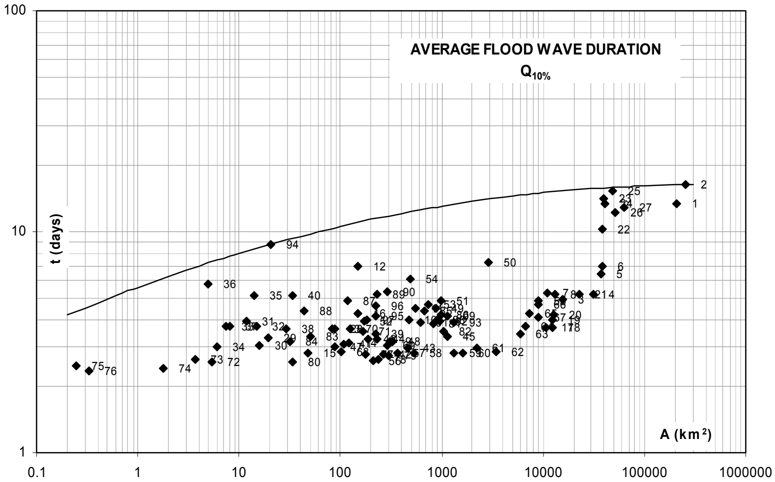

Fig. 5: Envelope curves of average flood wave duration over $Q_{10\%}$ discharge threshold in the Danube River basin in Croatia

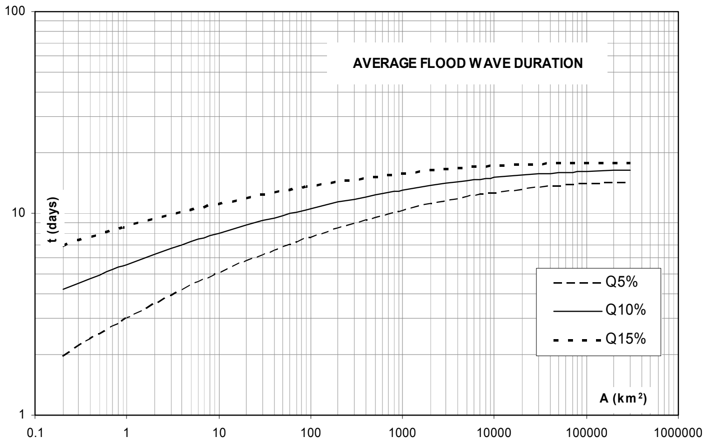

Fig. 6: Comparison of envelope curves of average flood wave duration over $Q_{5\%}$, $Q_{10\%}$ and $Q_{15\%}$ discharge thresholds in the Danube River basin in Croatia

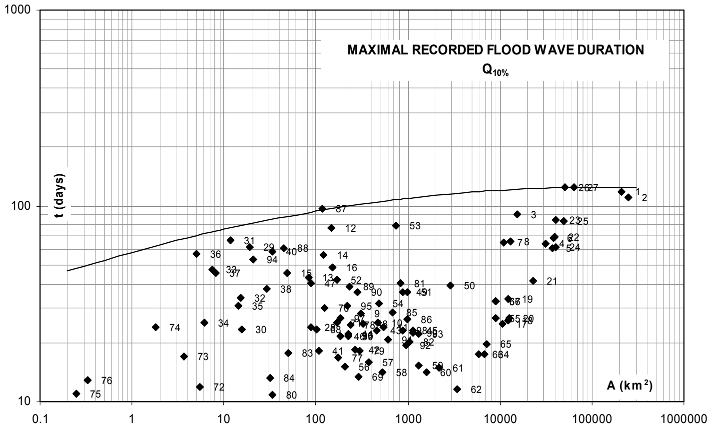

Fig. 7: Envelope curves of maximum observed flood wave duration over $Q_{10\%}$ discharge threshold in the Danube River basin in Croatia

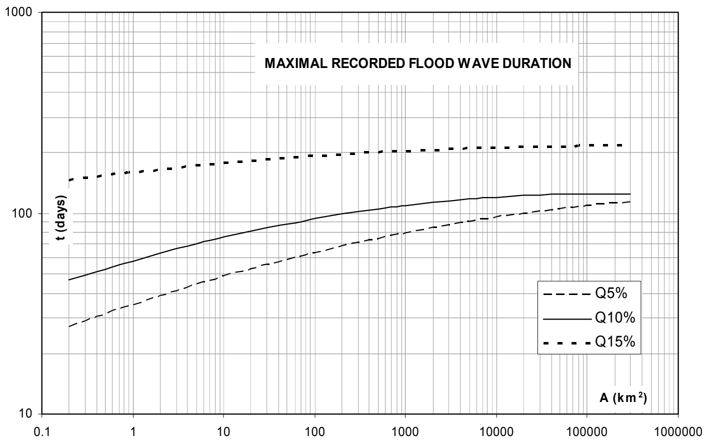

Fig. 8: Comparison of envelope curves of maximum observed flood wave duration over $Q_{5\%}$, $Q_{10\%}$ and $Q_{15\%}$ discharge thresholds in the Danube River basin in Croatia

The summary overview on calculated regional parameters “a”, “b” and “c” of Creager's formula is shown in table 2.

CALCULATED REGIONAL PARAMETERS “a”, “b” AND “c”				
FLOOD WAVE DURATION	**THRESHOLD**	**a**	**b**	**c**
AVERAGE	$Q_{5\%}$	3.0	1.250	-0.0085
	$Q_{10\%}$	5.6	1.170	-0.0060
	$Q_{15\%}$	8.6	1.126	-0.0050
MAXIMUM OBSERVED	$Q_{5\%}$	35.0	1.150	-0.0040
	$Q_{10\%}$	58.0	1.130	-0.0050
	$Q_{15\%}$	160.0	1.050	-0.0020

CALCULATED REGIONAL PARAMETERS “a”, “b” AND “c”				
AVERAGE NUMBER OF EVENTS				
THRESHOLD	**CATCHMENT AREA**	**a**	**b**	**c**
$Q_{5\%}$	A < 10 000 km^2	12.0	0.995	-0.0030
	A >10 000 km^2	125.0	0.940	-0.0300
$Q_{10\%}$	A <10 000 km^2	17.0	0.995	-0.0040
	A >10 000 km^2	570.0	0.600	-0.0040
$Q_{15\%}$	A <10 000 km^2	19.5	0.999	-0.0020
	A >10 000 km^2	596.3	0.650	-0.0070

Table 2. Summary overview on calculated regional parameters

4 Conclusion

The relations presented in this paper were calculated on the basis of 96 available sufficiently long homogenous time series of observed discharges on gauging stations in the Danube River basin in Croatia and can be used for preliminary estimation of possible flood wave durations at this region, as well as for evaluation of reliability of such values calculated by application of more complex hydrological methods.
Calculated envelope curves for average duration and maximum observed duration are not reliable for catchments under 5 km^2, because of unsufficient input data (only threee gauging stations).
Because of climate, relief and geolgical diversity of investigated area, further studies of durations of flood waves in the Danube River basin in Croatia should be directed to sub-regionalisation. Also, further regional studies of flood characteristics in the Danube catchment area in Croatia should be aimed to analysis of flood wave volumes.

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Static Puncture Tests for Geotextiles and Geotextile-Related Products

Adam F. Bolt, Angelika Duszyńska, Monika Piotrowska[1]

Abstract: The knowledge of mechanical strength reduction for geotextiles and geotextile-related products connected with their mechanical damage during installation and usage is very important to control their behaviour in soil. In this paper the two tests of static puncture resistance are discussed: the standard CBR test according to EN ISO 12236 and the new test methods the "pyramid test" developed by CEN TC 189 (draft PrEN 189066). Procedures of the performed tests are similar and differences between them are connected with: kind of loading, method of support, filling of the cylinder and materials used in the tests.

1. Introduction

The rising numbers of applications geotextiles and related products contributed to elaborating methods of testing and classification parameters of geotextiles, and standards to construct with using materials of geotextiles.

Reduction of mechanical strength material is often connected with mechanical damage caused by direct contact between soil backfill and geosynthetics under load.

The searching of optimal methods for the determination of puncture resistance of geosynthetics leads to the new proposition of tests. The most prevalent tests of mechanical durability are dynamic puncture test (cone drop method) and the static puncture test (according to CBR method). The work items of CEN TC 189 included the development of the new methods of static puncture test - the "pyramid test" (WI 00189066) to quantify the efficiency of geosynthetics in protecting different sub-bases.

In this paper the following tests of static puncture resistance of geotextiles and geotextile-related products are discussed:

- the standard test CBR according to EN ISO 12236,
- the pyramid test method presented in WI 189066.

2. Cone drop test

The dynamic perforation test, called worldwide "cone drop" test is standardized in EN 918. This test simulates dropping sharp stones onto geotextile surface.

EN 918 specified the determination of the resistance of geotextiles and geotextile-related products to the penetration by a steel cone, of 45° tip angle, dropped from a fixed height (500 mm) onto the center of 150 mm diameter

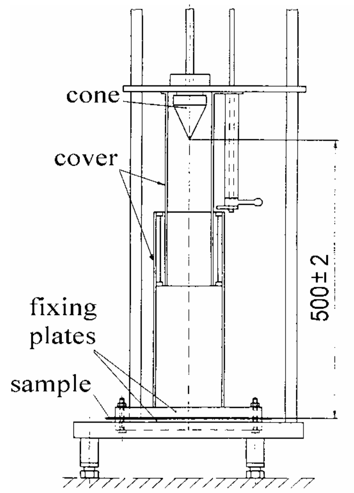

Fig. 1. An arrangement for cone drop test.

[1] Faculty of Hydro and Environmental Engineering, Gdańsk University of Technology, ul. Narutowicza 11/12, 80-952 Gdańsk, Poland

specimen. The specimen is secured, without any support(see figure 1), between the clamping rings of the clamping system. The degree of penetration is measured by insertion of narrow-angle graduated cone into the hole. The test result is the mean hole diameter in mm and the coefficient of variation.

3. CBR test

The CBR test simulates big stones pressed into geosynthetic laying on a soft sub-base.
This test according to PN-EN ISO 12236 is standardized worldwide. It determines the puncture resistance of geotextiles and geotextile-related products by pushing a 50 mm diameter flat-ended plunger through the center of 150 mm diameter specimen until failure is recorded. The specimen is secured between the clamping rings of the clamping system, without any support (see figure 2). The test result is the mean of the push-through force, the mean of the push-through displacement and the coefficient of variation.

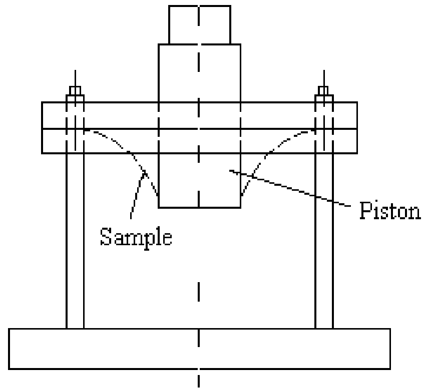

Fig. 2: An arrangement for CBR test

4. Pyramid test

The pyramid puncture resistance test is developed by CEN TC 189 in framework of WI 00189066. This test simulates a geosynthetic's efficiency in protecting a geomembrane or other contact surface against sharp rigid elements under short term loading. WI 00189066 specified two methods to determine the pyramid puncture resistance of geosynthetic on rigid and soft support (annex A).

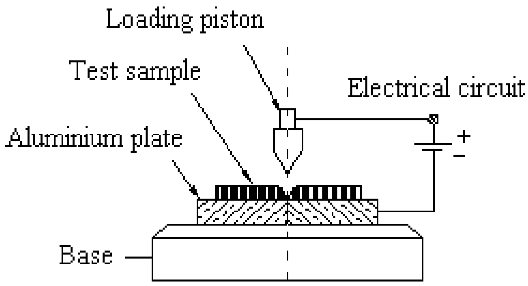

Fig. 3. Pyramid test configuration with rigid support

4.1. Rigid support

A test specimen lies flat on an aluminium plate (see figure 3) supported by a steel base, and a force is extorted the center of the test specimen by a steel pyramid until perforation occurs. The puncture load is registered by electrical circuit between the loading piston and the aluminium plate. The recorded push-trough load (the average of test values) is considered to be representative for the protection efficiency of specimen.

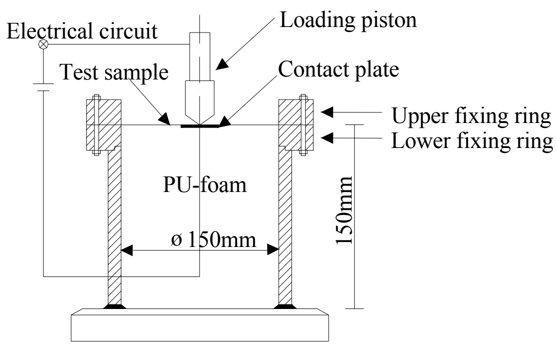

Fig. 4. Pyramid test configuration with soft support

4.2. Soft support

The static puncture test with pyramid-ended plunder and soft support is presented in Annex A of WI 00189066. In this method the test specimen is secured between the clamping rings (see figure 4). A resilient PU-foam (compressible foam which regains its original thickness after release of the load) is used as a support. This foam is situated inside a steel CBR-cylinder compression based. The flexible contact plate is placed on PU-foam under geotextile sample to capable closing of signalling electrical circuit. The test procedure is similar to the test procedure on rigid support.

5. Performed tests

5.1. Materials

Two types of geotextiles and related products were used in static puncture tests: four different polyethylene geomeshes Zloty Stok in CBR tests and two polypropylene non-woven geotextiles in pyramid tests. The materials used in the tests and their mass per unit area (determined according to EN 965) are presented in Table 1.

Table 1. Tested products.

Products	**Mass per unit area** [g/m^2]
geomesh 305 HDPE	750
geomesh 231 HDPE	570
geomesh 303 LDPE	725
geomesh 201 LDPE	450
non-woven geotextile Polyfelt TS80	380
non-woven geotextile Polyfelt PP P009	810

5.2. Testing machine

The Zwick 1476 testing machine of accuracy class 1 was used. The press is capable of providing constant test speed, recording force and displacement and providing an autographic read-out of force and displacement. This machine was equipped with a CBR- cylinder compression base and two types of loading piston - a steel plunger with a diameter of 50 mm for CBR tests and a loading piston with a diameter of 25 mm and with a polish and hardened pyramid-shaped apex for “pyramid tests”.

5.3. CBR test

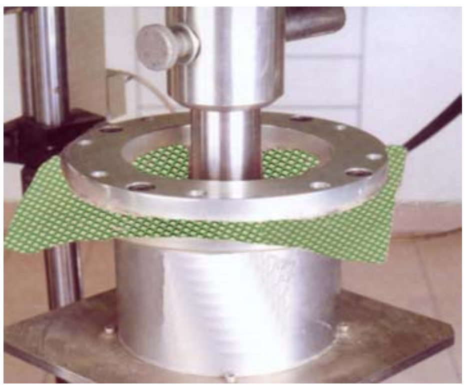
Fig. 5. CBR test according to EN ISO 12236

Four different geomeshes from and high density and low density polyethylene, with small and large meshes were used in standard CBR tests. The tests were performed according to procedure described in clause 3 and EN ISO 12236.
The figure 5 presented geomesh Zloty Stok 201 LDPE during CBR test.
Tests results for all of the used materials (the puncture force F, corresponding piston displacement L and coefficients of variation SD) are collected in table 2. Figures from 6 to 9 presented load - displacement curves for geomeshes. Comparison of these graphs indicates that samples of 201 LDPE geomesh are characterized by the greatest initially elongation and the lowest mechanical parameters. The 303 geomesh – also from LDPE but with larger meshes, carried out higher

loads but has a worse coefficient of variation. It can testify that the specimen of geomesh 303 LDPE used in tests was non-homogeneous material.
Performed tests confirmed geomeshes from high density polyethylene (231 and 305) are characterized by higher mechanical parameters then materials produced from low density polyethylene.

Table 2. Test results.

Product	F [N]	SD_F [-]	L [mm]	SD_L [-]
non-woven geotextile Polyfelt TS80	573	0,090	26,6	1,06
non-woven geotextile Polyfelt PP P009	2 579	0,110	43,4	1,25
geomesh 201 LDPE	314	0,014	49,2	3,32
geomesh 303 LDPE	490	0,116	54,2	9,97
geomesh 231 HDPE	565	0,060	35,1	8,02
geomesh 305 HDPE	826	0,032	28,9	0,88

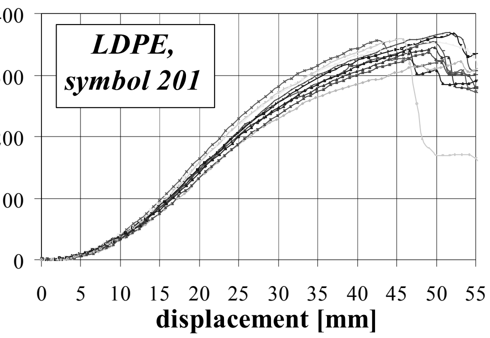

Fig. 6. Load versus displacement for geomesh LDPE 201.

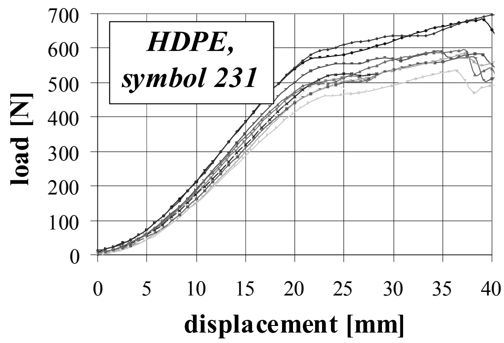

Fig. 7. Load versus displacement for geomesh HDPE 231.

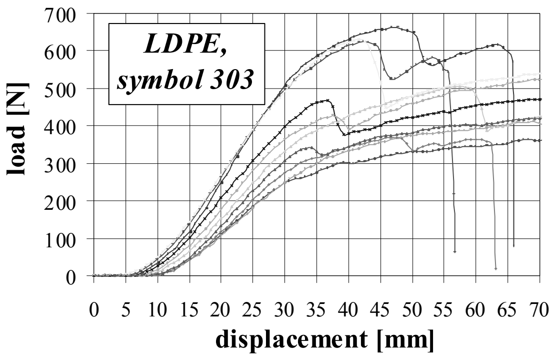

Fig. 8. Load versus displacement for geomesh LDPE 303.

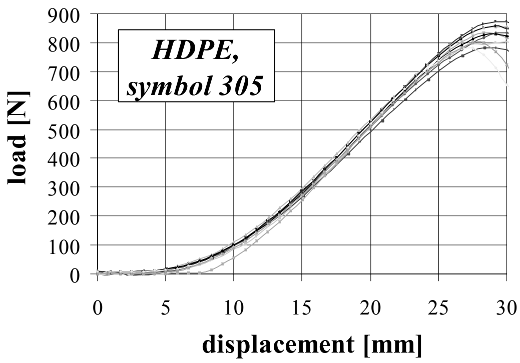

Fig. 9. Load versus displacement for geomesh HDPE 305.

5.4. Pyramid test - soft support

Two polypropylene non-woven geotextiles Polyfelt TS80 and PP P009 were used in the pyramid test with soft support. The tests were performed according to procedure described in clause 4 and Annex A of WI 00189066. The figure 10 presented the test preparation - contact plate is placed on PU-foam under geotextile sample to capable closing of electrical circuit. At

the moment of sample puncturing the loading piston is getting in the contact with this flexible contact plate and the control light is burning .
The figure 11 presents a fixed specimen of non-woven geotextile during the pyramid test with soft support.

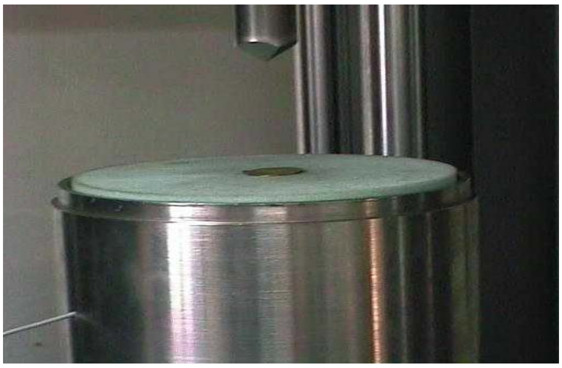

Fig. 10. Flexible contact plate on PU-foam.

Fig. 11. WI 001890066 - test method on a soft support.

The tests results are collected in table 2. Figures 12 and 13 present load - displacement curves for non-woven geotextiles.
Based on performed analyses of pyramid test results it was found that a switch to stop the testing machine automatically after the puncture occurrence is strongly recommended because as the final value puncture force is measured, when a electrical circuit is closed, there is a severe influence if the testing machine is stopped manually on a light signal created by the closed circuit. It leads to higher forces and is a reason for missing a clear peak for majority of curves presented for Polyfelt TS80 in figure 12. For Polyfelt PP P09, which is characterized by higher value average push-through load, the puncture of material is better visible (see figure 13).

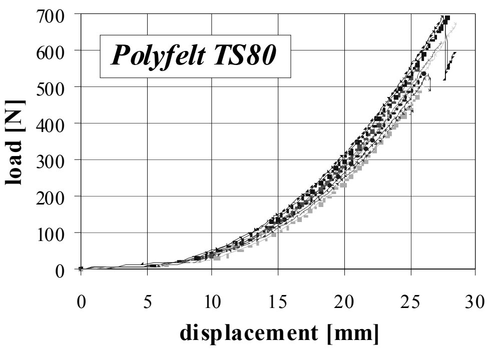

Fig. 12. Load versus displacement for non-woven geotextile Polyfelt TS80

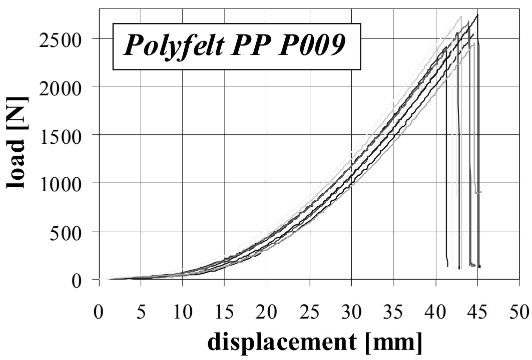

Fig. 13. Load versus displacement for non-woven geotextile Polyfelt PP P009

6. Discussion of results

Procedures of the performed tests presented above (CBR according to EN ISO 12236 and pyramid test on soft support according to WI 189066 annex A) are similar and differences between them are connected with:

a) Kind of loading piston - in CBR tests a steel plunger with a diameter of 50 mm is used, whereas in WI 189066 the geosynthetic is punctured with cylinder loading piston (25 mm diameter) with a polish and hardened pyramid-shaped apex.
b) Method of support :
- without support (CBR test),
- rigid support (i.e. aluminum plate in pyramid test),
- soft flexible support (i.e. foam in alternative pyramid test).

c) Filling of the cylinder compression base – in CBR test geotextile is put on an empty cylinder (the test in air) and in pyramid test with soft support (Annex A WI00189066) the cylinder is filled by resilient polyurethane foam (with defined hardness and density).
d) Event caused the end of the test – CBR test is ended when maximum push-through force was recorded by testing machine whereas the pyramid test is ended when the puncture load was registered by electrical circuit between the loading piston and the aluminium plate (the control light is burning).
e) Materials used – four different polyethylene geomeshes in the CBR tests and two polypropylene non-woven geotextiles in the pyramid tests. Using a geomesh in pyramid test isn't advisable because of holes dimensions in used geomeshes in comparison with the size of the pyramid apex.

7. Conclusions

Laboratory tests of puncture resistance of geosynthetics provided a lot of information about the behaviour of these products in soil. The knowledge of mechanical strength reduction for geotextiles and geotextile-related products connected with their mechanical damage during installation and usage is very important to control their behaviour in soil.
The choice of suitable puncture test method is mainly dependent on material structure and suitable simulating determined behaviour in geoengineering. Each test has its value for a specific situation.

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Importance of Protected areas in Integrated Water Management

Gorana Ćosić-Flajsig, ,[a]
Davor Malus[b]
Mladen Petričec[a]

[a]Croatian Waters, Water Management Institute, Ul. grada Vukovara 220, 10 000 Zagreb, Croatia, gcosic@voda.hr and petricec@voda.hr
[b]Faculty of Civil Engineering, Kačićeva 26, 10000 Zagreb, Croatia, malus@,grad.hr

Abstract
The need for additional, stricter and more comprehensive water protection measures than those normally conducted on the territory of the Republic of Croatia is evident in protected areas. In some parts of river basins/water districts, defined as protected areas on the basis of the National Water protection plan and EU Water Framework Directive[1], the need for special water protection measures is evident. Water protection is conducted while duly considering other national regulations as well, in the fields of health care, agriculture, nature protection, environmental protection, etc., with the purpose of protection and sustainable management of surface inland water and/or protection of habitats and species directly dependent on water. The Water Management Master Plan includes the Map and registry of protected areas. Integrated water management in protected areas is the basic condition for sustainable water management in such areas. Therefore, water protection measures in protected areas must be planned within integrated water management, and water management planning documents harmonized with other planning documents.
Protected areas are divided in two major groups:
(i) protected areas designated for human use or influenced by human activities; and
(ii) protected areas, which include water ecosystems and water-related ecosystems based on legislation and international conventions on nature protection.
In line with the above, in order to avoid possible conflicts and achieve economic and social goals, it is necessary to harmonize and fulfill different interests in such areas. In the coming period, water protection in protected areas will be priority activity of water management.

1. Introduction

Water use, in general terms, exerts a pressure on quantity and quality of natural resources, water resources in particular. Consequences of water use are manifested as changes and disruptions of water status, as well as of water ecosystems and water-related ecosystems. Humankind has sought an answer to the process of continuous deterioration of water status in planning of water resources use and sustainable water management. The 1992 Rio de Janeiro Conference and passing of Agenda 21 marked the beginning of a new approach to water resources management. Realization that the restriction on use of water resources results in the restriction of economic development has necessarily led to considerations of water management means and methods. Water pollution control, and water protection in protected areas in particular have priority in water resources planning and management. Regarding the significance of protected areas for water users, as well as numerous users themselves, quality water management is impossible without an integrated water management approach. In order to achieve an integrated water management approach, it is indispensable to have: a cooperation with the local community and population to determine local goals and water management plan, and water quality in particular, a state level care about achievement of water pollution control goals, and by extension of sustainable development.

2. Water pollution control in the Republic of Croatia

The basis for long-term water pollution control program in Croatia is founded on general principles followed by measures and procedures to meet water pollution control goals, and it is realized through State Water Protection Control Plan[2]. Water pollution control encompasses all water systems on land: surface waters including both running and stationary ones (natural and artificial), underground waters, as well as coastal sea water that wastewater from mainland and islands is discharged into, directly through wastewater outfalls, public sewerage systems and industry, or indirectly by means of mainland watercourses.
Water pollution control, water condition maintenance and water deterioration reversal are carried out by following water protection measures:

- administrative measures: water quality maintenance through measures for water pollution prevention and decrease, as well as implementation measures, making of plan documentation, bringing of the regulative acts for wastewater discharge, making of the cadastral map of water pollutants, and establishment of information system about water quality condition and efficiency of applied measures.
- measures for water quality maintenance are – prohibition of construction in areas where water quality of wells and underground waters for public water supply is in jeopardy, prohibition or restriction of construction in specially protected areas and valuable water ecosystems, restriction and prohibition of dangerous substances discharge from the Dangerous Substances Act.
- measures for pollution prevention and decrease are - planning, restoration and construction of public sewer systems, wastewater treatment plants, decrease of wastewater from technological processes, substitution of existing technologies with cleaner ones, introducing of the program of measures to decrease water pollution by agro technical means, restoration of erosion areas, construction and restoration of dump sites, particularly those with possible impact on drinking water, restoration of mainland sources of sea water pollution that limit sea water use.
- measures with extraordinary pollutions in cases of unfavorable hydrological and meteorological conditions, as well as accidental pollutions in case of unexpected spillover of dangerous and other substances that may cause deterioration of planned water class or water pollution.

Discharge control for dangerous substances into the water should be performed in accordance with «combined approach» defined by the emission control based on the best existing technology or corresponding boundary emission value, or on best ecological experience with nonpoint pollution sources, and in cases when water quality goals and standards demand conditions stricter than already mentioned, stricter discharge controls shall be determined. This approach has not been entirely applied in the Republic of Croatia and its full application is necessary in next period.
Systematic monitoring of surface water quality, and in part of underground waters, too, is conducted within the water management. Monitoring is, as a rule, defined by measuring stations located after the pollutant source, so the mentioned monitoring can't offer a complete picture about water condition in Croatia. In the near future more attention will be given to the establishment of various means to systematically monitor water condition with various goals defined.
Wastewater quality monitoring system may be assessed, as inadequate, so expert assessments should be used during analyses. In the process of wastewater condition assessment, data are

used from database for water protection fee calculation, from water management acts, as well as data and documentation from «Croatian Waters» services. Efforts to overcome noticed shortcomings in wastewater quality and quantity monitoring, and to produce a register map of pollutants will intensify in the near future.

Planning and assessment of measures conducted to prevent and decrease water pollution is currently based on water condition assessment (on the strength of existing water quality and quantity monitoring), defining of critical stretches with regard to water quality, and analysis of causes in water pollution pressures or in up to now inadequately or inefficiently conducted water protection measures.

Basic plan documents on the state and catchment level are Water Management Basis of the Republic of Croatia (which is completed) and Water Management Bases or Plans of water and catchment areas (planned). Those plans envision water protection within integrated water management on state level, i.e. water or catchment area level. To define and realize those plans, among other things, collaboration between water management and physical planning is required. Basis for water management planning of water protection on local level are County Water Protection Control Plans, which plan for water protection measures on county level, as well as Sewage and waste water treatment Acts. To define and realize mentioned plans, coordination between water management and physical planning is required on local level.

3. Protected areas in Croatia

Water protection measures are conducted all over entire area of Croatia, but in certain, protected areas, there is a need for additional, stricter and more complex water protection measures. Those stricter water protection conditions do not relate only to stricter control of discharge of dangerous substances in stated areas and heightened monitoring of water condition, but also to a restricted water use, as well as to a failure or limited conducting of measures of watercourse regulation and protection from harmful water effects. On the basis of State Water Protection Control Plan[2] which defines protected areas as «particularly protected areas» it is therefore necessary to plan for additional water protection measures within the water management framework, with respect to other national regulations from areas of health, agriculture, natural protection, environmental protection and others. The goal is a protection and sustainable management of surface mainland waters and/or protection of habitat and species directly dependent on water. Protection of protected areas is currently defined on the basis of various Croatian laws and regulations, and its implementation falls within authority of different ministries and state institutions. It is therefore necessary to bring additional water protection measures in protected areas into accord, among others by plan documents related to them.

Within Water Management Master Plan[3], a Map and Registry of Protected Areas was made. It was made according to the contents of EU's Water Framework Directive[1], based on Croatian legislation[4] and accepted international conventions.

Protected areas comprise two basic groups:

(i) protected water areas intended for human use or under influence of human activities
(ii) protected areas containing water ecosystems and ecosystems dependent on water, based on legislation and international conventions related to protection of nature.

Water protection in protected areas is based on obtaining the water quality (I and II kind of water) suitable for various users. With regard to bringing into accord various interests of space and water users, protection of protected areas in accordance with protection goals and water use standards set, management plans for protected areas will be made, with crucial element of water management within areas in question.

According to water resources use, protected areas comprise:

1. Protection of potable water areas
2. Protection of areas intended for fish and mussel harvests
3. Protection of areas intended for swimming
4. Protected areas sensitive to nutrients, including areas marked as vulnerable zones with regard to protection of water from pollution caused by nitrates from agricultural activities, and sensitive to discharge of wastewater from public sewer systems.
5. Protected areas sensitive to discharge of wastewater from public sewer systems.

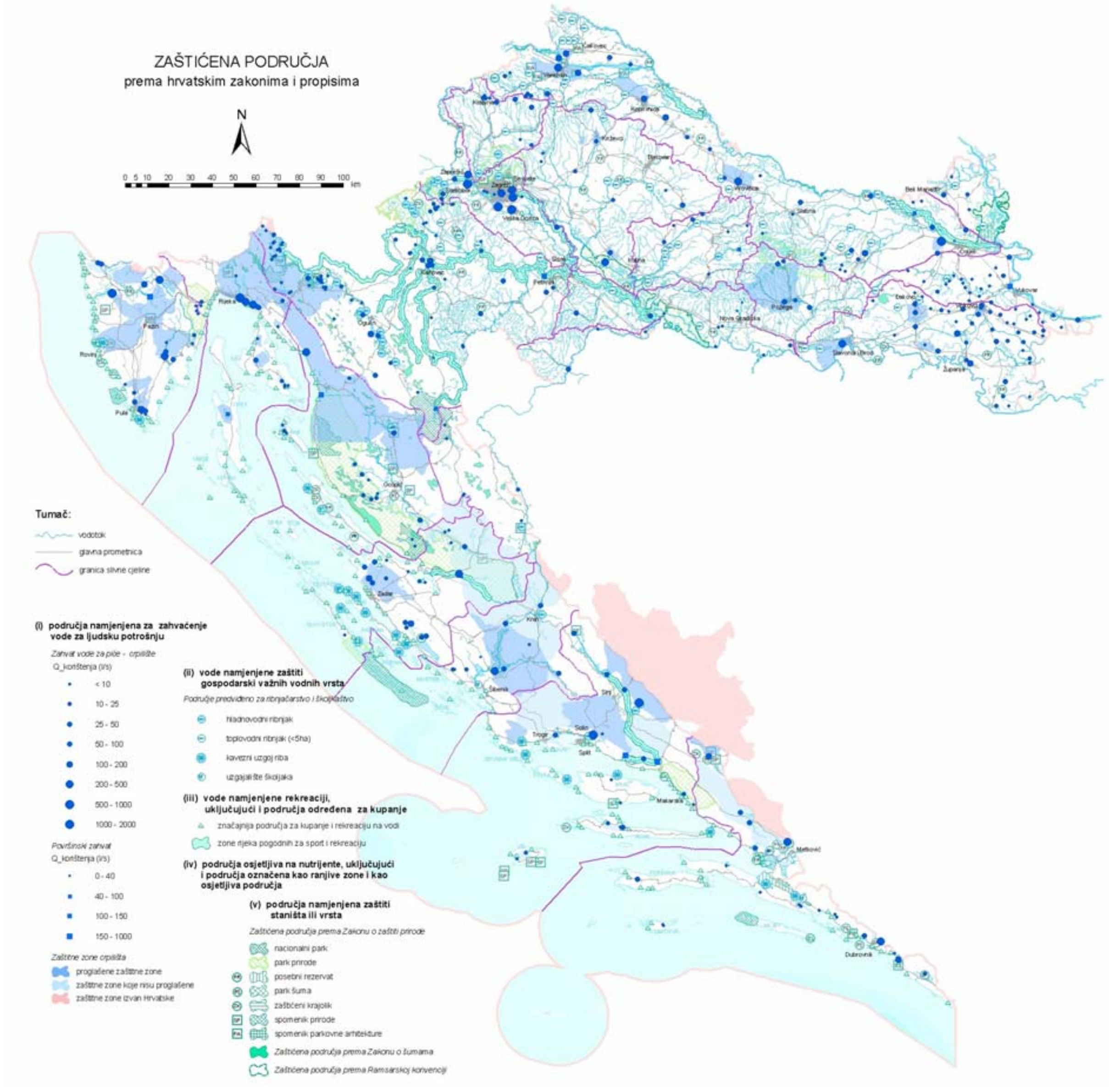

Fig.1 Protected areas in Croatia

Currently there are no legislative regulations in Croatia according to which either protected areas vulnerable to nutrients or protected areas sensitive to discharge of wastewater from public sewer systems can be defined, so those areas have not been mapped yet. It will be the subject of further work.

Protected areas according to laws and conventions on nature protection:

1. Protected areas according to the Law on Nature Protection
2. Protected areas according to the Law on Forests
3. Protected areas according to Ramsar Convention
4. Protected areas intended for protection of habitat and species where maintenance or improvement of existing water condition is a factor of protection.

Protected areas intended for protection of habitat and species where maintenance or improvement of existing water condition is a factor of protection have not been mapped, because currently there are no legislative regulations in Croatia according to which they may be defined. So it will be the subject of further work in cooperation with the Ministry of Environmental Protection And Physical Planning.

Registry and map of protected areas is focused on waters demanding special protection. Particular attention within protected areas is given to potable water areas, where the protection is achieved on the basis of physical-plan instruments – sanitary protection zones of wells for public water supply system. Sanitary protection zones (declared and undeclared), due to specific nature of karst, cover some 18 % of area of Croatia, whereas entire area of protection regions amounts to some 28 % of mainland territory of Croatia.

Integrated water management within protection areas is the fundamental precondition for sustainable water management in those areas. In accordance with what has already been stated, it is necessary to bring into accord and care about all interests in such areas, in order to avoid possible conflicts and achieve economical and social interests. Future period in water management has as its priority the water protection in protected areas.

4. Demands of European water policy

On international level, water protection is regulated by the EU's Water Framework Directive (2000)[1]. In accordance with this, it is necessary to create a registry and map of protected areas according to division on water bodies. Right now, there is neither a guide how to create the map and registry nor the format for its form, so it is left to national characteristics. In addition to maps, the registry shall contain descriptions of legislative regulations due to which the protected areas have been defined, as well as the possibility to obtain information on protected areas within water body, and it shall be a constituent part of water management within river catchment/basin.

In addition to priorities that water protection measures in protected areas represent, European water policy poses demands for additional water monitoring in protected areas[5]. Besides water monitoring obligations for purpose of keeping an eye on condition of all waters mandated by the Water Framework Directive (surveillance, operational, investigative), monitoring frequency in protected areas must be complemented so that following demands for water protection are met:

- potable water intake location
- surface waters that yield on average more than 100 m^3 daily will be designated as monitoring points and subjected to additional monitoring. Monitoring of all priority substances will be carried out, as well as other substances discharged into water in significant quantities, which may affect water condition. Monitoring frequency is in accordance with number of users.
- areas of protected habitats and species
- Waters that make up those areas will be included in operational monitoring programs where the risk of not meeting ecological goals is determined on the basis of impact assessment and surveillance monitoring. Monitoring will be conducted until those areas meet the conditions from wastewater discharge permit, which established the goals to achieve.

Likewise, with water body monitoring defined, programs for protected areas are amended with specifications contained in the Community regulations[6] according to which the specific protection areas are established.
Based on the pressure and impact assessment, as well as water condition assessment, surveillance and operational programs are established for river catchment management plan. In particular cases, investigative monitoring programs are established, too. It is necessary to closely follow condition indicators of every relevant water quality element. The plan also gives the assessment of reliability degree and precision of results obtained by monitoring programs.
Special management plans for protected areas are anticipated for protected areas. Within them, a crucial role belongs to the water management plan for protected area, which is the constituent part of water body management plan. Efficiency and economical sustainability of the protected area management plan is possible only through an «open planning» approach, with an inclusion of all interested parties, i.e. the public.

5. Conclusions

Based on State Water Protection Control Plan and EU's Water Framework Directive, protected areas are defined within the Water Management Basis of Croatia, and Map And Registry of Protected Areas has been made. Protection of protected areas is currently defined on the strength of various Croatian regulations and laws, and the implementation is within the authority of various ministries and state institutions. It is therefore necessary to bring additional water protection measures in protected areas into accord among all users, among other things by plan documentation related to them, and by the management plan for those areas. Regarding the importance of protected areas for water users, as well as the number of users, quality water management is not possible without an integrated approach, so water management measures should be planned within integrated water management, and water management plan documents brought into accord with other plan documents. Integrated water management within protected areas is a fundamental precondition for sustainable water management in those areas.
Interests of all interested parties are brought into accord through management plans for protected areas, so that possible conflicts are avoided and economical and social interest achieved. European water policy poses demands for additional water monitoring in protected areas, so a greater attention will be given to it in Croatia, too. First in terms of rationalization of the existing monitoring, and then in terms of designing a tailor-made monitoring, in accordance with a water management plan for the protected area. Water protection in protected areas of Croatia in the future period is of priority in water management.

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Life Cycle Assessment Based Tools for the Development of Integrated Waste Management Strategies fer Cities and Regions with Rapid Growing Economies

Assoc. Prof. O. Čermák, PhD,, Assoc. Prof. M. Čermákova, PhD .
Dipl. Ing. K. Jankovičová, Dipl. Ing. Ľ. Horanová, PhD, RNDr. I. Škultétyová PhD

Slovak University of Technology, Department of Sanitary Engineering, Radlinskeho 11, 813 68 Bratislava, Slovakia,
cermak@svf.stuba.sk cermakova@svf.stuba.sk jankova@svf.stuba.sk horanova@svf.stuba.sk iskultet@svf.stuba.sk

Abstract

The presented paper is about project, which aims at the development of tools to support waste management planning and optimisation in European cities, in particular cities from EU Accession Countries, has been initiated within the European Commission's Fifth Framework Programme: Energy, Environment and Sustainable Development Work Programme - Key action: City of tomorrow and Cultural Heritage. The Consortium of this project consists of twelve Partners, from the European (Germany, Austria, Luxemburg, Spain, Greece) States well as EU Accession States (Poland, Lithuania, Slovakia), with TU Darmstadt as a coordinator.
The projected outcome of the project are two decision supporting models for waste management: planning:waste prognostic model and assessment model for environmental, economic and social performance of integrated waste management strategies
The Partners from the selected cities (Tarragona and Barcelona, Xanthi, Wroclaw, Kaunas, Bratislava Nitra) are collect data needed to develop/optimise waste management strategies these cities. The required scopes of data are derived from dliverable and for the sustainability indicators in deliverables

1 Introduction

The review of the results of the EU 5th Action Programme shows that despite much progress in waste management still large discrepancies in the implementation of waste policy among the EU Member States and regions can be observed. The problem of waste strikes strongest the large cities, especially their highly populated zones where the opportunities for waste minimisation through e.g. home composting of bio-fraction are limited and the lack of free space significantly restricts the waste management infrastructure. Poorly planed waste management system can cause a serious nuisance for a city dwellers. For instance, in the northern countries on average 20% of the household waste is collected separately, while in the southern only 5%
Difficult situation in waste management exists in the EU Accession States. These countries currently undergo a process of harmonisation of their national environmental laws to the EU policy. In the coming years they need to fulfil a number of requirements to adjust to the

European standard. One of the primary tasks is to develop waste management plans at a local, regional as well as national level. The other, more difficult task is their implementation.
In this paper the problems of the waste management situation in the EU Accession countries.. Furthermore, the objectives of the research project initiated within the European Commission's Fifth Framework Programme: Energy, Environment and Sustainable Development Work Programme - Key action: City of tomorrow and Cultural Heritage, are briefly presented. Objective of this project is to develop decision supporting tools for waste management planning and optimisation. An insight to the waste management assessment model, which is planned as one of the deliverables of the mentioned project, will be given. Finally, focusing on the environmental assessment part of the model, a short overview of LCA and Streamline LCA methodology will be given as well as the way this will be applied to waste management planning purposes.

2 Project Description

Addressing the above outlined needs the main objective of the proposed research project is to develop tools to support planning of new and optimisation of existing waste management systems in the European regions with fast growing economies, such as EU Accession Countries and south European countries. The projected outcome of the project will be two decision supporting models for waste management planning:

- **waste prognostic model** {enable predicting changes in municipal waste generation (quantity and composition) based on other developments taking place in those regions (e.g. socio-economic developments, technological developments, changes of consumption patterns, etc.)}.
- **assessment model for environmental, economic and social performance of integrated waste management strategies** { will be based on sustainability criteria and indicators that will allow for quantification of the satisfaction of environmental, economic and social functions of a given waste management scenario}.

Optimal waste management scenarios will be verified for the selected cities from fast growing European regions in six cities in Slovakia, Poland, Lithuania, Greece and Spain.
The selected cities come from various European regions and climate zones, and thus also the waste composition and quantities vary. The size of cities ranges from less the 100 thousand to over 4 million inhabitants, however all of them are important municipalities in the respective countries. The scale of waste management problems in selected cities is varied. In Polish, Slovak and Lithuanian cities waste management plans do not exist yet. Waste is only landfilled there. In the selected Greek city, there is a need to develop a new waste management plan. Whilst in the Spanish cities, revision and optimisation is needed due to problems with implementation and too ambitious objectives of the current waste management plan.

3 Expected Impact

Adequate waste management plans are prerequisite of an efficiently functioning waste management systems. (Council Resolution 97/C 76/01). Therefore, one of the primary objectives of the 5th EAP was to develop waste management plans at the national and local

level. The review of the results of the EU 5th Action Programme shows that despite much progress in the waste management the objectives were not achieved.
A need for developing sound waste management practise across Europe, which promotes waste recovery and recycling has been reaffirmed, along with the further demand for waste management plans at the national and local levels (EEA 2000, 6th EAP).

This research project addresses the above stated needs for improvement waste management practice across Europe. The objective of the proposed project is to develop support tools for waste management planning and monitoring.

It is believed that the waste generation prognostic model will be a useful tool to plan waste management systems with appropriate capacity and thus functioning efficiently in both environmental and economic terms. Another project deliverable, sustainability criteria and indicators with optimisation tool will allow for optimisation of the waste management system at the planning phase and thus minimise its negative impacts. This systematic approach to waste management planning will ensure better environmental and life quality for the future for the targeted regions and in consequence for the whole Europe. The objective of the proposed project is to develop support tools for waste management planning and monitoring.

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The targeted **End-Users** of this project are various actors involved in waste management planning and monitoring: primarily, in the cities and regions with fast growing economies (for planning, assessment and optimisation) secondly, in any other European regions (for monitoring, benchmarking and optimisation).
Namely, the targeted End-Users are:

- **Municipalities** and **local/regional authorities** responsible for spatial and waste management planning and development in cities,
- **Institutes** and **Consultancies** developing or/and evaluating local waste management plans,
- **Officials** and **decision makers** who verify the proposed developed strategies for waste management in cities and regions.

For all these end-users the developed tolls will provide an objective, practical and science-based decision support.

4 Project Workplan

Structure of the Work Plan: The work plan consists of eleven task-specific workpackages and one additional package , which is concerned with project administration.
Additionally three general thematic tasks can been distinguished:
Task I: Data collection and analysis:
Task II: Methodology and tools development:

Task III: Implementation and dissemination of the results

In the first time, data on municipal waste generation trends along the social, economic and political developments in the advanced EU Member States will be collected and analysed. The objective is to understand the reasons underlying waste generation trends and thus enable predicting qualitative and quantitative waste characteristics for the purpose of waste management planning. Firstly, the leading partner will define the scope of needed information. Further, all the involved, partners will collect data from (at least) their respective countries and as far as possible other (neighbouring) countries. In this way data from following countries will be provided: Germany, Spain, Austria , Poland, Greece, Luxemburg and the Netherlands .

In other time the Partners from the selected cities (Tarragona and Barcelona, Xanthi, Wroclaw, Kaunas, Nitra, Bratislava) will collect data needed to develop/optimise waste management strategies these cities. The required scopes of data will be derived. The goal of this work is to develop a Waste Prognostic Model for cities

Qualitative criteria and quantitative qualitative sustainability indicators for the waste management assessment/benchmarking and optimisation will be developed. The criteria will be based on case studies of Waste Management in the selected "model cities" in the advanced EU Member States. The project Partners will focus on the respective areas of their expertise: waste collection/logistics and waste treatment/disposal, with regard to three aspects of sustainability: environmental, economic and social partners. Further for the deteremined conceptual sustainability criteria a set of quantitative indicators will be provided. The developed sustainability criteria and indicators will be subject to the verification by Scientific Committee in other time.

The goal of work is to implement the developed tools for waste management planning in the selected cities (Tarragona and Barcelona, Xanthi, Wroclaw, Kaunas, Nitra). The objective of this package is twofold: *firstly*, to verify the developed tools in practice by the third persons (Partners from the respective cities) and *secondly* to design optimal waste management for the selected cities.

At this stage the output data will be compiled to develop the Handbook. Also the experiences of the first Users of developed software tools will be collected and the models respectively modified/improved.

This work will be targeted at dissemination of the projects results, with emphasis of the primary application regions, i.e. European regions with fast growing economies. For that reason the Handbook on waste management planning will be translated to the native languages of respective Partners.

The following Project's Milestones are planned:

1 Waste Prognostic Model - Software Tool
2 Software tool for Sustainable Waste Management Planning, Assessment & Optimisation
3 Optimal waste management scenarios for 6 selected cities in Tarragona and Barcelona, Xanthi, Wroclaw, Kaunas, Nitra
4 Handbook on Waste Management Planning Assessment and Optimisation & Results Dissemination

5 Data collection in the selected cities

To gather data for waste management planning in the selected cities from: Spain, Poland, Greece, Slovakia and Lithuania with the use of developed in the project tools: waste prognostic model sustainability criteria.

The required data for waste management planning in the selected cities will be collected by the respective Project Partners The partners have well-established contacts with the municipalities, through a long term co-operation on waste related project in the region. The scope of data needed to implement a waste prognostic model and the developed sustainability indicators defined by the Partners responsible for deliveries. Information on environmental national legislation, policy plans and recommendations relevant for waste management planning will be needed. Besides, data on:

- current state of waste management, existing waste processing facilities, elaborated plans;
- demographic and geographic data and prognosis;
- existing infrastructure and their ownership structure;

spatial plans, etc.

The common understanding of certain waste related terms and definitions as well as scope of needed data consulted with the responsible partner from the Scientific Committee.

6 Conclusion

The following strategic impacts of the proposed project are expected:

- Improvement of the quality of a city life through better planning of its infrastructure
- Stimulation of improvement of waste management in various European regions
- Support for the idea of sustainable development in waste management sector
- Support for implementation of EU waste policy
- Encourage the municipalities to improve their actual waste management towards integrated systems
- Creation of new working place in control and monitoring of integrated waste management systems as a whole

Supporting the local politicians in introduction of new waste management strategies - tools for the effective and professional discussion with the public

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Managing Irrigation Projects by Using Extended GIS Model

Milan Čistý

Abstract: Integrated land use and water management play significant role in sustainable agriculture in the Slovak Republic. The most productive soils are located in regions with occasional water scarcity. Large scale irrigation projects are built in order to stabilize the farming industry and offset adverse effects and damage caused either by occasional shortage or an uneven distribution of precipitation.

In connection with the process of social and economic transition, which is under way in the region, irrigation projects need to meet other requirements, than those for which they have been designed. The paper presents application GIS and modern heuristic methods to evaluating rehabilitation needs of irrigation systems.

Key words: Irrigation, Rehabilitation, GIS, Genetic Algorithms

1 Preface

Transformation of the socio-economic system in Central and Eastern Europe, followed by the decline in the agricultural production and favourable hydrologic conditions, resulted in a negative impact on the exploitation of irrigation systems in the region. This situation lead to unfitness of irrigation equipment at times of dryiness (e.g. this spring). This fact will have negative impact to nation's agricultural output. Some of the larger sprinkler irrigation systems, which were built in the Slovak Republic starting from early fifties, are now approaching the end of their useful life. Some of them are out of operation, due to their wear-out, obsoleteness or more serious failures.

The need for irrigation in the individual regions of Slovakia is presumably determined by annual rainfall and its distribution throughout the year as well as by other climatic factors, as air temperature and saturation deficit are. Hence now, in the majority of irrigation networks, there are more consumers of irrigation water. As a result, there is higher demand for irrigation water during the peak period than is available in the system due to its limited capacity, i.e. both irrigation piping system and pump stations. That is why rehabilitation and modernisation of these systems has become a centre of primary interest for irrigators of today.

Geographic Information Systems (GIS) are introduced for use in irrigation industry in Slovakia (Jenčo, 1999). Recent advances (in both GIS technology and user implementation) treat the GIS as a true spatial modeller, in which the connectivity of the network, the hydraulic behaviour of components, and its relationship to consumers are paramount. Windows operating systems and inter-application communication encourage the use of GIS as an integrated component, rather than an isolated tool. This paper is describing the possibilities for extending analytical tools of GIS by OLE Automation technology, so they could be used for designing of rehabilitation of the irrigation projects.

2 Methodology

Used GIS package Autodesk MAP includes integrated Microsoft Visual Basic for Applications scripting environment. Microsoft Visual Basic allowed programmers to create stand-alone Windows applications which previously required the use of complex C programming languages. Visual Basic for Applications is a subset of Visual Basic, designed specifically for certain applications such as MS Word, Excel, Access, etc. With the release of AutoCAD R14, AutoCAD has included VBA as part of the AutoCAD package. VBA for AutoCAD includes most of the functions and controls available in the stand-alone version of Visual Basic along with specific functions and controls specifically designed for AutoCAD.
ActiveX Automation focuses on objects. An object in AutoCAD is just like an object in daily life, such as a person or a house. This object has various characteristics that define it; these defining attributes are called properties. A property can be a single feature (a person's height or hair color), another object (a house), or a collection of objects (your treasured LP collection). To manipulate an object, you use methods. Methods define an action that can be performed on the object (a car can be driven) or done with the object (a person can walk and jump).
To exemplify, one of object in AutoCAD MAP is called the Drawing object. When a drawing is open, it is represented programmatically as an instance of the Drawing object, which provides a drawing with properties and methods that can be used to manipulate it. One of the Drawing properties is for instance FullName, the full path name of a drawing, and one of its methods is Save. Objects can contain other objects. Some types of contained objects are contained as individuals, and some as members of collections. The ActiveX objects of an application, their hierarchy of container relationships, and all the properties and methods that knit them together into a coherent programming interface are referred to collectively as the application's object model.

AutoCAD ActiveX Automation object model (can be found in the AutoCAD Automation Reference on-line manual) lists the AutoCAD objects that are available through ActiveX Automation. The topmost object is the Application, which encapsulates AutoCAD itself. This object contains the Preferences object as well as the Document object that represents the current drawing.
The Document object in turn contains Plot and Utility objects as well as several collection objects, each of which manages a group of individual objects. For example, the Views collection contains every view in the drawing, and the Model Space and Paper Space collections contain every entity in model space and paper space, respectively.
By means of Visual Basic it is possible to manipulate with the GIS object model (programmatically manipulate with GIS data), add new features to program or attach and integrate thereto external programs. The software system presented here has attached the following analytical tools for network rehabilitation: linear programming model for optimization, simulation model of the network hydraulics, some specific analytical tools (longitudinal profiles, etc) and genetic algorithm model for simulation model calibration and for rehabilitation optimization. Application genetic algorithms is now described in more detail.

3 Genetic Algorithm Model

The core of presented system is an ActiveX genetic algorithm component which is not specific to any particular domain. This component has been integrated with MAP, a Geographical Information System (GIS) for Microsoft Windows. By means of the Visual Basic for Application also Epanet Toolkit (Rossman, 2000) for water systems network modeling was included.

Genetic algorithms are stochastic search methods that mimic the process of natural selection and mechanism of population genetics. Description of the GA can be found in Goldberg (1989). The following pseudo-code shows how a GA works:

```
BEGIN /* genetic algorithm*/
   Generate initial population;
   Compute fitness of each individual;
   WHILE NOT finished DO LOOP
         BEGIN
               Select individuals from old generations for mating;
               Create offspring by applying recombination and/or mutation
               to the selected individuals ;
               Compute fitness of the new individuals;
               Kill old individuals to make room for new chromosomes and
               insert offspring in the new generation;
               IF Population has converged THEN finishes := TRUE ;
         END
  END
```

In case of pipe network rehabilitation, the optimization problem lies in minimizing the cost of upgrading the network while preserving standard operational conditions. Rehabilitation options can include pipe removal and replacement (with pipes from any set of those with available diameters) or pipe duplication with pipes of any diameter. The chromosome can be either a binary or integer string of numbers (representing possible decisions), which thus defines the network design.

A model is first set up, incorporating all the options for individual network components. The GA then generates trial solutions, with each being evaluated by simulating its hydraulic performance on the simulation module. Any hydraulic infeasibility, such as failure to reach a specified minimum pressure at any demand point, is registered, and penalty cost calculated. Operational (e.g. power-consumption) costs can also be calculated at this point, if required. Penalty costs are then combined with anticipated capital and operational costs to obtain an overall measure of quality of the trial solution. The process will continue for many thousands of iterations (Fig. 1), and a population of good feasible solutions will evolve.

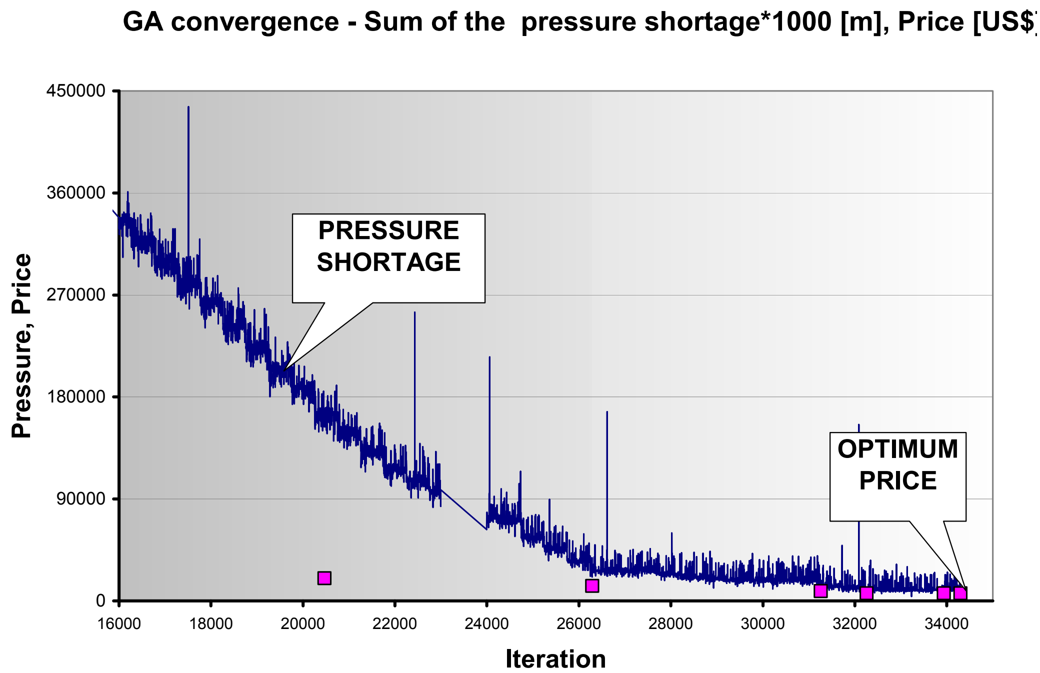

Fig.1 GA Convergence

In order to use simulation models which are incorporated to GIS package e.g. for the design of the reconstruction of these systems one has to perform their calibration. The purpose of using genetic algorithms in the calibration process is to eliminate redundant manual effort spent to search for calibrated parameters values using trial and error approach, which becomes often very time consuming business and almost irresolvable in more complicated cases. To search for the best possible solution of a problem GA uses such principles as selection, breeding or mutation.

The main (although not the only possible) variables used in GA to resolve calibration problem within the system are usually pipeline roughness coefficients. Roughness coefficients (i.e. variables searched for) could be searched for individually - for each pipe, or per group of pipes with an assumed identical values of roughness coefficient (where such group had been created prior to the beginning of calculation). Selection into groups can be based on the pipeline diameter, material, age and location or on the combination of these factors.

In a case using the method of genetic algorithms each potential solution of calibration problem is encoded in a chromosome, which has a number of genes identical with the number of groups of pipes with previously postulated identical (but not yet known) roughness coefficient or with the number of sections, provided we look for an individual roughness coefficient for each pipeline. Gene values are assigned pre-set interval of values containing the expected result. Should an algorithm fail to find a good solution it is either necessary to search for the source of errors or to extend these intervals of legitimate values.

4 Conclusions

Economic and hydraulic effectiveness of the rehabilitation of sprinkler irrigation systems is largely conditioned by the fact how effective was the rehabilitation of their pipe network. Due to complexity of networks and inability to cope well with discrete, non-linear combinatorial problem, such as pipe network optimization, the conventional optimization techniques are ill-suited for solving this task. The main practical advantages of using genetic algorithm approach are:

- Ease computing of complex networks (i.e., each part of the system that can be simulated could also be the subject of optimization).
- Relatively simple methodology (compared to conventional methods of mathematical programming).

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Author address:
Čistý Milan, Slovak University of Technology, Faculty of Civil Engineering, Department of Land and Water Resources Management, Radlinského 11, 813 68 Bratislava, Slovak Republic (E-mail: cisty@svf.stuba.sk)

Vertical Deformations Condition of Embankment Dams with Application of 2D and 3D Analysis

Stanislava Dodeva

PWME "Water Management of Macedonia"
III Makedonska brigada 10, 1000 Skopje, Republic of Macedonia
e-mail: dodeva@water.org.mk

Abstract

One of the most frequently used methods for analyzing the stress-deformation condition of the embankment dams is the Finite Element Method (FEM), both for two-dimensional and three-dimensional models.
The two-dimensional analysis enables to analyze only the cross section of the embankment dams, without possibility to simulate different valley-wall slopes, which is actually possible to be done applying three-dimensional analysis. But, on the other hand, this analysis engages significantly more time in the preparation of the input data phase and in the phase of interpretation of the output results. In order to compare the results of both analyses, certain research on stress-deformation condition has been performed on embankment dam applying 2D analysis. On same dam latter, located in 5 valleys with different valley-wall slopes, 3D analysis has been performed. This research also, enables to define the limit of the ratio of the width of the dam site and the height of the dam for application of 3D analysis instead of 2D analysis.
Two-dimensional analysis has been performed with the computer program package GEO-SLOPE, component SIGMA/W. Linear-elastic constitutive model for the material behaviour has been applied, finite elements network was automatically generated with isoparametric finite element with 8 nodes, while the results were graphically presented. Three-dimensional analysis has been performed with originally created computer program DAM3D, which uses isoparametric three-dimensional quadratic finite element with 20 nodes, joint elements between different types of materials, linear-elastic constitutive model for the material behaviour and graphically presents the results.
In this paper, from the whole comprehensive analysis, only the vertical deformations condition is presented for 2D and 3D cases, comparison of the results is performed and a conclusion about the limit of the ratio of the width of the dam site and the height of the dam for application of 3D analysis instead of 2D analysis, is defined.
According to the obtained results, it can be concluded that the maximal values of the vertical deformations of the embankment dams, resulting from 2D and 3D analysis, are very close for the dams in the valley with valley-wall slopes of 1:3 and flatter. For these types of dams, vertical deformations obtained by 3D analysis are 92% of those obtained by 2D analysis. For the other two types of dam, located in the narrow valley, vertical deformations are constrained and present 75% and 83% from those obtained with 2D analysis.
Regarding the limit of the ratio of the width of the dam site and the height of the dam for application of 3D analysis instead of 2D analysis, during its definition, all results from the stress-deformation condition are considered. General conclusion is that if dams are located in the valleys with valley-walls slopes of 1:3 and flatter, it is recommended to apply 2D analysis, while if dams located in narrow valleys are analyzed, then, it is recommended to apply 3D analysis.

1 Introduction

Two-dimensional analysis in the framework of this research is performed by applying the computer program package GEO-SLOPE, which contains several components. In order to define the stress-deformation condition, the component SIGMA/W was used. This whole computer program package uses isoparametric finite element with 8 nodes, enables automatic generation of the finite element network and simulation of several different models of material behaviour and has wide opportunities for graphical presentation of the results.
For the needs of three-dimensional analysis, in the originally created computer program DAM3D, three-dimensional isoparametric quadratic finite element, with 20 nodes is used. In order to simulate as close to the real condition as possible and to get clear presentation of the interaction process between the structure and the base and valley-walls, on the contacts between different types of materials, joint elements are introduced. The basic difference between this element and the ordinary finite element is that the joint element enables differential displacements of different materials along the contact area, while finite elements are tied in the nodes and there is no possibility for relative displacements on the contact areas of different materials. Computer program DAM3D uses linear-elastic model on material behaviour, enables half-automatic generation of the finite element network and graphically presents the results.

2 Analyzed Dams

For two-dimensional analysis, embankment dam of height of 100 m, with the following parameters, is analyzed:

-slope of the embankments 1:1,8,
-width of clay core at the base is 50 m and in the crest 4 m,
-the width of the two layers filter material next to the core is 4 m,.
-materials: rock for the embankments, filter material and clay for the core.

The dam is located on rocks, which are undeformabile and impermeable. Construction in layers is simulated, so, there are 9 layers of 10 m width, while the last 10 m of the dam are divided into two layers, one of 6 m width and the last one, eleventh, of 4 m. width. The cross section of the analyzed dam together with the finite elements network is presented on the following figure.

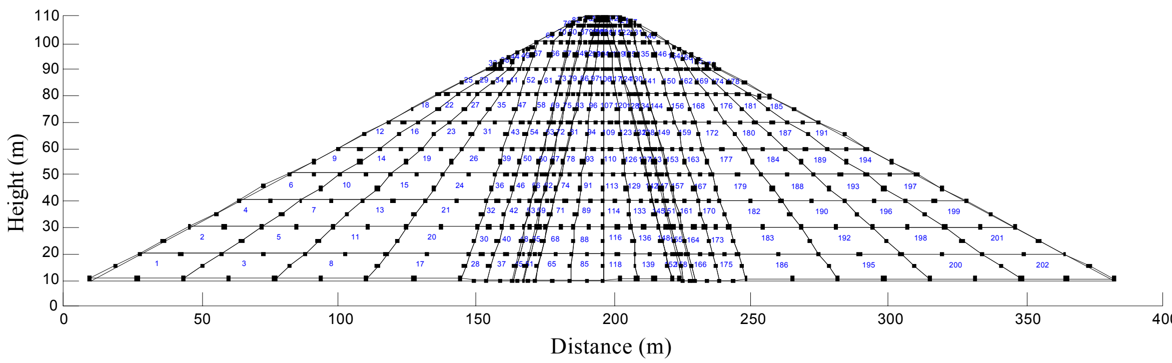


Fig. 2.1. Cross section of the embankment dam with finite element network
No. of elements: 202
No. of nodes: 657

Regarding the material behaviour, linear-elastic model is applied with the following parameters:

Table 2.1

Material	**E** (KN/m^2)	ν	γ (KN/m^3)	φ (^{0})
Rock	60.000	0,30	20	42
Filter	50.000	0,35	19	36
Clay	10.000	0,45	21	20

For the three-dimensional analysis with the computer program DAM3D, models of five dams with identical cross section as the dam in 2D analysis, are located in valleys with different valley-walls slopes. The analyzed cases are: valley-wall slope of 1:1, 1:2, 1:3, 1:4 and of 1:6. Accordingly to this, different width of the riverbed are adopted: for the valley with the valley-slopes of 1:1 and 1:2, 20 m, for the valley with the valley-slopes of 1:3 and 1:4, 40 m and for the valley with the valley-slopes of 1:6, 60 m.
Due to the double symmetry of the dams, the models of the dams present only ¼ of the whole dam construction. The finite element network is half-automatically generated, using isoparametric "quadratic" three-dimensional elements with 20 nodes. The height of the dam is divided into 9 layers, seven of them are 13 m high, and the eighth layer is 5 m high and the last one, the ninth, 4 m. The cross sections of all five dams have identical finite element network, while the longitudinal sections are differently divided, as it is shown on the figures from Fig. 2.2. up to Fig. 2.6. On these figures, the number of elements and number of nodes, for each type of dam, are presented.
In order to analyze differential displacements on the contact zone between the dam and the base (river bed) and valley-walls, in the process of modelling, joint elements on those contact zones are introduced. These elements have only two dimensions and no thickness. Total number of joint elements for the dams in the narrow valley, with valley-wall slope of 1:1 and 1:2, is 89, while for the dams located in flatter valleys, the number of joint elements is 105.
Dams are located on undeformable and impermeable rock base and are laying on valley-walls with the same characteristics. Materials are following linear-elastic constitutional model of behaviour with the same values of the parameters as presented in the Table 2.1. The parameters of the joint elements are the following: Kn = 1.000.000 KN/m^2 and Ks = 10.000 KN/m^2. The stiffness in normal direction is about 16 times higher then the stiffness module of the rock in the other (tangent) direction.

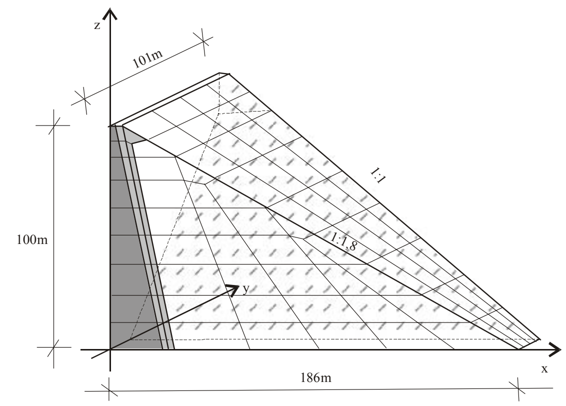


Fig. 2.2. Three-dimensional model of a dam in valley with valley-walls slope of 1:1

No. of elements: 228
No. of nodes: 1592
No. of joint elements: 89

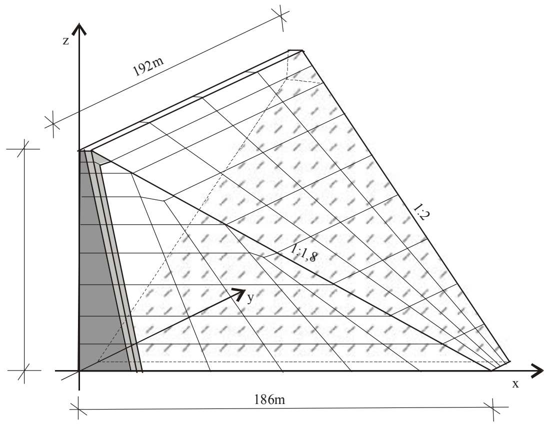


Fig. 2.3. Three-dimensional model of a dam in valley with valley-walls slope of 1:2

No. of elements: 228
No. of nodes: 1592
No. of joint elements: 89

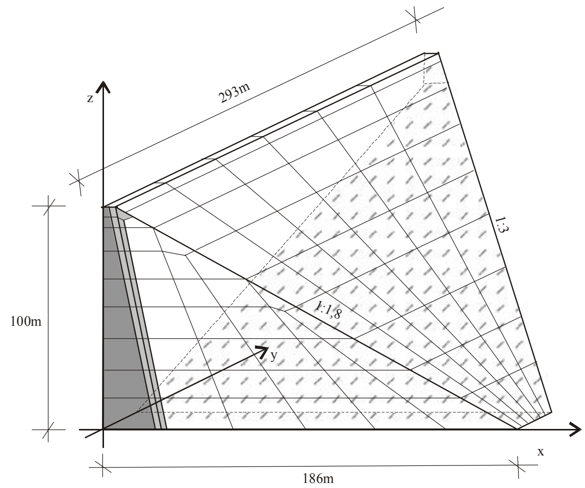


Fig. 2.4. Three-dimensional model of a dam in valley with valley-walls slope of 1:3

No. of elements: 342
No. of nodes: 2188
No. of joint elements: 105

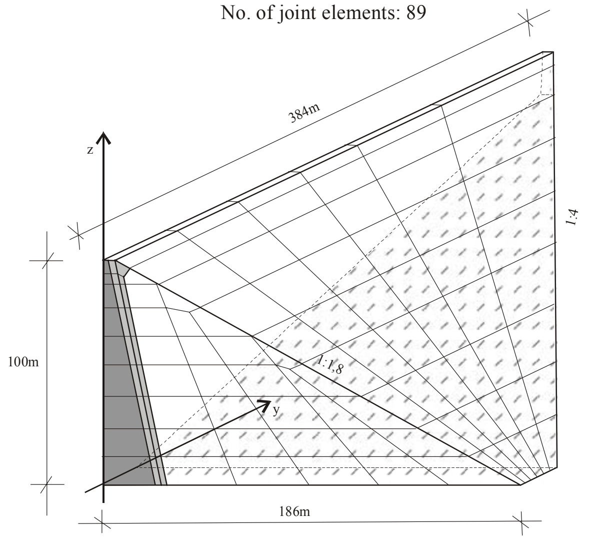
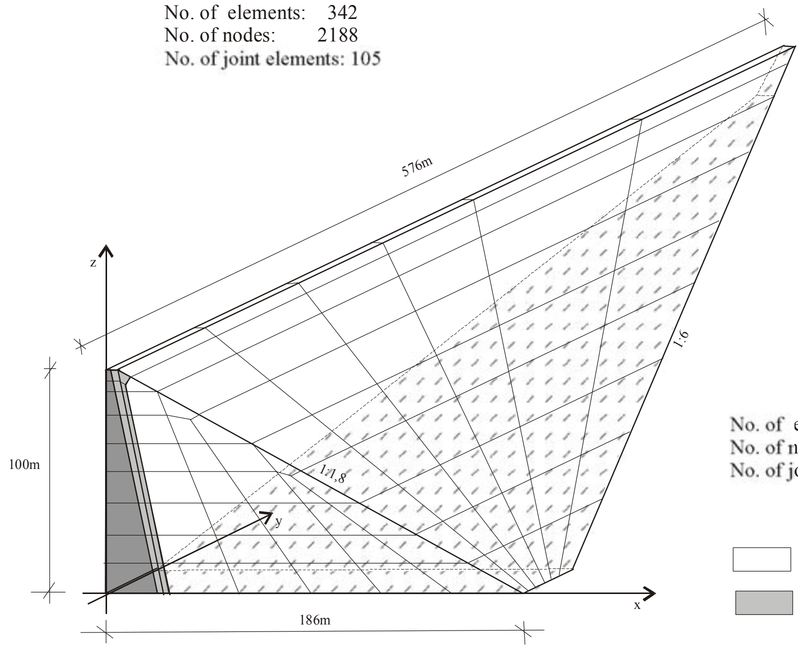


Fig. 2.5. Three-dimensional model of a dam in valley with valley-walls slope of 1:4

No. of elements: 342
No. of nodes: 2188
No. of joint elements: 105



No. of elements: 342
No. of nodes: 2188
No. of joint elements: 105

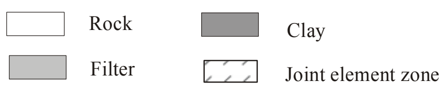


Fig. 2.5. Three-dimensional model of a dam in valley with valley-walls slope of 1:6

3 Results

In this paper, comparison of the results obtained with two-dimensional analysis using computer program SIGMA/W and the results obtained with three-dimensional analysis using computer program DAM3D, is performed. The comparison is addressed to cross-section of the dams, actually the plane *xz*. Also, the results for the vertical deformations of the dams in longitudinal sections obtained with three-dimensional analysis, are presented.
Results of the two-dimensional analysis for the vertical deformations are given on the Fig. 3.1.

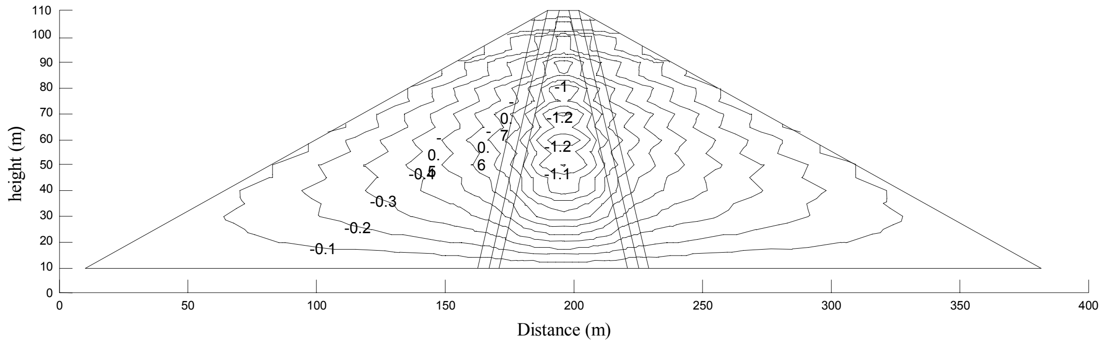


Fig. 3.1. Two-dimensional dam analysis
Vertical deformation (m)

The maximum vertical deformations (settlements) are in the middle part of the clay core and are 1,20 m, while the settlements of the dam crest is 0,1 m.
The results of the vertical deformations obtained by the three-dimensional analysis are presented on the cross sections of the dams, from Fig. 3.2. up to Fig. 3.6.
As in the other analyses, the maximum deformations are in the clay core, at 50-60 m from the basis, which means on about 50-60 % of the height. According to the results, with flattering of the valley-walls slopes, vertical deformations are increasing: maximum vertical deformation for the dam in the valley with valley-wall slope of 1:1, are 0,90 m; for the dam in the valley with valley-wall slope of 1:2, are 1,0 m; for the dam in the valley with valley-wall slope of 1:3, are 1,1 m. For the two remaining dams, located in the valley with valley-wall slopes of 1:4 and 1:6, there is no difference in the maximum values of the vertical deformations, which are 1,10 m, as well as in the isolines shape and locations, compared to the dam located in valley with valley-wall slope of 1:3.
Regarding the settlements of the dam crest, for all types of dams are 0,10 m, and there are no differences both, in the values or in isolines pattern.
Comparison of the vertical deformations obtained with three-dimensional analysis presented on Fig. 3.2. up to Fig. 3.6., with the vertical deformation obtained with application of two-dimensional analysis, presented on Fig. 3.1., shows good compatibility of isolines pattern and location of the maximum deformations, while there is certain differences in the values of the maximum deformations. Next Table 3.1. presents the comparison between the maximum vertical deformations from 3D analysis to maximum vertical deformations from 2D analysis. The value of the maximum settlements obtained with 2D analysis is 1,20 m.

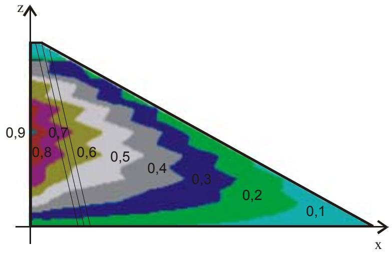


Fig.. 3.2. Dam in valley with valley-wall slope of 1:1

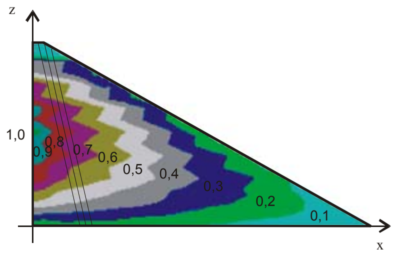


Fig.. 3.3. Dam in valley with valley-wall slope of 1:2

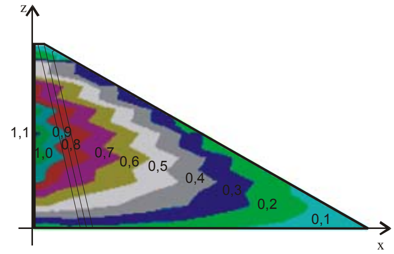


Fig.. 3.4. Dam in valley with valley-wall slope of 1:3

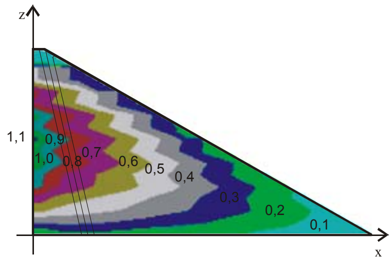


Fig.. 3.5. Dam in valley with valley-wall slope of 1:4

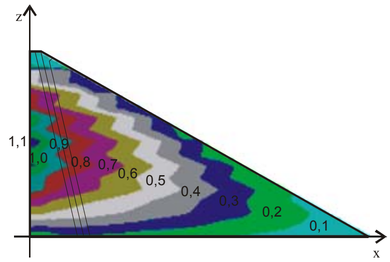
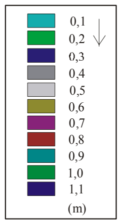


Fig.. 3.6. Dam in valley with valley-wall slope of 1:6



Vertical deformations in the plane xz

Table 3.1. Vertical deformations

	1:1	1:2	1:3	1:4	1:6
3D (m)	0,90	1,0	1,10	1,10	1,10
$\frac{3D}{2D}$ (%)	75	83	92	92	92

There is rather high compatibility of the maximum values of the vertical deformations resulting from 2D and 3D analysis for the dams located in the valley with valley-walls slope of 1: 3 and flatter. For other two dams, located in narrow valleys, the settlements are constrained and present only 75 % and 83 % of those settlements resulting from 2D analysis.
Three-dimensional analysis gives graphical presentation of horizontal and vertical deformations in both planes of symmetry: *xz* and *yz*. For the purpose of this paper, vertical deformations in the plane *yz* are also presented, on the Fig. 3.7. up to Fig. 3.11. These figures are showing the influence of the inclination of the slope of the valley-walls on the development, pattern and values of the vertical deformations in the plane *yz*.
On the axis of symmetry, *z*, the values of these deformations are identical with the vertical deformations in the plane *xz*, actually, maximum values are increasing with flattering of the slope of the valley-walls, up to the ratio 1:3.
For the dam in the valley with valley-walls slope of 1:1, maximum settlements in the plane *yz* are 0,90 m, for the dam in the valley with valley-walls slope of 1:2, maximum settlements are 1,00 m and for the dam located in the valley with valley-walls slope of 1:3, maximum settlements are increasing again up to 1,10 m. For the next two dams in the valleys with valley-walls slope of 1:4 and 1:6, maximum settlements are the same as previous dam, 1,10 m. It is interesting to note that the zones of deformations proportionally are increasing with the width of the riverbed, actually with flattering of the slope of the valley-walls.

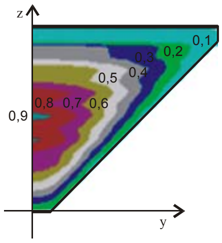


Fig.. 3.7. Dam in valley with valley-wall slope of 1:1

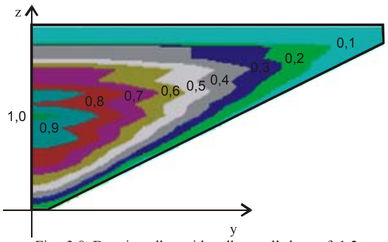


Fig.. 3.8. Dam in valley with valley-wall slope of 1:2

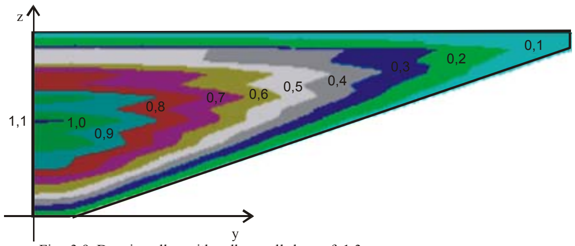


Fig.. 3.9. Dam in valley with valley-wall slope of 1:3

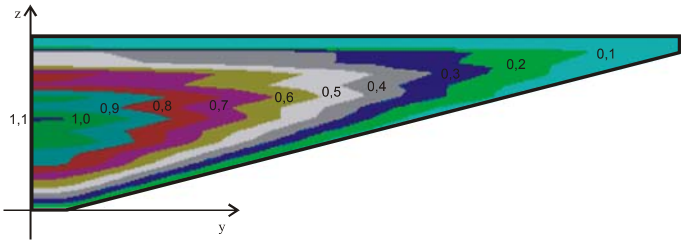
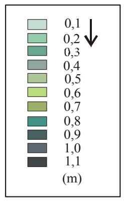


Fig.. 3.10. Dam in valley with valley-wall slope of 1:4



Vertical deformations in the plane yz

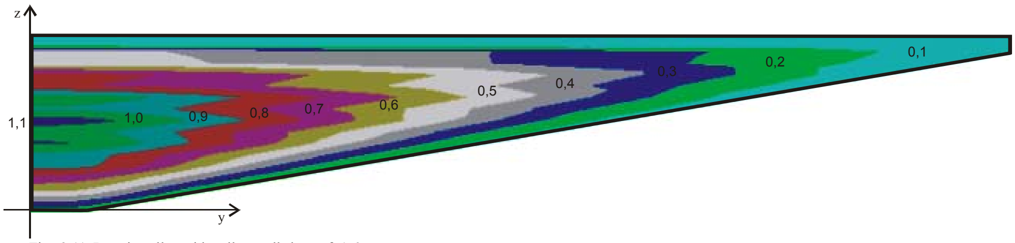


Fig.. 3.11. Dam in valley with valley-wall slope of 1:6

4 Conclusions

- Regarding the results for the vertical deformation for the plane *xz* obtained by three-dimensional analysis, for the first three dams, it can be concluded that with flattering of the slopes, maximum values of the vertical deformations are increasing, and also, the zones of larger vertical deformations are increasing. This fact indicates on "hanging" effect of the dam upon the narrow valley, which effect constrains the vertical deformations. As the valley becomes wider, the "hanging" effect has less influence, so that the dam settles down with larger values of settlements.

- Comparison of the results of vertical deformations obtained by two-dimensional and three-dimensional analysis (for the plane *xz*), shows good compatibility for the dams located in the valleys with valley-walls slope of 1:3 and flatter, while for the dams in narrow valley, the differences in the maximum values are more expressed.

- Regarding the results for the vertical deformation for the plane yz obtained by three-dimensional analysis, for the first three dams, it can be concluded that with flattering of the slopes up to 1:3, maximum values of the vertical deformations are increasing. Also, the zones of larger vertical deformations are increasing with increasing of the width of the riverbed, up to the last dam.

- Considering all the results obtained by both analysis, which results include stress-deformation conditions of all dams under research, the limit of the ratio of the width of the dam site and the height of the dam for application of 3D analysis instead of 2D analysis, can be defined. According to the results, for analyzing of dams in narrow valleys, actually dam in the valleys with valleys-walls slope up to 1:3, three-dimensional analysis should be applied, while for the dams located in valleys with flatter valley-walls, two-dimensional analysis should be applied.

5 References

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Influence of the Transported Water on the Steel Pipeline

Vanda Dubová, Jozef Kriš

Slovak University of Technology, Faculty of Civil Engineering ,Department of Sanitary Engineering, Radlinskeho 11, 813 68 Bratislava, Slovakia, E-mail : dubova@svf.stuba.sk

Abstract
The paper deals with corrosion tests done on water pipeline Jelka - Galanta - Nitra. The corrosion of the steel pipes was tested in two periods – the first in the years 1995-96, when the water was disinfected by the gaseous Cl_2 and the second in the years 2001–03, when as disinfectant was used the ClO_2. Tests were done on the samples according to Slovak Technical Standards as well as on the samples with 2.5 times greater area. Described are the tests and compared the results (especially corrosion velocities) of both tests series.

1 Introduction

The water pipeline systems were in the past, but also in presence constructed from steel conduits that were except of their good mechanical attributes, liable to corrosion. The internal corrosion of steel pipeline causes vast problems and financial losses, whereas the failure-free operation of water supply systems depends on it's functioning. All water supply providers meet with internal corrosion of pipelines, to higher or lower extent. To know, if the anticorrosion measures are needed, is necessary to understand the interaction of transferred water and pipeline material. Therefore is comprehensible that the operators are monitoring corrosion, especially when there a change of disinfectand of distributed water occurs.

Therefore we have provided corrosion tests on long-distance pipeline system Jelka – Galanta – Nitra, which is situated in southern Slovakia, since the assessment of corrosive influence of water only on base of chemical analyses is not always sufficient. We have accomplished the biannual and annual tests on this group water conduit already in the years 1995-96, when the transferred water has been disinfected with gaseous Cl_2. In later period the sanitary provision was reconstructed to ClO_2. In transitional period both gaseous Cl_2 and ClO_2 were dosed into water, now its only ClO_2 used for this purpose.

2 Corrosion tests and their evaluation

The group water conduit Jelka – Galanta – Nitra is constructed mainly from steel piping DN 700, which is 52 km long, whereby the water must be pumped 4 times.

In water piping system the groundwater is transferred, which is with its physical and chemical attributes up to standards of the requirements of Ministry of Health SR Regulation No 29/2002 Coll. from 9.1.2002 „On requirements for drinking water and drinking water quality", which fully corresponds with valid EU standards. The quality of well water does not significantly change within a year in spite of longer transport distance and water retention in the system; the chemical status of transported water does not significantly change. The pH values of transported water range from 7.6 – 7.7, $ANC_{4,5}$ 4.1 – 4.3 $mmol.l^{-1}$ a Ca^{2+} 90-98 $mg.l^{-}$

[1]. Water is secured by chlordioxide, whereas on the discharge in Jelka – at the beginning of the trace in water occurs 0.16 mg.l^{-1}ClO_2, on the discharge in Galanta 0.12-0.13 mg.l^{-1} ClO_2 and on the inflow to storage reservoir in Nitra there are 0.05 – 0.06 mg.l^{-1} ClO_2.

We have accomplished the corrosion tests on three points of the system, concretely in Jelka - at the beginning of the trace behind the accumulation reservoir on the water input into the water supply system, then in Galanta on piping station on 13-th km of the trace and at the end of the trace in Nitra – in armature chamber of the storage reservoir. (Figure 1)

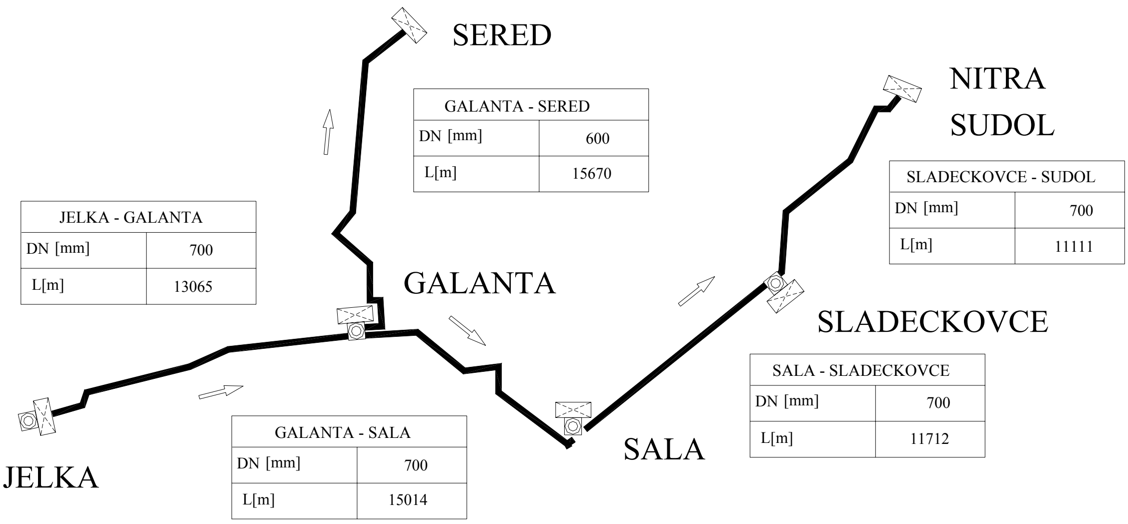

Figure 1: Scheme of water distribution system

Compared to the measurements accomplished in the years 1995-96 the corrosion monitoring facilities are placed at the same places in Jelka and Nitra, only the monitoring facility on the trace by Galanta has been added.

The corrosion tests were done in compliance with Slovak Technical Standard (STS) 83 0615 „Requirements for water quality transported by pipeline“, which results from the difference between the weight decline of test samples by the day 30 and 60 after its exposure to the influence of stream water. The corrosion velocities were computed from corrosion declines on the base of which have we assigned water with (on of the 3 possible) aggressiveness levels, and determined if the anti-corrosion measures are necessary or not. For more exact determination of corrosive velocity and its process have we continued the testing and we did measurements exceeding the requirements of STS, by the day 191, 364 and 554.

On every monitoring place were inserted 4 holders with tested samples. They were concretely installed into glass tubes, and with help of PVC hosepipes connected directly onto the pipeline. Into every holder 5 samples isolated by dielectric element were inserted. The tested samples were made of steel plate of class 11, with size 42 x 42 mm and thickness of 1mm. The 30-day corrosion was monitored in most highly placed holder in the tube, in next holder have we measured and changed the samples after 60 days, then instead the third holder, we have inserted the new holder with samples where also the annual 365-day corrosion was monitored and the under most holder has been used for monitoring of 1.5 year – 554-day corrosion.

We have monitored corrosion declines, corrosion velocity, velocity of water in the tube, chemical composition of water and in addition to previous tests also the sedimentation amount in particular samples to recognize, if there exists a relation between the sediment amount and the corrosion velocity, respectively the corrosion type.

After 120 testing days the samples of 2.5 bigger surface were used in the two upper holders. There were inserted only two testing samples in to the holder, which were of the same summary surface as in the holder with standard samples, but without dielectric element. This element causes inequality between the samples as well as initiate an increase of the sediment amount around them the influence the corrosion. These dielectric elements were placed by bigger separation. Hereby we have reached the possibility of recognition of the influence of pipeline undulation on the corrosion process.

We have monitored and evaluated 30-, 60-, 191-, 364- and 554-day corrosion, while in Galanta, due to the later installed testing facility, the 0.5 and 1.5 year measurements has been shortened by two weeks, i.e. only 177- and 540-day corrosion were evaluated. On big samples 30-, 60- a 364-day corrosion was monitored.

Corrosion velocities evaluated on standard samples are introduced in Tab. 1

Tab. 1: Comparison of corrosion velocities on the trace J-GA-N (standard samples)

Days	**Corrosion velocity** v_t (μm.year^{-1} = 10^{-6}m. year^{-1})				
	Jelka		**Galanta**	**Nitra**	
	2001-03	***95-96***	***2001-03***	***2001-03***	***95-96***
0 – 30	95.4- 92.1	76.67	77 – 79.2	74.5 – 94.5	71.4
0 – 60	59	52.62	60.2	54.5 – 58.6	53.8
0 – 191 /177*	54	26.79	36.9*	31.4	29
0 – 364	44	29.28	29.73	30.63	
0 – 554 /540*	36.2		27.15*	23.26	
30 – 60	**23.5-26.8**	**28.00**	**38.6 – 45.3**	**27.5 – 39.3**	**36.2**
30 – 191/177*	46.5	16.81	26.1 – 28.7*	20.2 – 21.5	21.5
30 – 365	*39.,7*	24.98	*24.4 – 25.,6*	*17.5*	
*30 – 554/540**	*32.,9*		*23.5 – 24.2**	*27.3*	

As we can recognize from the measurement results, the corrosion velocities calculated from the beginning of the test (i.e. from the day zero) were with increasing exposure time of decreasing tendency. The highest corrosion velocities of all monitoring cases were reached at the beginning of the trace in Jelka, while the results from Galanta and Nitra do not vary significantly. On the base of corrosion velocity between the day 30 and 60 (according to STS) water can be categorized into the I. level of (up to 50 μm.year^{-1}) – the water is moderate aggressive, whereof results that not other anti-corrosion measures are required. The highest corrosion velocity between the day 30 and 60 was detected in Galanta.

After visual reviewing of the samples have we recognized that the sediments and corrosion products in Jelka were steadily adhered to the samples, which were all coated with rough, gritty layer of sediment. After cleaning of the samples, their whole surface was disrupted. This disruption was foveolar and nearly coherent, whereby some foveales were osculating. There were less sediments and corrosion material in Galanta, and these were located unevenly, mainly by margins and concentrated in bigger clusters. The sediments were adhered less steadily to the samples and they have formed mostly isolated foveales, sporadically vertical corrugations on the sample surface. The sediments in Nitra were of similar character as these in Jelka, but their layer was thinner, more uneven and les adherent.

The approximate sediment amount in gram/sample by particular experiments is introduced in Tab. 2

Tab. 2: Comparison of corrosive declines and sediments

Place	Days	Standard samples			„Big“ samples	
		U_t (µm)		Sediments	U_t (µm)	Sediments *
		2001–03	*95-96*	*(g/sample)*	*2001–03*	*(g/standard.sample)*
Jelka	30.	7.58 – 7.84	6.30	0.2493 – 0.3054	6.16	0.2224
	60.	9.77	8.63	0.3494	7.26	0.2473
	191.	28.28	13.21	1.0556		
	364.	44	28.9	1.6056	18.68	0.6203
	554.	55		2.2459		
Galanta	30.	6.33 – 7.38		0.3161 – 0.3730	5.16	0.1914
	60.	10.55		0.5174	6.57	0.2633
	177.	17.90		0.8116		
	364.	29.73		1.3275	15.71	0.7232
	540.	40.16		1.698		
Nitra	30.	6.94 – 7.51	5.87	0.2284 – 0.2470	4.86	0.169
	60.	8.81 – 9.96	8.84	0.3433 – 0.3572	6.33	0.2044
	191.	16.44	14.69	0.6092		
	364.	23.26		0.8290	15.07	0.5309
	554.	46.49		1.8262		

* Sediment amount is calculated in regard to the extent of standard sample

By comparison of the corrosion test results from the year 1995-96, when the water was sanitary secured by gaseous Cl_2, with present standard tests with usage of ClO_2 it did not come to significant increase of corrosion declines and velocities. Higher corrosion declines and velocities were currently found in Jelka, by the day 191 and 364. Significant increase of corrosion declines occurred by the day 554 in Nitra, whereby all five samples from this set did not differ much from each other neither by declines nor by the sedimentation amount. Only on this particular place occurred an increase of corrosive decline in the third half-year i.e. between the days 554 and 364 (23.23 µm) against the second half-year i.e. between the days 364 and 191 (6.84 µm). Except of this one measurement, the other corrosion declines (increases of declines in a time) had with growing exposure time decreasing tendency, whereby the dependence between the time of exposure and corrosive declines is not linear, as visible from Fig. 2.

In Tab. 2 there are introduced also the results of corrosion declines measured on big samples and reference amounts of sediments and corrosion products, which are given for comparison with standard samples and their sediments in calculated size to the standard sample. The increases of corrosion declines on big samples had on all three monitored places with exposure time decreasing tendency and they are lower as by standard samples. The ratio between the declines measured on standard samples and on big samples is approximately equal as between the sediment amount of standard samples and big samples. This ratio is growing with the exposure time in Jelka and Galanta, by 30-day measurement the corrosion declines of big samples are 70 - 80% of the standard sample value and by annual monitoring there are only 43 – 53%. The ratio of corrosion declines of standard and big samples is more balanced in Nitra, the corrosion declines of big samples are 65 - 70 % of the standard sample value by 30-day measurement and 64% by annual monitoring.

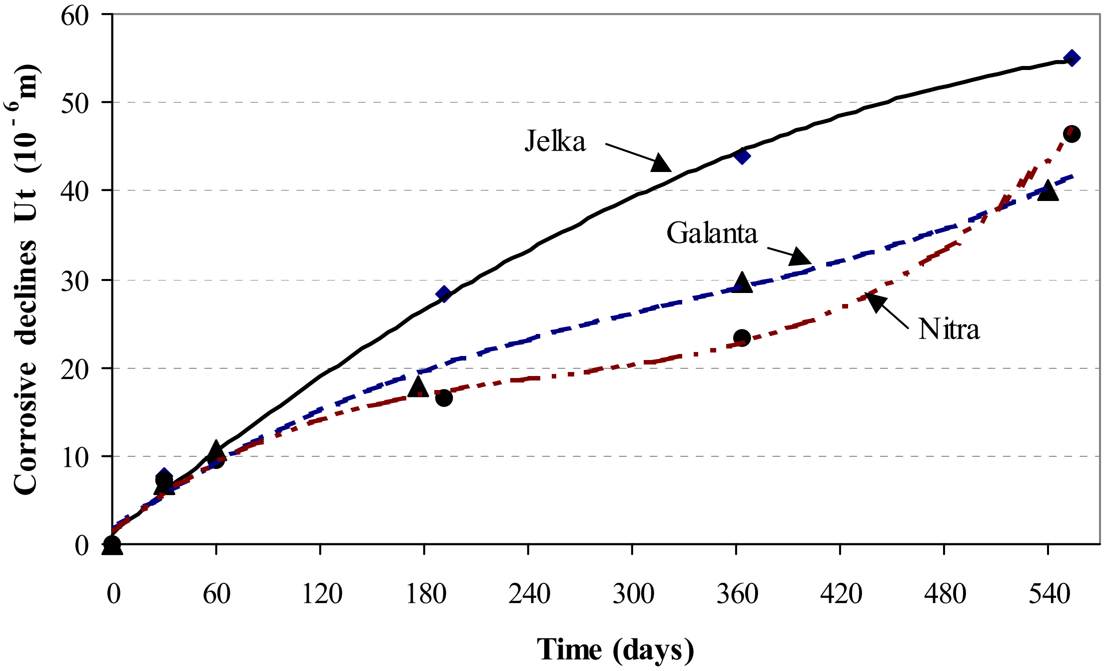

Fig. 2 : The process of corrosive declines in Jelka, Galanta and Nitra in the time of sample exposure

Comparing the sediment amount (g/sample) and the corrosive declines (g/sample) is possible to state, that the increasing sediment amount intensifies corrosive declines and the corrosion self. As we can see on Fig. 3 and especially Fig. 4, where samples from all (standard) measurements are ranged by the sediment amount, the corrosive declines increase slower than sediments. The sediment amount of particular samples was changing not only by particular experiments, but also within the single experiment. The smallest amount of sediment was mostly found on lowest sample of frames, what is documented in Fig. 3, where all corrosive declines and sediments on particular samples from the experiments from Jelka are presented. They are ranged by samples in frame, whereby the first value from the left refers to the highest placed sample of the frame.

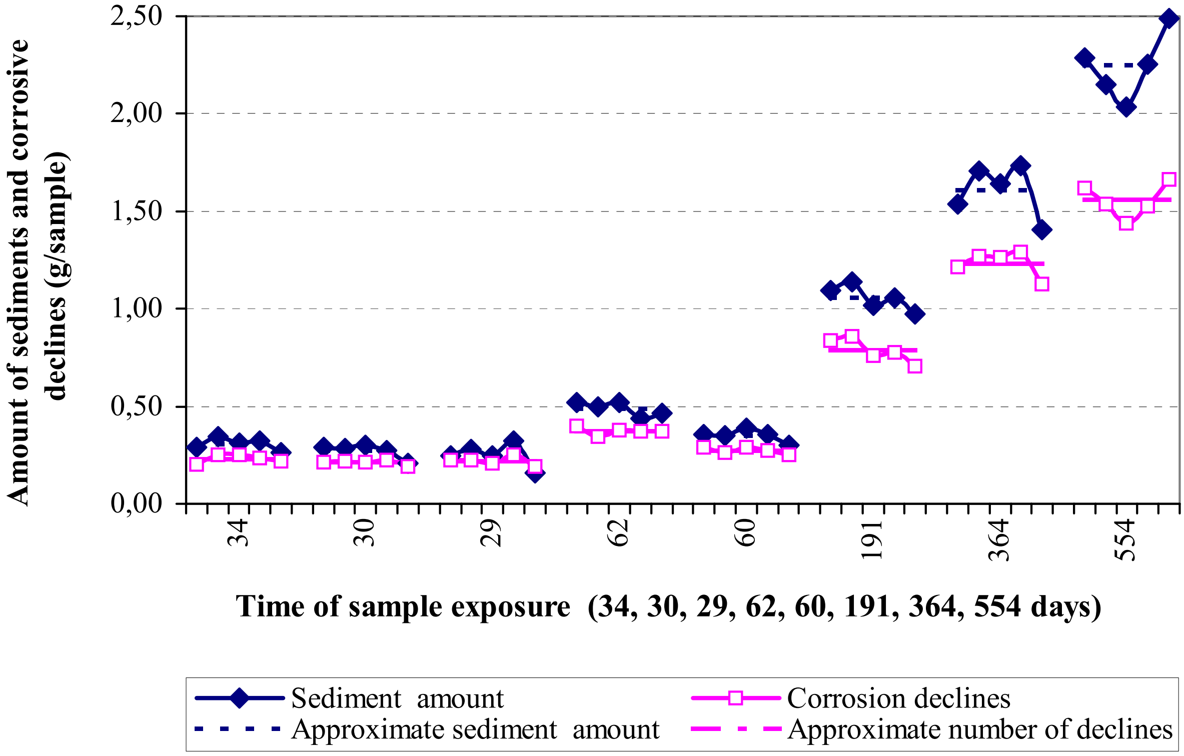

Fig. 3: Comparison of sediment amount and corrosive declines of particular samples in Jelka within their exposure time

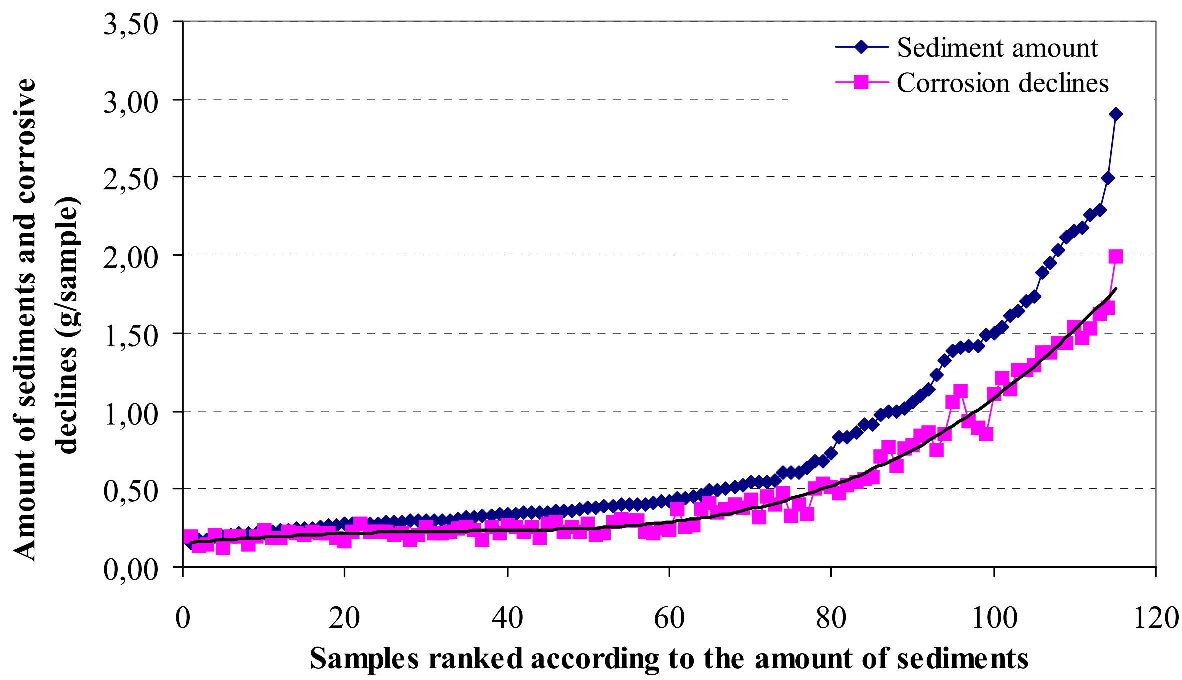

Fig. 4: Comparison of the sediment amount and corrosion declines of particular samples within their exposure times ranged by sediment amount.

3 Conclusions

We can summarize, that by comparison of present experiments with experiments from the years 1995-96 any significant changes of corrosive velocities did not occur in spite of the change of the sanitary water securing. The visual examination of samples implies the change of corrosion type, while the corrosion in Nitra and Jelka has been of foveolar type before, and the foveoles were deeper and rare – and the residual surface unfaulted, currently is almost the whole surface especially in Jelka perturbed with foveaoles, which are often interfered.

On the base of the results of 30-, 60- and 364-day measurements the corrosion occurs smaller by big samples than by standard samples. The corrosion declines, velocities and amounts along the trace are more stabile, they change less significantly as by standard samples. The decrease of corrosion declines by big samples to 43-64% of standard sample values by 364-day measurements implies a possibility of influencing the corrosion declines by pipeline undulation.

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Analysis of Sustainable Water Use in Croatia

Dragutin Geres

Croatian Waters, Zagreb, Croatia, and University of Osijek, Faculty of Civil Engineering, Osijek, Croatia; e-mail: dgeres@voda.hr

Abstract

The paper highlights the status of the water use in Croatia. The major findings indicate that water is key to development in Croatia. The goal of paper is to supplement the data for future research in relation to sustainable water use. The problem analysis of water use is based on the application of the European Environment Agency framework for environment assessment: driving forces, pressures, states, impact and responses. The basic source for water use in Croatia is surface water, followed by groundwater and marginal quantities of desalinized water.In Croatia, 38 per cent of water is used for urban purposes, 60 per cent in industry and energetic and 2 per cent in agriculture. The quantity of water used for power production is not included in these indicators, for which purpose Croatia uses 33.5×10^6 cum of water per year. Water uses in Croatia varies greatly, depending on natural conditions, culture, and economic conditions. Water resources can meet the water requirement of household, industry, agriculture, power production as well as those needed to resolve problems of the environment and ecosystems.

1 Introduction

The paper analyses the use of water in Croatia. Water resources are affected by numerous factors, resulting from agriculture and industry, urban areas, households, and tourism. Other driving forces which influence the water resources are related to natural variability of rainfall and climate changes. Extreme hydrological events, such as floods and droughts create additional stress in the water supply sector, which are important from the health point of view, and to the issues of the ecosystem status. The problem of excessive use of water resources is extremely complex, not from only the hydrological standpoint, but also regarding the socio-economic and political conditions. The solutions must meet the environmental requirements and be socially and politically feasible. The objective of the paper is to analyse the conditions for improvement of water use, support to sustainable water use and protection of resources. Therefore, it is suggested that the research and collection of data concentrate on improving the present status of information.

2 Elements of environmental assessment and definitions

The study is based on the interdisciplinary process of identification, analysis, and assessment of relevant natural and human processes and their interaction which determines the present and future status and quality of the environment and resources in the corresponding spatial and temporal scale [1]. Agriculture, population and its growth, urbanization and industry are the fundamental *driving forces* affecting the hydrological cycle. They result in *pressures* on water resources, such as those related to water abstraction or different uses: municipal use, industry, agriculture, etc. Climate changes are also considered. The *status* of water resources is assessed by their quantity and quality. The *influences* are described through general information and regional examples. The estimate of these influences provides information for setting of goals for future research and policies. Potential social *responses* are described by regulatory instruments, control of water supply and water requirements, financial instruments, and infrastructure. These principles are shown in Table 1.

Table 1. Elements of environmental assesment

Driving forces →	Pressures →	Status →	Influences →
Agriculture	Water abstraction	Runoff	Water stress
Population	Surface water	Renewable	Drought
Industry	Ground water	Resources	Degradation
Climate	Climate changes	Quality	• Quantity
Rainfall			• Quality
Temperature			Ecological status
↑	↑		↓
Policy	Responses ←		Goal Setting
Laws	Infrastructure		1.
Directives	Supply and demand		2.
Etc.	Control		.
	Costs		.
			n.

3 Factors affecting the water resources

3.1 Population and urbanization

The changes of population number, distribution, and density, are the key factors influencing the assessment of water demand. The number of population in Croatia is given according to census, as follows [2]:

1.	1951	3,876,300	4.	1981	4,601,459
2.	1961	4,159,800	5.	1991	4,784,267
3.	1971	4,426,200	6.	2001	4,381,352

In the Republic of Croatia, 54 percent of the population live in urban centres. Out of this, 16 percent live in towns with the population over 500 thousand, 12 percent in towns with the population of 100-500 thousand, 13 percent in regional centres with the population of 30-100 thousand, 13 percent in minor towns with the population of 7-30 thousand, 27 percent in local centres with the population of 2-7 thousand, and 19 percent in communities with the population under 2,000 (census 1991). Table 2 gives the data on Croatia, EU member countries, and Europe as a continent [3].

Table 2. Data on Croatia, EU member countries and Europe

Country	Area (000 sq.km)	Population, 1996 (000 inhabitants)	Population density (per sq.km)
1. Croatia (1991)	56.5	4,784	84.6
2. EU member countries (15)	3,240.3	372,343	average 115
3. Europe, continent	10,200	680,000	67

Water use in households and small-scale industry shows large variation from country to country. Consumption measured as the quantity and percentage of overall water use is also growing in the majority of countries in the period from 1980 to 1995. It is assumed that the population will grow, with changes in the lifestyle, that the water tariffs will increase, and that the society will show more concern for water. This should result in more economical water use.

3.2 Industry

Industrial activities result in pressures on the environment. These may be direct pressures, such as emission of pollutants, production of hazardous waste, and consumption of natural resources. Indirect pressures result from consumption and use of industrial products. In Croatia, the added value of industry in gross domestic product – GDP is around 20 percent. In EU member countries, it is about 30 percent, and in Europe 29 percent. Comparing of data on water use in industry is very difficult, due to inconsistency of data. Water use in industry, excluding cooling water, is about 10 percent, and water used for cooling and power production is 32 percent of all abstracted water. In many European countries water requirements in industry are decreasing. This is, first of all, the effect of economic recession, but also of technological improvements [4]. The growth of industrial production stagnates and decreases in relations to other sectors of economy.

3.3 Agriculture

The agricultural sector creates pressures on water resources by water abstraction and by potential pollution of the resources by fertilizers and pesticides. Agriculture is therefore an important driving force in sustainable water management. In Croatia, agriculture contributes 7 percent to GDP (1998 data). In EU member countries, the contribution of agriculture is 2.3 percent on average, and in Europe 6 percent [5]. The total agricultural population in Croatia has been decreasing at the rate of 4.9 percent since 1961. The data on irrigated areas are given in Table 3. A variable role of irrigation in European agriculture is noticed depending, first of all, on climatic conditions.

Table 3. Agricultural and irigated areas

Country	Total area (000 sq.km)	Total agr. area (000 ha)	Percentage of agr. area (%)	Irrigated area (000 ha)	Percentage of irr. area (%)
1. Croatia	56.54	3,208	56.7	3.1	0.1
2. EU countries	3,240.3	87,903	27	11,354	12.9
3. Total Europe	10,200	135,945	13	16,717	12.3

Water use in European agriculture amounts to about 30 percent of abstracted water and about 55 percent of all consumed water.

3.4 Tourism

Tourism, in forty years of development, has become very important in national economies, with the participation of about 1.2 percent (average) of GDP in OECD countries. In Croatia, the share of tourism in GDP in 1996 was 2.53 percent, and in 1998, 2.47 percent [2]. Tourism results in seasonal growth of population, often in periods with minimum or low recharging of

water resources. This causes pressure on water resources, through increased water consumption. Another property of tourism is considerable seasonal variation, with maximum in holiday times, and minimum in the remaining part of the year. Water consumption in tourism is approximately double of that of the local population. Croatia had the maximum number of tourists in 1987 and 1988 (10.4 million arrivals per annum). In 1990, it was 8.5 million; in 1995, 2.4 million; in 1998, 5.5 million; and in 1999, 4.8 million of tourist arrivals. The number of tourists in EU is the highest in France (61 million arrivals per annum), followed by Italy (52 million), and Spain (43 million).
Water consumption issues are followed by problems of waste water. The largest part of water is not consumed, but returns after use as waste water, treated or untreated, into the sea or rivers. If water is not treated, the problems of environmental pollution occur.

4 Pressures on water resources

4.1 Water abstraction

Analysing and comparing the data on water abstraction and water use, one may notice that the data from different sources often do not match. This statement is equally valid for Croatia and for other European countries. In most cases, the differences are caused by different definitions of the analysed case. For example, the definition of water requirements for industry may range from water quantity in the industrial sector to inclusion, along with industrial requirements, water use for cooling of power plants and water for power generation. Table 4 presents the data on total abstracted water quantities. The basic source for water abstraction is surface water (about 75 percent of all abstracted water for all purposes). Ground water is used 25 percent, with only minor quantities of water obtained by desalination of sea/brackish water, and recycling of treated waste water [6].

Table 4. Total abstracted water quantities

Country	Population (000)	Total abstracted water (mill.cu.m./annum) DOBRIŠ 1986/83	OECD 1995	ETC-IW 1995	Abstracted water per capita (cu.m./annum) from ETC-IW data
1. Croatia Source: SLJH-2000	4,784	1,403 (for 1998)	-	-	293
2. 15 EU countries	372,752	251,938	226,772	245,761	659
3. Europe, continent	680,000	480,000	-	-	700 (Dobriš)

In countries with sufficient number of aquifers, over 75 percent of water for public water supply is abstracted from ground water. Ground water, in general, is of better quality than surface water, and ground water resources are exploited to a higher extent than surface water. Such practice leads to excessive abstraction of ground water resources and to lowering of ground water level. This in turn leads to degradation of the sources of surface watercourses, destruction of wetlands, and in the case of coastal waters, penetration of salt water into aquifers [7].

Table 5. Share of ground water in total abstracted water and in public water supply

Country	Abrstracted ground water (%)	Public water supply Ground water (%)	Surface water (%)
1. Croatia	42	86	14
2. 15 EU countries	25	62	38

Table 6. Water use by sectors

Country	Total abstracted water (million cu.m/annum)	Urban consumption (%)	Industry (%)	Agriculture (%)	Power (%)
1. Croatia (SLJH 2000)	1,403	38	60	2	-
2. 15 EU countries	251,938	14	18	30	38
3. Europe, continent	480,00	19	53	28	-

Abstracted water quantities in EU countries are used for municipal purposes in 14 percent of cases, in agriculture (30 percent), industry (18 percent, cooling water excluded), as cooling water and in power generation (38 percent). The falling trend may be attributed to changes in management strategy, which turns towards water demand management, reduction of leakages, more efficient water use, and use of recycled water. The share of municipal water requirements in all abstracted quantities in Croatia is 38 percent, the average for 15 EU countries being 14.1 percent. In a major number of countries, municipal water requirements include, along with household demands, industrial requirements, agriculture, small-scale industry, public services, and recreation requirements. It is often impossible to obtain separate statistic data. The share of household demand in total requirements is between 30 and 60 percent. In Croatia, in 1998, the share of households is 58 percent, and other activities 42 percent of municipal water demands [8]. Water use in households and small-scale industries in litres per capita per day shows great differences. The quantities range from 132 to 250 l/cap/day in EU, in Eastern Europe 150 to 300 l/cap/day. In Croatia, the consumption in 1998 was 240 l/cap/day.

For industrial use, excluding cooling water, in 15 EU countries, about 10.4 percent of all abstracted water is consumed. According to data from SLJH-2000 the quantity for Croatia is 60 percent. This figure is subject to dispute. Probably this quantity includes cooling water as well. Observing only water quantity provided from public water supply, the figure of 13.1 percent of all abstracted water is obtained.

4.2 Climate changes

Estimates of climate changes indicate that air temperatures will increase by 1°C to 3.5°C in 2010. which, with increased rainfall in Northern Europe and its decrease in Southern Europe, may lead to reduction of renewable water resources in Southern Europe. Increased temperature may result in earlier snow melting, causing winter runoff and reduced runoff in spring and summer. The variation of drought risk and intensity is the most serious adverse impact of climate change on water resources. Climate changes might have a considerable impact on the flood regime [9].

5 Status of water resources

5.1 Water resources

Fresh water resources are continuously recharged by natural processes in the hydrological cycle. Precipitation on land surface on Earth provides more than 110, 000 cu. km of water per annum. Approximately 65 percent of precipitation returns to the atmosphere through evapotranspiration. The remaining part is runoff. This part recharges the aquifers and supplies water to watercourses and lakes. The average runoff determined in Europe is about 3, 100 cu. km per annum or, as an equivalent, 4, 560 cu. m per capita per annum. The quantities of available water in Europe vary depending on annual runoff, ranging from over 3, 000 mm in a part of Norway to less than 25 mm in southeastern Spain. Transboundary runoff makes a considerable part of the resources in many countries. In Hungary, for instance, it is 95 percent of all resources. The quantity of renewable water resources is shown in Table 7. Total renewable resources, available to a country, may be perceived as resources from its own territory (endogenous resources) increased by water coming from upstream countries (exogenous resources) [10].

Table 7. Quantity of renewable endogenous and exogenous water resources

Country	Renewable resources (million cu. m/annum)	Renewable resources per capita (cu. m/annum)
1. Croatia	169,000	35,300
2. 15 EU countries	1,452,000	3,900
3. Europe, continent	3,000,000	3,560

The highest water demand is present in areas with high concentration of population. Water demand n Europe has grown from 100 cu. km per annum in 1950, through 551 cu. km in 1990, to 660 cu. km per annum in 2000.

5.2 Quality of water resources

Analyses of the volume of abstracted water and the volume of resource often neglect the fact that water can meet human needs, as well as the environmental needs, only if its quality corresponds to the intended purpose. As traditionally in the majority of countries attention is paid mainly to water quantity issues, the water quality issue will become increasingly important for planning and management of water resources and infrastructure [9]. The major types of use and functions of water resources include: drinking water supply, water for bathing and recreation, water for industry, fishery, irrigation, livestock farming, and environmental purposes. The required water quality varies depending on the use or purpose. Table 8 gives some indices of water quality depending on the use and purpose.

Table 8. Relations between water quality indices and the use and purpose+

Indicators	Ecologic function	Use				
		Drinking water	Bathing and recreation	Irrig-ation	Livest-ock	Fish-ponds
Organic matter	+	+				+
Nitrates	+	+			+	+
Phosphorus	+					+
Suspensions	+	+	+			+
Colour		+				
Temperature	+					
Mineralization		+		+	+	+
Micro-organisms		+	+	+	+	
Phytoplankton	+	+				+
Inorganic micro-pollutants	+	+		+	+	+
Pesticides	+	+		+	+	+
Organic micro-pollutants	+	+				

Marks: +: function and use depending on indicator
empty: function and use not depending on indicator

6 Impact on water resources

6.1 Pressures causing water stress

We speak about water stress in cases when abstracted water quantities are in discrepancy with available water quantities in a given area. The index of water availability and pressure on water resources is the ratio between the quantity of abstracted water and total renewable resources.

From the data in Chapters 4 and 5, the following ratios were calculated:
- For Croatia, abstracted water quantities in relation to total resources are 0.9 percent; in relation to its own resources 3.3 percent; together with water used for power generation and cooling the result is 19.8 percents of the total resources.
- For Europe (average): total abstracted water makes 15 percent of renewable resources [11].

Potentially, all countries have sufficient resources to cover their national water demand. The highest water demand is concentrated in areas with high population density. Frequently, urban water demands are higher than the available water resources. Seasonal or multi-annual variations of available water resources may sometimes lead to water stress in areas where, in the long run, there are sufficient water resources. Water resource planners often base their decisions on water use on resources which may be expected in periods of drought or low discharges in rivers. A recommendable indicator for such decisions is the 90 percent discharge (Q90), i.e. the quantity of water that may be used 328 days per annum (90 percent of the time).

6.2 Drought

The water demand in Europe has grown from 550 cu. km in 1990 to 660 cu. km in 1999. The occurrence of droughts shows the vulnerability of water resources in relation to variations in

the meteorological and hydrological cycle. The expected growth of water demands will result in a conflict between human needs (commercial, social, and political) and environmental requirements. Assessment of droughts is a complex problem. There is no consensus on the definition of drought, except in general terms. The most frequent definition is: *The basic property of drought is reduction of available water in a given time and in a given area*. The drought may be described by a number of indices which may be classified in two groups:
1. environmental indices: hydrometeorological and hydrological indices, directly affecting the hydrological cycle;
2. water resource indices: used to measure the impact of drought on the use of water resources, e.g. impact on water supply for municipal purposes or agriculture, impact on aquifer recharging, water abstraction, impact on water levels (surface and ground), impact on fishery, recreation, etc.

The drought period in Europe lasted from 1988 to 1992, when in most countries the registered rainfall and runoff were lower than multi-annual averages. The impact of drought depends on the combination of the hydrological conditions and pressures on water resources. The highest drought in the early 90's in Europe occurred in areas with highest pressures on resources. These are not necessarily the areas with the highest hydrological drought.

6.3 Impact of excessive abstraction of ground and surface water

Excessive abstraction of ground water is defined as the abstraction higher than the quantity of recharge – renewal, resulting in lowering of the ground water level. The effects of excessive ground water abstraction are lower discharges in watercourses, endangered wetlands, and penetration of saltwater into aquifers. Excessive abstraction of surface and/or ground water may result in serious consequences for the related terrestrial and aquatic ecosystems [12]. Water abstraction modifies the natural hydrological regime and discharges in surface waters (rivers, lakes, wetlands). This causes direct impact on the ecological status of the aquatic ecosystem. Low discharges in watercourses may result in further problems connected with pollution, such as lowering of the dilution capacity, reduction of the oxygen content, and increasing of nitrate and phosphate concentrations which may cause eutrophication problems.

7 Social response and water demand management

Potential social responses related to water resources are described by regulatory instruments: national, EU, and international, and through two major aspects of sustainable water use: management of water demand and water supply infrastructure. The paper does not describe the regulatory – legislative elements.

7.1 Water demand management

The goal of sustainable water management is the balance between abstraction of water quantities for public water supply, industrial and agricultural purposes, quantities for recreational and environmental requirements, quantity of waste water, and impact on dispersed water sources. The principle of control is applied, based on permits/concessions, in order to achieve balance between different water demands. Economic instruments are being introduced, such as water abstraction fees and the mechanism of water tariffs for users. When the economic instruments are applied to public water supply, they have the strongest impact on the poorest segments of the society. In order to ensure revenues, the municipal companies must increase the price in case the water consumption is reduced. The general benefit of water consumers to save money by saving water is therefore small. However, in this case it is possible to save the costs for infrastructure. In Croatia, water consumption per household is 166.5 cu. m per annum, and it is paid HRK716 per annum, at the overall price of water from public water supply of HRK4.3 per cu. m. In Euros, this is €99 per annum per household. The price of drinking water is different in European countries. It depends on the area, type of service, etc. In Western Europe, the price ranges from €52 per annum per household in Rome to €287 per annum per household in Brussels. In the countries of Central Europe, the water price is lower, ranging from €20 per annum per household in Bucharest to €59 per annum per household in Prague. If the annual price water per household is considered in relation to gross domestic product (per capita), then the price in Bucharest is 3.5 percent; in Prague 2.3 percent; in Portugal 2.2 percent; etc. In Croatia, the price of water is 1.8 percent of GDP per capita.

7.2 Infrastructure

The technical condition of water supply mains and distribution networks has a direct impact on total abstracted water quantity [13]. The efficiency of the network, defined as the ratio of water quantity delivered to the user and abstracted water is: in Croatia 67 percent, in France about 75 percent, in Spain about 80 percent, in Italy about 74 percent, in Austria about 88 percent, in the Czech republic about 70 percent, etc. The estimated leakages from water supply networks (excluding household connections) are: in the United Kingdom 8.4 cu. m/km/day, in Germany 3.7 cu. m/km/day, in Croatia 14.7 cu. m/km/day, etc. Leakages from water supply networks can be reduced in different ways, such as: repair of visible leakages, establishing of leakage control zones, detection and repair of leakages invisible from ground surface, telemetry of flow in the pipes, reduction of pressure, replacement of old pipelines, detection and repair of household connections, reduction of overflow in storages, etc.

Generally it may be said that water consumption grows when there is enough water at a low price. These two concepts are slowly disappearing, particularly in cases of increased resource pollution, drought, and increasing of water tariffs. The purpose of storage reservoirs and flood storages is to abridge uneven distribution of natural water resources in time. Regulation may be seasonal or multi-annual. In Croatia there are 58 storage reservoirs with the total gross capacity of 310 million cu. m (excluding flood storages and multipurpose storages). In the European Union, there are about 3500 major storage reservoirs with the total gross capacity of about 150, 000 million cu. m. The countries with the most storage reservoirs are Spain, Norway, Sweden, etc. In Spain, there are 849 storage reservoirs, in France 521, and in the United Kingdom 517 storage reservoirs.

Recycling of waste water is growing in EU countries. Desalination of sea or brackish water is in the initial phase. This procedure is considered in the areas without fresh water. The total quantity of desalinated water in Croatia and in Europe is very small.

8 Conclusions

1. In Croatia, water is used for urban purposes 38 percent, in industry 60 percent, in agriculture 2 percent, power generation not included. In Europe, abstracted water is used as follows: for urban purposes 14 percent, industry 10 percent, agriculture 30 percent, for cooling and power generation 32 percent, and for other purposes 14 percent.
2. In the 80's and 90's, water consumption in industry is reducing as a consequence of economic recession. Water consumption is reduced also due to technological improvements of equipment.

3. Agriculture is one of the largest driving forces and pressures affecting the water resources. In Europe, about 30 percent of abstracted water is used in agriculture, and in Croatia only 2 percent, because irrigation is not developed.
4. Water consumption in tourism is typically seasonal, coinciding with limited water resources. Water consumption by tourists is approximately double of that of regular users.
5. Forecasts of climate changes indicate the increase of temperatures by 1°C to 3.5°C and reduction of rainfall by 10 percent. This could result in reduction of renewable water resources by 40 to 70 percent.
6. The water stress is generally connected with excessive water abstraction in relation to the available resources. Excessive use of surface and ground water has a serious impact on related terrestrial and aquatic ecosystems.
7. Economic instruments are essential for achieving of sustainable water management. The price at which water is sold generally does not reflect the real costs, and the price is not the same for all users. The price of water in Croatia for one household is €99 per annum, or 1.8 percent of GDP per capita. In Europe the prices range from €52 to €287 per annum per household.
8. The efficiency of the water supply network has a direct effect on total abstracted water quantities. In many countries, water leakages from the network are very high. In Croatia, leakage is 14.7 cu. m/km/day, in the United Kingdom 8.4 cu. m/km/day, and in Germany 3.7 cu. m/km/day.

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Safety of Protective Embankments

Danka Grambličková & Emília Bednárová

Slovak Technical University, Faculty of Civil Engineering, STU, Radlinského 11, 813 68 Bratislava, Slovakia, gram@svf.stuba.sk, bednarov@svf.stuba.sk

Abstract
Protective dam's safety is influenced by various factors. In addition to the uncertainty of hydrologic basis the geology is one of the most decisive factors. The paper deals with the assumption of efficiency of slurry wall in the underground of protective dam from the point of view of its interaction with the environment

There are many factors influencing the safety of dams and protective embankments. Disregarding potential danger of overflowing of embankments (e.g. in case of unreliable hydrological data), the most frequent causes of dam damages and failures are seepages. They result usually from development of preferred routes, due to which failure of filtration stability, breaking of covering layers due to uplift, or dam stability breaking occur. These damages may be caused either by unreliable geological data, or by inadequate site development, construction faults and errors, and by operation itself. Last but not least the impact of time, bringing about alteration and deterioration of materials in the body or in the subsoil of embankments is to be mentioned.

The first important stage, determining the embankment's safety, is the engineering-geological investigation. The geological structure of the subsoil of protective embankments in Slovak conditions is rather diversified. While in the area of the East Slovakia Lowland impermeable silty clays or clay loams prevail, with local admixtures of gravel and sandy soils, in the area of the Morava river catchment on our territory sandy soils prevail, and in the Danube Lowland gravel soils are most abundant. However, generally the environment may be considered as heterogeneous, or even anisotropic.

Since the protective embankments are mostly constructed as homogenous (clayey or loamy) structures, the geological structure of the subsoil is the decisive factor of the type of the safety risk. Simplified schemes of the geological structure of embankments subsoil and their impact on their safety are presented in Figure1. For instance, if the embankment subsoil consists of soils with low permeability (Fig.1a), the risk of its safety decreasing with regard to seepages is unlikely, because for development of the seepage curve within the embankment body the flood discharges must have been extremely long-lasting. However, in case of more permeable soils in the embankment subsoil (Fig.1b) with permeability range 10^{-5} $m.s^{-1}$, the negative effect of seepage waters may be demonstrated already after about 26 days.

With increasing subsoil permeability also the risk of piping downstream of the dam toe in the area of its base joint is increasing (Fig.1c). This risk may occur also under conditions of low permeable subsoil - locally, if preferred route occurs (Fig.1d). Combination of permeable sub-soil with covering layer having low permeability (Fig.1e), characteristic for the areas of the left side Danube embankments, is exposed to the risk of breaking the covering layer due to uplift. However, such stress occurrence may not by exclude even in conditions existing in East Slovakia Lowland - for instance in case of occurrence of a local continuous permeable soil layer (Fig.1f). In case of covering layer weakening (Fig.1g) incidence of outflows cannot be eliminated, thus the safety of protective embankments is negatively influenced.

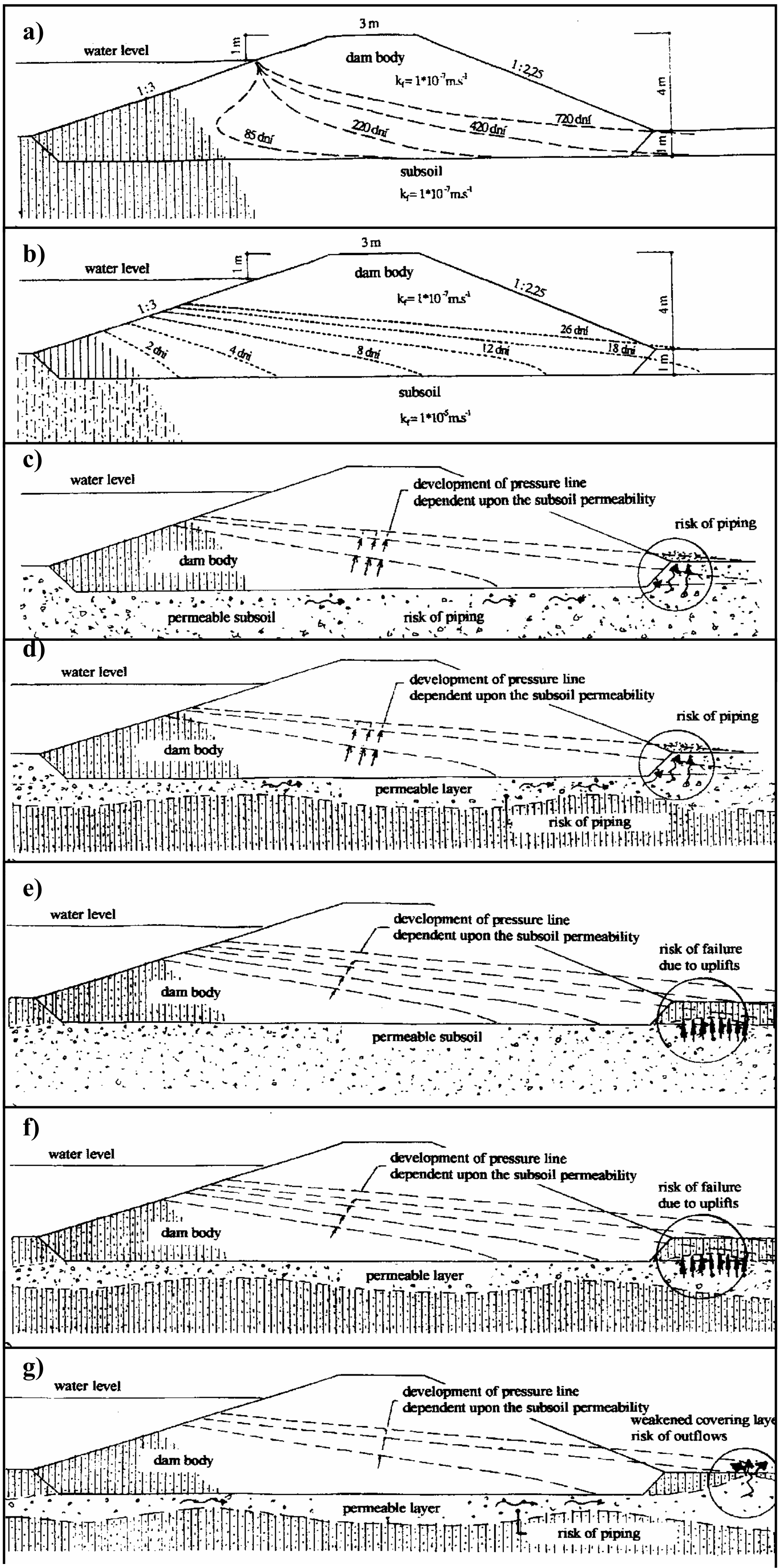

Fig.1: Schemes of geological structure of the subsoil and its influence on embankments safety

Potential variants of embankments subsoil structure in our conditions have been presented in the above schemes in a simplified form, demonstrating, that even a minimum unsafety (e.g. existence of a continuous permeable soil layer), may result in increasing negative impact either on the embankment stability, or on covering layer stability, or eventually on the filtration stability of subsoil strata.

The occurrence of permeable subsoils represents the largest risk in the safety of protection embankments. The change in the subsoil filtration regime is almost instantaneous in the time of extreme flood flow. The Danube river flood protection embankments downstream from Bratislava can be given as an example. Tha gravel soil layers with permeability of around 10^{-3} m.s^{-1} (locally even larger – $8{,}8.10^{-2}$) can be found here, below the cover layers not thicker than 3 m. Numerous seepages and waterlogging sites occurred along the embankments during the flood in August 2002. These effects indicated break-down of cover layers and suffosion of fine sandy particles. The assumption of threatened stability of the protection embankment, as well as optimum proposal of remedial measures, were evaluated with numerical method of finite elements [1]. The solution comes from the differential equation:

$$\frac{\partial}{\partial x}\left(b k_x \frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(b k_y \frac{\partial h}{\partial y}\right) = (S_v + b\, S_s)\ \frac{\partial h}{\partial t} - Q \qquad (1)$$

describing the unsteady groundwater and seepage water flow in the plain within saturated and unsaturated zone, h being the piezometric head, (m), b - thickness of the water-beating layer (m), k_x, k_y - filtration coefficients in the direction of the axis x, y /m.s^{-1}/, t - time /s/, S_v - coefficient of the water storage capacity of free water table, S_s - specific storage capacity /m^{-1}/, Q - discharge or runoff /m^3.s^{-1}/. Q is positive for inflow and negative for runoff from the concerned area.

Unequivocalness of the equation solution is determined by boundary conditions on the border of the investigated area. In case of unsteady flow it is necessary to extend them with initial conditions. Programme SEFTRANS [2] was applied in the solution, enabling consideration of three types of boundary conditions on the border of the concerned area, i.e. Dirichlet, Neuman, and Cauchy conditions. Initial boundary conditions for unsteady flow may be started from the programme set, produced in advance during computations, or order them directly in task simulation.

The analysis [3] of filtration flow below the embankment under extreme hydrodynamic stress, which simulated 2002 flood, confirmed endangered stability of embankment downstream toe and cover layers (see Fig.2). The exceedance of critical pressure horizon close to the embankment downstream toe by about 1,05-1,44 m was confirmed for every cover layer

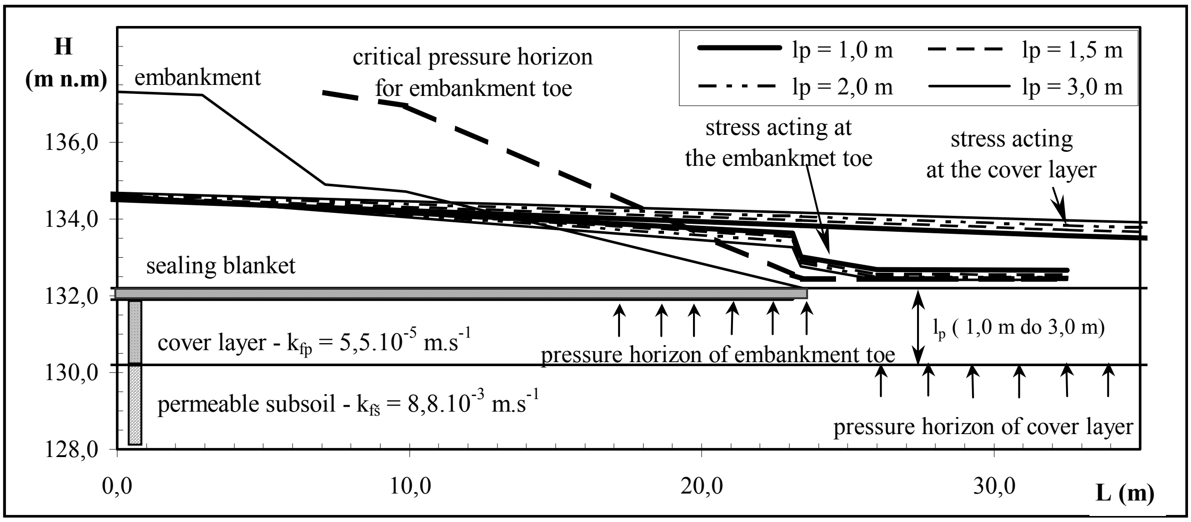

Fig. 2: Stress acting at the embankment toe and cover layers

thickness. The pressure line horizon of cover layers in the range 1,79-1,99 m above the terrain indicated danger of these layers collapse in the sites with decreased thickness (less than 2 m) in the distance of about 30 m from the embankment toe. In the area of thin cover layers, velocities exceeded critical values (see Fig.3) in the initial stage of modeled extreme stress, close to footing bottom of the embankment. It indicates endangered stability of embankment, caused with filtration stability break, with respect to suffosion prone Danube gravels.

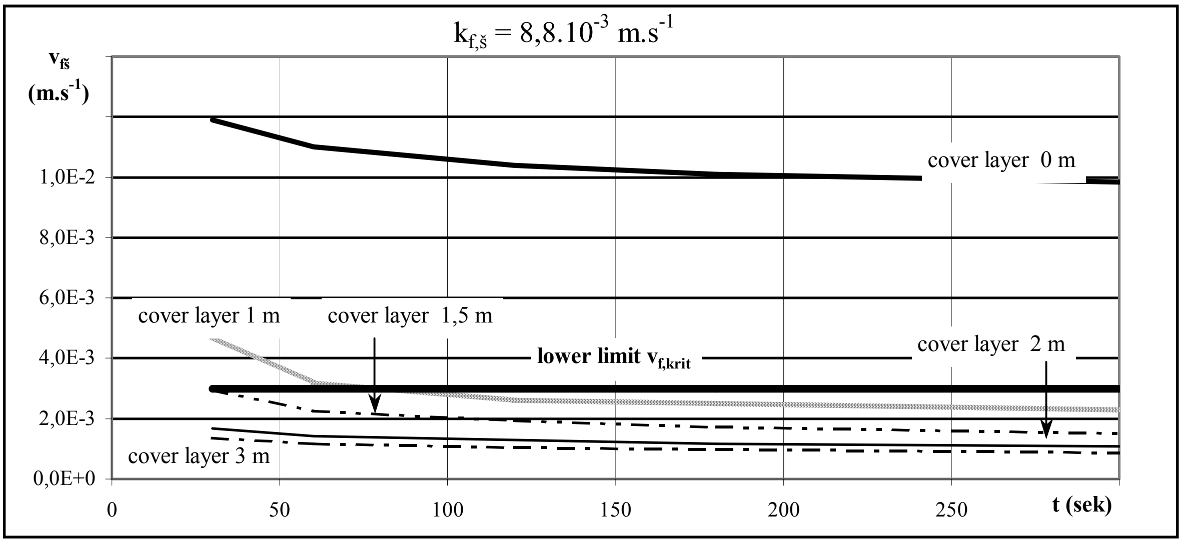

Fig.3: Development of filtration velocities below the embankment *for various thickness of cover layers*

The sites without cover layers represent a different situation, from the viewpoint of pressure conditions. The pressure horizon effects only the sealing blanket. The water starts to flow out at the terrain almost immediately close to embankment toe, thanks to intensive filtration flow below the footing bottom of embankment (subsoil permeability in the order of 10^{-3} – 10^{-2} m.s^{-1}). At the same time, this fact reduces stress acting at the embankment toe below the critical value (see Fig.4). The danger of embankment break is therefore no longer topical because of excessive pressure in the subsoil, but because of intensive flow below the footing bottom and close to embankment toe. The values of filtration velocity exceed critical velocities substantially here (see Fig.3), thus creating conditions for the suffosion development. There is a risk of contact erosion – entrainment of fine soil particles in the contact with sealing blanket, or the risk of surface suffosion – washing the fine particles out at the terrain, close to the embankment downstream toe.

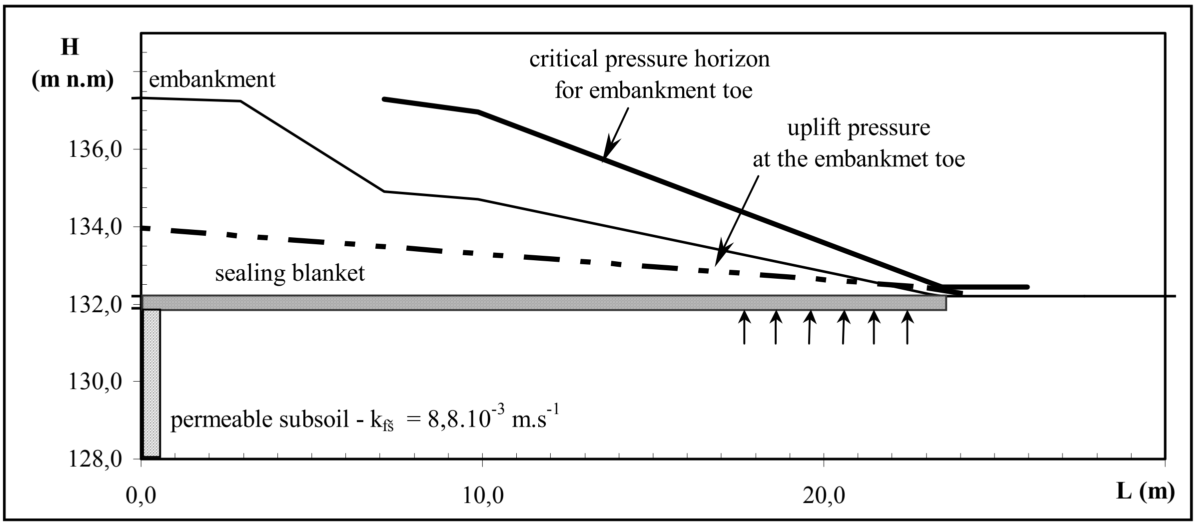

Fig.4: Comparison of supposed uplift pressure and critical pressure horizon acting at the embankment *toe – without cover layer*

The results of analysis confirmed a need of remedial measures. The choice of measures was limited with local conditions, which determined location of possible measure in the distance of maximum 10 m from the embankment toe, as well as with a requirement to influence groundwater regime in a minimum extent under normal and minimum water level in the Danube. The mentioned conditions therefore excluded realization of fixed cut-off wall.
The resulting proposal of remedial measures had to take into account mentioned factors and to eliminate all negative effects, which endanger protection embankment safety. The proposal of combined remedial measures consisted from (Fig.5):

- Stabilizing additional fill , which aims at the improvement of the embankment downstream toe stability against the lift and breakdown and stability of cover layers. This fill also protects embankment toe against the development of internal suffosion.
- Drainage system of horizontal drains, combined locally with vertical drains. This system aims at the reduction of pressure in the subsoil to the required values, thus eliminating danger of cover layers breakdown.
- Hung-up cut-off wall, which reduces filtration velocities below the embankment into acceptable values and re-distribute maximum value $v_{f,max}$ from the area of embankment subsoil further downstream. The impact of such cut-off wall at the water level regime in the given geological conditions is minimum under normal load conditions.

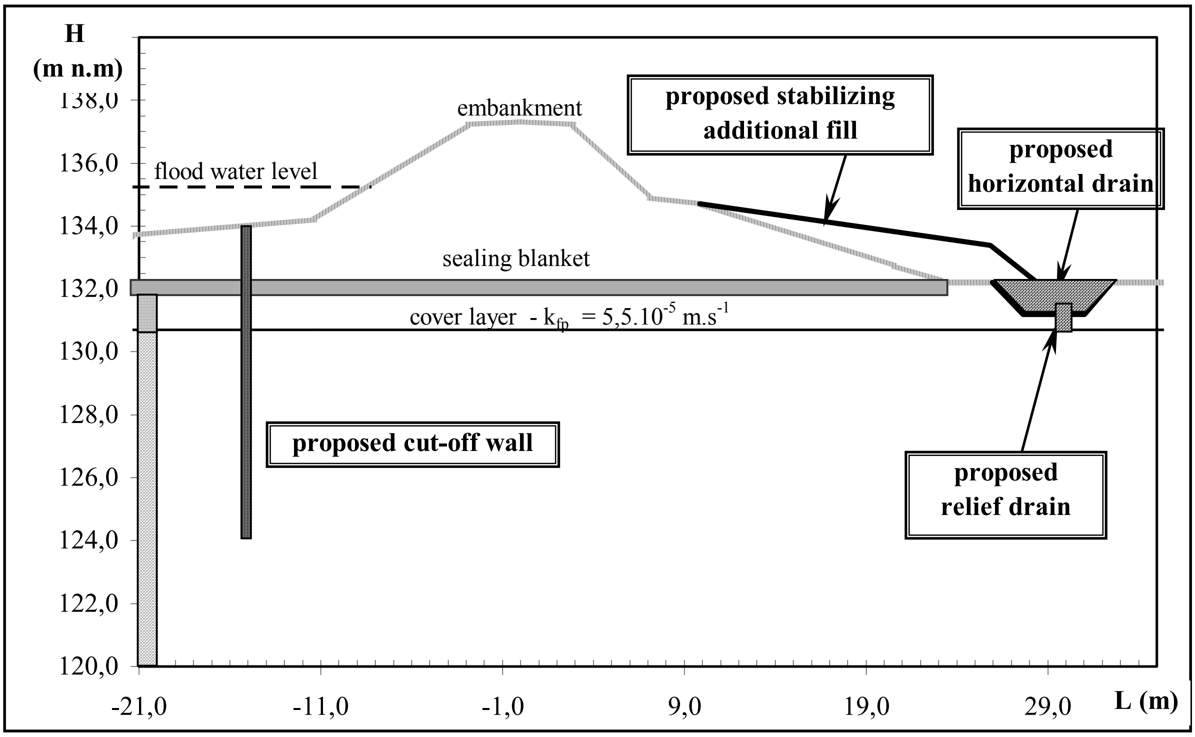

Fig. 5: Proposal of remedial measures

A similar example dealing with flood control problems in the western Slovakia along the Morava river may be presented. Solution of these problems followed immediately after the catastrophic failure of protective embankments in the Czech Republic in 1997.
Different geological conditions in the area of embankments along the Morava river (especially the accessible depth of the impermeable subsoil) made possible to consider various alternatives in designing protective measures. For instance, in one reach of protected area an

slurry wall was designed, having the length of about 7 km, bonded down to the impermeable subsoil (about 15 m - 18 m), with the aim to minimize the risk of filtration stability failure, and to reduce seepages causing flooding of the concerned territory during flood discharges. Attention was focused on several issues in dealing with the designing of the slurry wall, one of them being the analysis of the effect of bonded slurry wall on the groundwater level regime at various discharges in the Morava river. i.e. in addition to flood discharges also at average and minimum water stages, as well as the discussion on the assumed efficiency of the bonded or suspended slurry walls in concerned geological conditions [4]. Some information following from these studies are presented.

The subsoil of the embankments belongs to the Vienna Basin, consisting mostly of sandy soils with permeability of about 10^{-4} to 5. 10^{-4} $m.s^{-1}$, containing also neogenous clays. Their depth is variable - in the rage of about 4,5 m to 18 m, and even deeper.

In searching for optimum design of slurry wall one of the decisive parameter is the time development of the pressure line. Processed course of the pressure line in a profile, having the thickness of the permeable subsoil 18 m and assumed maximum flood discharges lasting 10 days, is presented in Fig.6. Under given geologic and morphologic conditions three cases were analysed - without protection of the embankment subsoil, with protected embankment subsoil by means of slurry wall, bonded down into the Neogene clays (about 18 m), and with shortened slurry wall (13 m), built in into sandy clays ($k_f = 1.10^{-4} m.s^{-1}$).

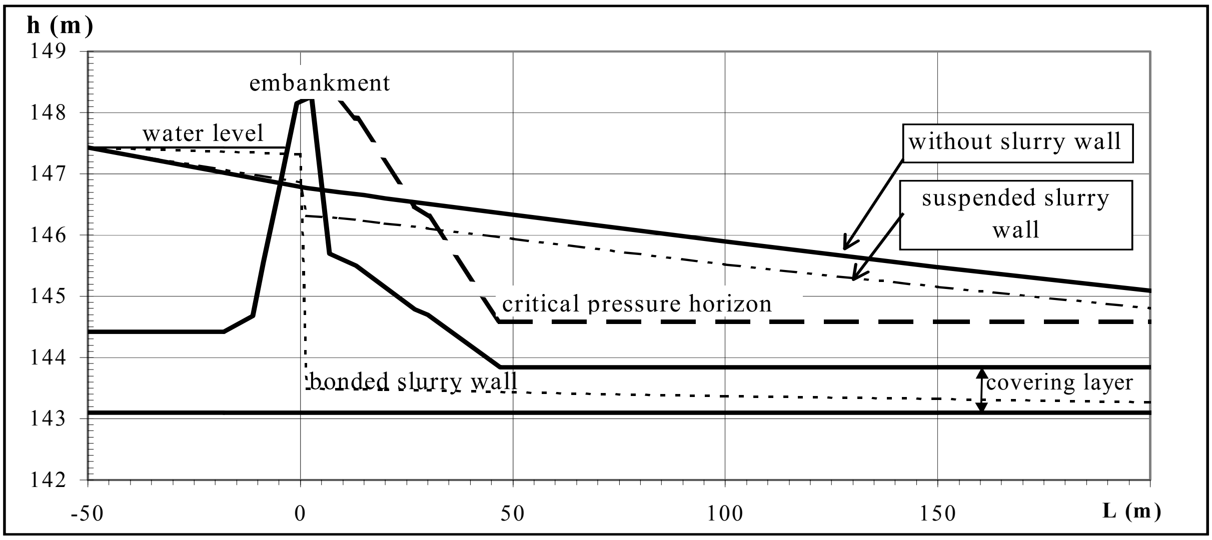

Fig. 6: Pressure lines in the protective embankment subsoil

As the comparison has shown, there is only a negligible difference between the protection of embankment subsoil with suspended slurry wall and unprotected subsoil at flood discharges duration of 10 days. The efficiency of suspended slurry wall considerably decreases with increasing duration of flood discharges. This has been confirmed also by the development of the pressure line, which has been processed for the case of suspended slurry wall in Fig.7.

The analysis of results, obtained by means of numerical simulation (finite element method) revealed, that the effect of the suspended slurry wall on the pressure line reduction is manifested mostly at the initial stage of flood discharges duration an is significantly determined by the permeability of the medium. With increasing duration of flood discharges it decreases to 10 %. Similar efficiency of the suspended slurry wall was proved also on the basis of in situ measurements under conditions of the Danube embankment.

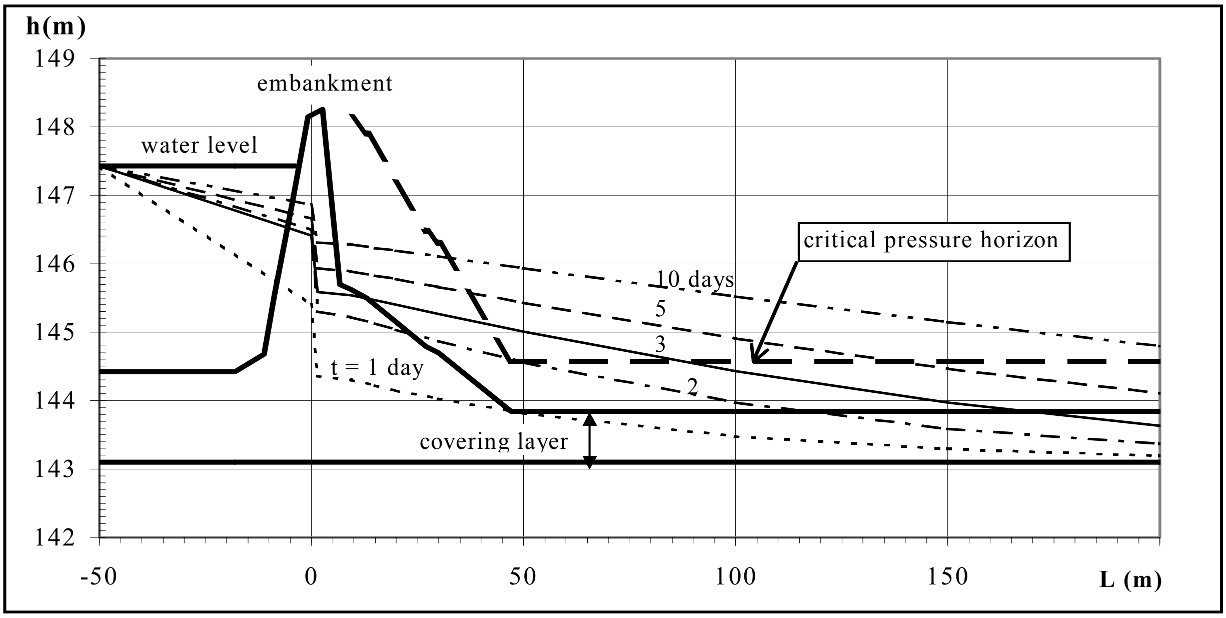

Fig. 7: Development of the pressure line in the embankment subsoil in case of suspended slurry wall

Achieved information indicate, that with regard to minimalization of negative effects of flood discharges on the safety of protective embankments and reduction of seepages through their subsoil to required level, slurry wall bonded down to the impermeable subsoil has a substantially higher efficiency than the suspended slurry wall. However, such choice of flood protection is not always realisable, and also it is not everywhere inevitable. It depends upon many factors, decisive being the human activity. Finally it is to be mentioned, that the described information represent only a fraction of problems involved in flood protection under Slovak conditions. It is evident that they may not bee generalised, but they may, within the scope of experience, contribute to solution of similar problems.

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The Effect of Antecedent Basin Saturation on Flood Extremity

K. Hlavčová, S. Kohnová, R. Kubeš, J. Szolgay, M. Zvolenský

Department of Land and Water Resources Management, Faculty of Civil Engineering, Slovak University of Technology, Radlinskeho 11, 813 68 Bratislava, Slovakia
hlavcova@svf.stuba.sk, kohnova@svf.stuba.sk, kubes@svf.stuba.sk, szolgay@svf.stuba.sk, zvolensk@svf.stuba.sk

Abstract

Since medium- and long-term precipitation forecasts are still not reliable enough, real time estimation of the degree of extremity of flood events that might occur in course of forthcoming dangerous meteorological situations, could be useful as additional information in flood warning systems. One of the possibilities how to approach answering such a question is to use all routinely available data in conjunction with existing practices for off line flood risk estimation. In the paper the joint effect of synthetic extreme precipitation together with different degrees of antecedent basin saturation on extreme floods was analysed with regard to future flood risk estimation.

The Hron river basin with an area of 1766 km^2 located in the central part of Slovakia was chosen as the pilot basin in the study. The effect of antecedent basin saturation on the extremity of floods was quantified, and critical values of the basin saturation index leading to floods with a higher return period than the return period of precipitation were identified. A method how to implement these critical values into on line flood risk warnings in hydrological forecasting and warning system in the basin was suggested.

Keywords: rainfall-runoff model, antecedent basin saturation, extreme flood, event based runoff generation, flood warning

1 Introduction

In the following a methodological framework using a combination of traditional methods for flood risk estimation together with on line monitoring the state of a basin with a mathematical model with regard to future flood risk estimation in flood forecasting and warning was developed. Runoff resulting from extreme events and a set of antecedent soil moisture conditions that have occurred in the past during flood events was simulated using an event-based flood generation model. Several scenarios of the joint occurrence of synthetic extreme precipitation and antecedent basin saturation were so constructed and analysed. In such a way the effect of antecedent basin saturation on the extremity of floods was quantified. A critical value of the soil moisture index leading to floods with a higher return period than the return period of precipitation was identified. A method how to implement these critical values into on line flood risk warnings in hydrological forecasting and warning system in the basin was suggested.

Developments in flood estimation methods for flood protection purposes have advanced very rapidly in recent decades. The trends are based on using new statistical approaches, the application of new principles of regionalisation, and objective methods of selecting regional types (e.g., cluster analysis). These methods will not be followed here further. Other efforts focus on the use of rainfall-runoff models. In this field standardized procedures do not exist but the application of hydrological modelling in engineering activities is more and more often used. Both continuous simulation and event based approach was used for flood frequency estimation in a number of studies.

The main principles used in various flood modelling studies (Calver and Lamb (1996), Cameron et al. (1999, 2000a,b), Hashemi, Franchini and O'Connell (2000), Franchini, Hashemi and O'Connell (2000), Rahman et al. (2002), Lindström, et al. (1997) Bergström et al. (1992), Miklánek, et al. (2000) and Svoboda, et al. (2000)) can be applied for the purpose of this study, their detailed reproduction however would not be feasible. The continuous modelling approach relies on a rather complicated methodology of stochastic rainfall modelling especially when floods are caused by various atmospheric circulation patterns, the genesis and duration of flood–inducing rainfall may vary, and rainfall floods and snowmelt origin play a role in the flood regime of a particular region (it is the often case in Central European conditions). The event-based approach may be regarded as simpler and more appropriate, however the joint distribution of initial basin states, the distribution of magnitude and form of rainfall events, joint modelling of temperature and rainfall may also lead to difficulties, when considering both rainfall and snowmelt induced floods.

It was therefore intended here to overcome these difficulties by selecting a simple only rainfall-based approach for the analysis of floods. This restriction will be based on the analysis of the seasonality of floods, which shows the possibility using only rainfall-based simulations for flood modelling. Further the dependence between initial basin state described by the initial soil moisture conditions and the magnitude flood-inducing rainfall will be analysed in order to incorporate it into the modelling framework for future flood risk estimation for the decision makers as a parameter.

2 Description of the pilot basin and input data

In this study the Hron River basin with an area of 1766 km^2 was selected for the hydrological modelling to estimate different conditions of antecedent basin saturation on the extremity of floods. The Hron River basin is located in central Slovakia, and it was chosen as representative of mountainous regions in Slovakia. (Fig. 1) 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. 70 % of the basin area is covered by forest, 10 % by grassland, 17 % by agricultural land and 3 % by urban areas In the rainfall-runoff modelling input data from the 1961-2000 period were used: daily precipitation from 23 rain-gauge stations , mean daily air temperature and long-term mean potential evapotranspiration from 6 meteorological stations, and mean daily discharges in the basin outlet in Banska Bystrica.

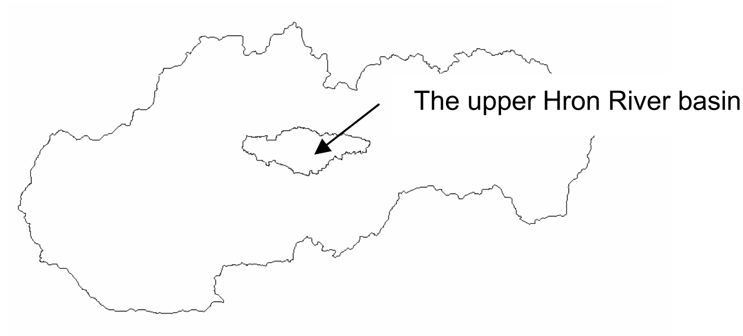

Fig. 1 Location of the Hron River basin in Slovakia

The flood forecasting problem in the Hron basin is complex and is characteristic for the mountainous regions of the country. In the alpine high mountain regions flash floods represent a threat to local villages build in narrow valleys. Due to the character of runoff concentration floods from rainfall of cyclonic origin represent main danger to major cities and industrial areas with heavy and chemical industry, electric and atomic power plants in the middle part and the lowest part of the catchment. The cities of Brezno, Podbrezova and Banska Bystrica in catchment are particularly affected. In the lower part of the catchment the floods generated in the upper parts threaten arable land and industrial facilities built along the river.

The seasonality of annual maximum floods was also studied in the basin. The seasonality index used by Burn (1990) was applied. It is a vector that represents the variability of the date of occurrence of flood events, its direction is the mean date of occurrence of maximum annual flood peak and the length is the variability of the date of occurrence around the mean value.

The results of the seasonality analysis for the Hron are shown in Fig. 2 in the context of the seasonality of mean annual floods for Slovakia (Čunderlík, 1999). It can be seen that for the whole basin, irrespective of the basin size, the annual maximum floods prevailingly occur in the warm season of the year. This feature developes with the growing size of sub-basins and at the basin outlet. Based on these findings the subsequent analysis of flood events was limited to the warm seasons of the observation period, thus only rainfall-induced floods were studied.

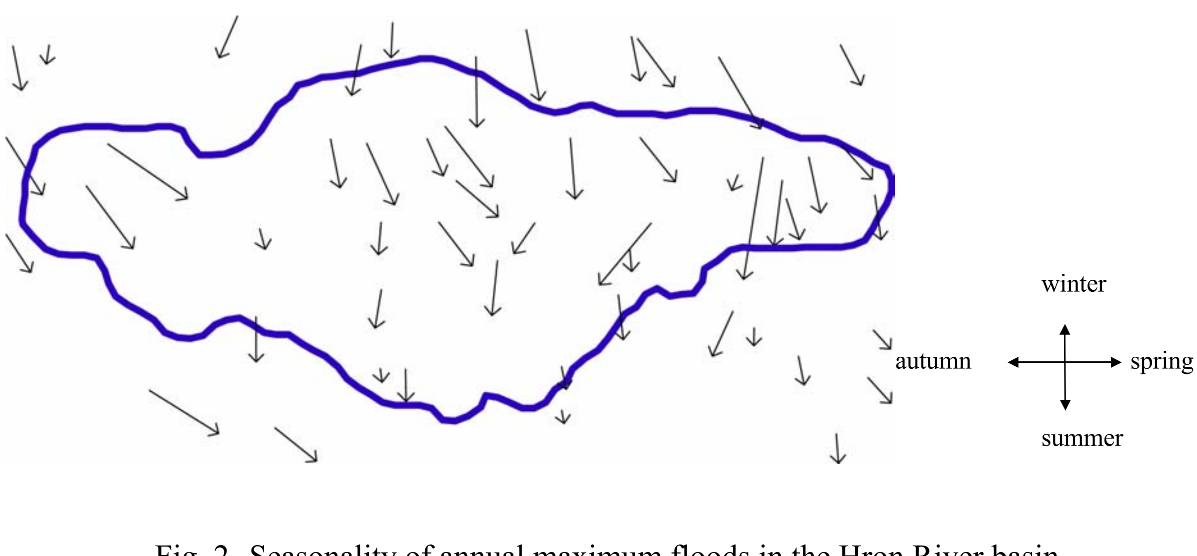

Fig. 2 Seasonality of annual maximum floods in the Hron River basin

Further a dataset characterizing the coincidence of extreme rainfall and antecedent moisture conditions of the basin was created with the aim to study the joint distribution of initial basin states and magnitude of rainfall events causing floods. A flood with the maximum mean daily discharge caused by rainfall was selected from the warm season of each observed year. The antecedent saturation of the basin before each selected flood was characterized by the water content in the soil layer determined from the continual simulation by the rainfall-runoff model; in the following text it is called the soil moisture index (SMI). For all selected events the soil moisture index varied in the range of 90-180 mm, which was less than the calibrated maximum field capacity with the value of 213 mm. (the value of 220 mm.)

An analysis of the relationship between the soil moisture index and causative precipitation depths for all selected floods was performed. The results of the analysis are shown in Fig 3; they indicate the independence of both characteristics (with a correlation coefficient of -0.005) and the possibility of their random combinations in the following event based simulations.

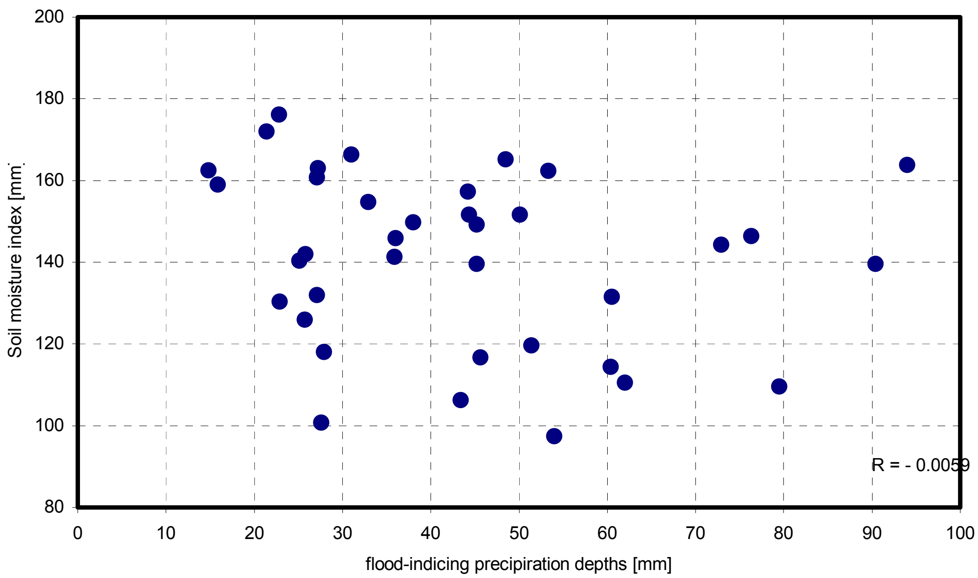

Fig. 3 Relationship between the soil moisture indexes and causal precipitation depths for selected extreme floods

3 Description of the rainfall-runoff model

The Hron rainfall-runoff model developed at the Slovak University of Technology in Bratislava for simulating runoff was used (Kubeš, 2001) which is based on the principles of the HBV model (Bergström, Forsman 1973). This conceptual lumped model, which is using daily time steps, consists of three basic components with 15 calibrated parameters. More detailed description of the model is given in Kubeš (2002).

The rainfall-runoff model was calibrated on Hron River basin against discharge data from the period of 1961-1980 using a genetic evolution algorithm. Following four criterions were alternatively used: the Nash-Sutcliffe criterion for optimalization *(R^2)*, mean daily error (Bias), the mean absolute error (Abserr) or the first lag autocorrelation criterion (RCOEF).

The model was validated for the period of 1990-1998, the acceptable agreement between the simulated and observed mean daily discharges from the calibration period and the fitness of the model for runoff simulation in the pilot basin was confirmed. Expressed in terms of the Nash-Sutcliff criterion, the value of 0.86 was considered as a sufficient agreement between measured and simulated discharges in the calibration period and in the validation period the value 0.83 was achieved. Comparison of simulated and measured mean daily discharges for a chosen year of the validated period is illustrated in Fig. 4.

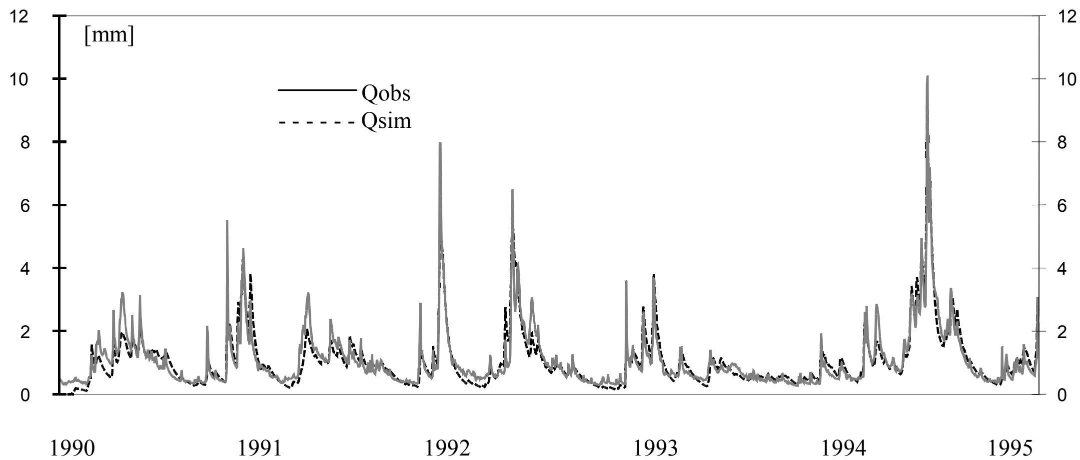

Fig. 4. Comparison of measured and simulated mean daily discharges [mm/day] for the part ofthe validation period

4 Simulations of synthetic extreme floods using synthetic extreme precipitation

As the result from the analysis of seasonality of floods and the relationship between the soil moisture index and causative precipitation depths for the selected annual maximum floods it was concluded, that it is possible to use a simple event based simulation of floods. The assumption of independence of the initial basin state and magnitude and form of flood inducing rainfall events was adopted.

The following simple event based flood generating model was set up. A set of initial conditions for simulation was created by selecting and storing all basin state variables in the rainfall runoff model occurring at the beginning of the annual maximum floods in the past.

In the following flood simulations synthetic extreme precipitation consisting of values of the annual maximum basin average 1-to-5 day precipitation depths with return periods of 5 to 100 years and the observed antecedent basin saturations occurring before annual maximum floods were combined at random. In the simulation by the rainfall-runoff the model actual flood-inducing daily precipitation depths were replaced by synthetic extreme precipitation with different return periods. The duration of the synthetic precipitation (1- to-5 days) was chosen to be equal to the duration of the actual precipitation at the initial state and its distribution over individual days was also determined according to the daily percentile distribution of actual precipitation. An example of such simulated results for the year 1984 is illustrated in Fig. 5.

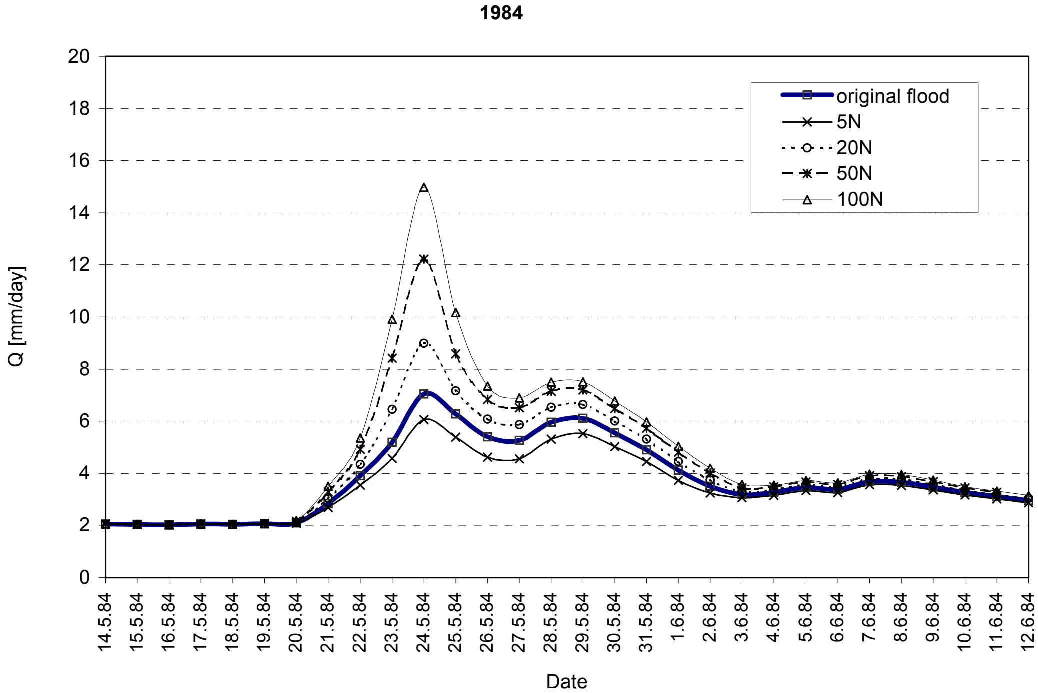

Fig. 5 Simulations of floods caused by synthetic precipitation with the durations of 5 days and return period of 5 to 100 years in 1984.

Subsequently all synthetic flood peaks were statistically analysed. The flood frequency curve using the P3 distribution was constructed and the return periods of the simulated maximum daily discharges *(NQ)* determined for all simulations. The ratio of the return period of the causative precipitation *NP* and that the resulting synthetic flood wave *NQ* were computed. The relationships between the ratios of *NP/NQ* and soil moisture index *SMI* occurring before the event (as computed by the model) were analysed. The relationships determined for the synthetic flood-inducing precipitation with the return period N of 5 years, all four fitted curves in log coordinates are shown in Fig. 6. Coefficients of determination of the fitted relationshiops are presented in Table 1.

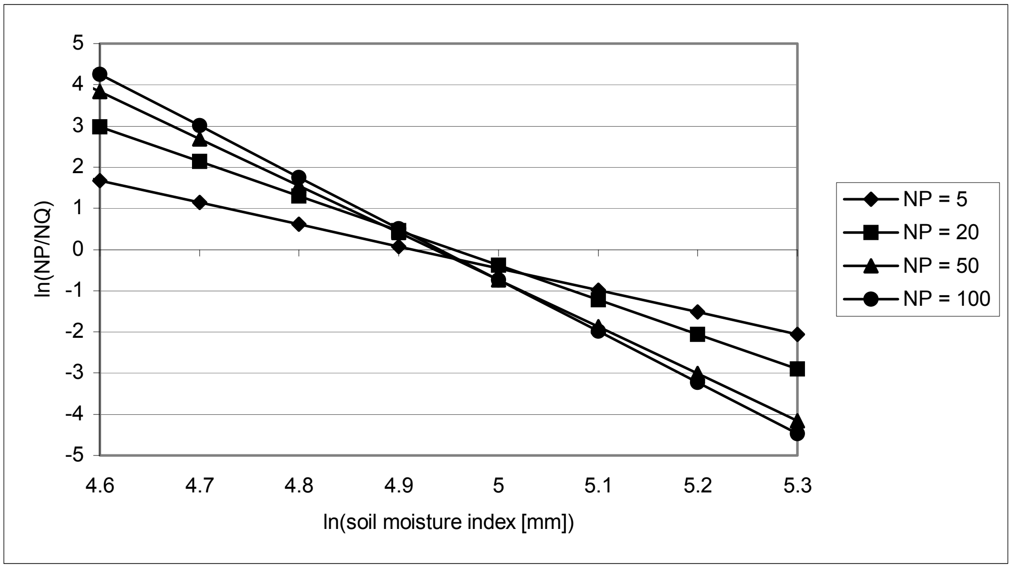

Fig. 6 Relationship between the logarithm of the ratio of return periods of synthetic precipitation and generated floods ln(NP/NQ) and logarithm of the soil moisture index ln(SMI) for N = 5, 20, 50, 100

The soil moisture values read from the fitted curves for the ratio NP/NQ = 1 are listed in Table 1. It can be seen, that for this given catchment there seems to be stable value of a critical soil moisture index around 140 mm. When this values is exceeded, it is very likely, that the return period of a flood will be higher than that of the flood inducing basin average precipitation.

This feature may be basin specific, but it can be used in the Hron basin's flood warning system for real time estimation of the degree of extremity of flood events that might occur in course of forthcoming dangerous meteorological situations. In Figs. 7 and 8 it is indicated, how this feature could be used as a simple decision support tool. Both figures relate the return period of precipitation to the expected return period of flood for different soil moisture index values. These estimate can be used as a useful additional information in flood warning and may help decision makers to take preliminary precautionary measures well in advance.

Return period	N = 5 years	N= 20 years	N= 50 years	N= 100 years
Coeff. of determination R^2	0.85	0.77	0.74	0.79
Critical SMI (mm)	138	143	141	142

Table 1. Critical soil moisture index values

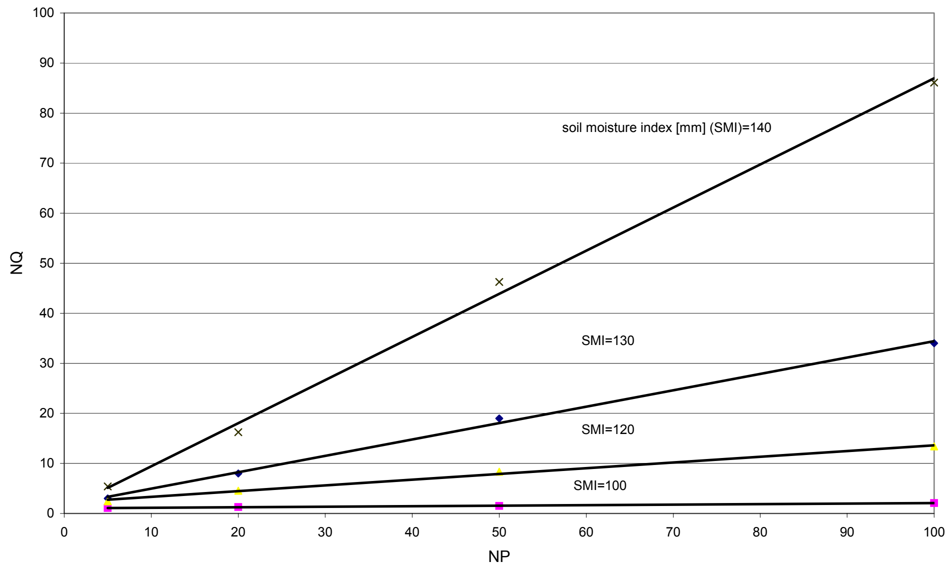

Fig. 7 Relationship of the return period of precipitation to the expected return period of flood for different soil moisture index values.

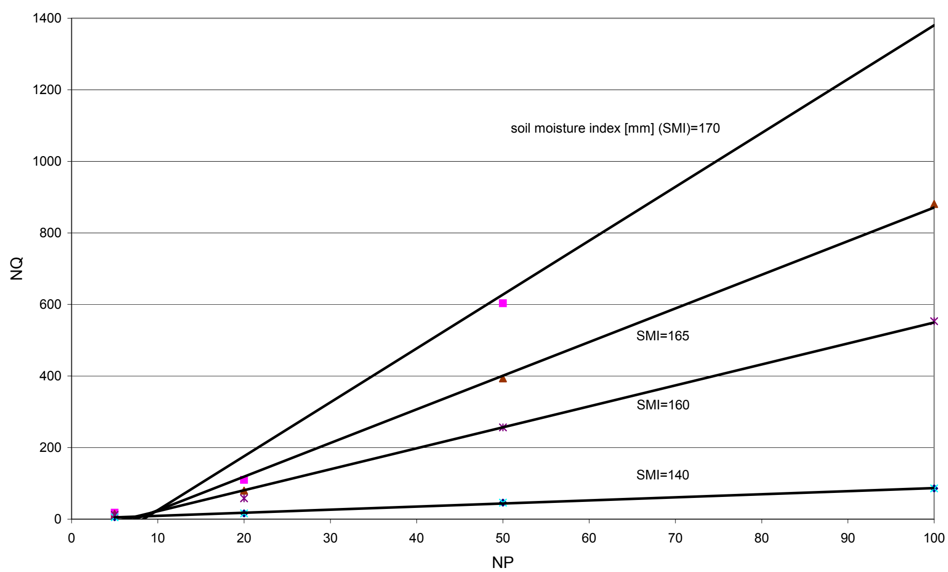

Fig 8 Relationship of the return period of precipitation to the expected return period of flood for different soil moisture index values.

5 Conclusions

Simulations of the joint occurrence of extreme precipitation and various antecedent saturations of a basin confirmed and quantified the expected strong effect of antecedent basin saturation on a flood's extremity. In all the simulated events the flood's extremity increased with increases in the antecedent basin saturation and, in many cases, the return period of a flood was much higher that the return period of the causal precipitation. In addition it can be also concluded, that the common assumption of engineering hydrology that a design flood with given return period may be caused by precipitation with the same return period may not be valid.

The nonlinearity of the relationship between the ratio of the return periods of precipitation and floods and soil moisture index increased with the increase in the causative precipitation's return period. But it was shown in the Hron River basin that, practically independently from the return period and the duration of the causal precipitation, one critical value of the soil moisture index can be indicated when the return period of a flood starts to be higher than the return period of causal precipitation. This fact and the derived relationships can be exploited in a flood warning system in the basin.

The results of the study can be used in estimating flood risk and flood protection in the Hron River basin. The critical soil moisture values can be determined by continually monitoring the antecedent soil moisture in the basin or by continual runoff simulation with a hydrological model. The results can also be applied in design flood estimation in the basin, but they should be confirmed for other basins.

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Seepage Problems of the Liptovská Mara Dam

Jozef Hulla

Slovak University of Technology, Department of Geotechnics,
Radlinského 11, 81368 Bratislava, Slovak Republic; hulla@svf.stuba.sk

ABSTRACT: Foot drain of the Liptovská Mara dam was founded high over the groundwater level. Seepage regime can be analysed only with the help of observation objects on the base of water level and velocity regimes. The dam subsoil was sealed with a short grout curtain that has been completed during reservoir filling and operation of the power project. Its quality has been controlled by water pressure tests; the required criteria were not attained, in spite of that is the curtain efficiency good.

1. Introduction

The Liptovská Mara – Bešeňová system is situated on the Váh river, between the cities Liptovský Mikuláš and Ružomberok. The system was put into operation in 1975. Purposes of the system are: to increase discharges in the Váh river for industry and agriculture, decrease flood discharges, electricity production, to create conditions for recreation and water sports; reservoir total volume is 360 mil.m^3 (greatest in the Slovakia), installed power plant capacity is 198 MW (Abaffy, Lukáč, Liška 1995).

The bedrock of the Váh valley consists of slate and sandstone flysh layers of Paleogene. The dam (maximum height above foundations 52 m, lenght in the crest axis 1225 m) is an earth-filled heterogenous dam with oblique, cranked sealing on the upstream side (Figure 1). Very short grout curtain reach to the depth of 10 m in the left–side bound, 20 m in the valley plain and 53 m in the right-side bound.

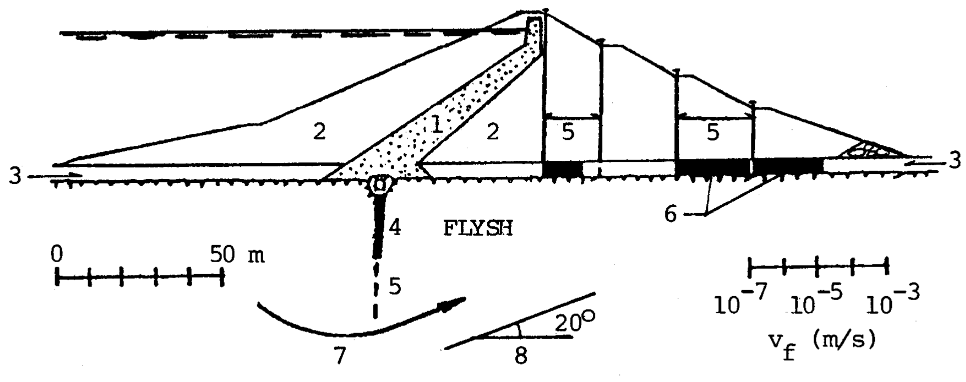

Figure 1. Cross-section of the Liptovská Mara dam: 1 – silt core, 2 – gravel, sand, 3 – alluvial deposits (gravel), 4 – grout curtain, 5 – observation boreholes, 6 – filtration velocities (v_f), 7 – curtain underflowing, 8 – fissures declination.

2. Water pressure tests

The reservoir was began to fill at the time when the grout curtain had not been completed. Not even after several stages of grouting works the prescribed Verfel´s criteria were not achieved, whereas the worst results took place right under the base of the grouting gallery at the depth of 0-10 m. Compared with the natural medium the permeability of the curtain was five times lower, but the Verfel´s (1983) and another criteria was not met at 60 – 20 % of the levels checked (Figure 2).

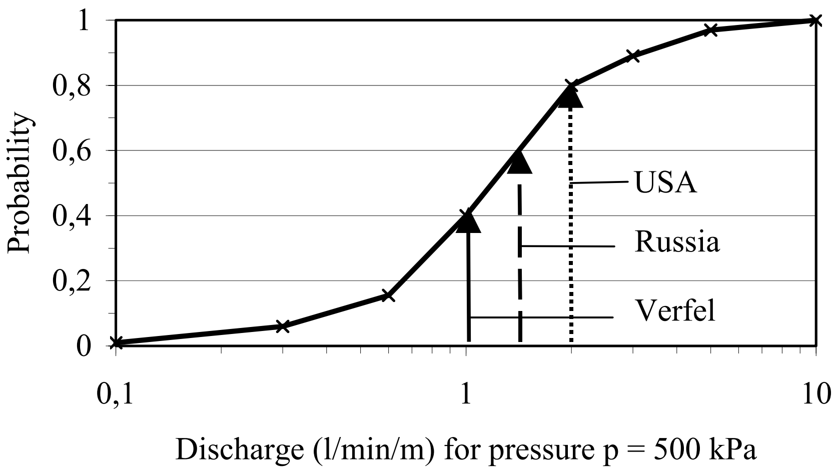

Figure 2. Water pressure tests results for the depth 0-10 m and different sealing criteria.

According to experiences from the Slovak dams several different criteria can be used for water pressure tests only by evaluating of permeability of natural rock medium and in decision making processes on the need of a curtain grouting. During curtain grouting it is enough to monitor the quality of the work on the bases of pressures and mixture consumption. In the our opinion at the high water presure the grout curtain can be faulted during tests and results are not correspond to the reality. Effectiveness of the completed curtain can be tested during the filling of the dam, and during its operation, the best way by monitoring of the characteristics of the seepage regime.

3. Groundwater levels

Due to seepage during the reservoir filling and operation, under the dam the groundwater levels increased on average by 0.8 m and did not reach the level of the bottom drainage. Seepage can only be monitored by water flow measurement in the observation boreholes. Groundwater level isolines for seepage regime in 2002 are given in Figure 3 with two concentrated seepage positions.

4. Filtration velocities measurements

With respect to the assumed problems had the measurements of filtration velocities by single borehole tracer methods (Halevy et al. 1967) been performed regularly.

Filtration velocities can be determined on the base of electrolyte solution vertical motion or dilution process monitoring in a perforated tubes. Results of the measurements are representative for a small area in the vicinity of the borehole given by several multiplications of its diameter. The borehole interconnects various pressure horizons almost always, and vertical water flow takes place.

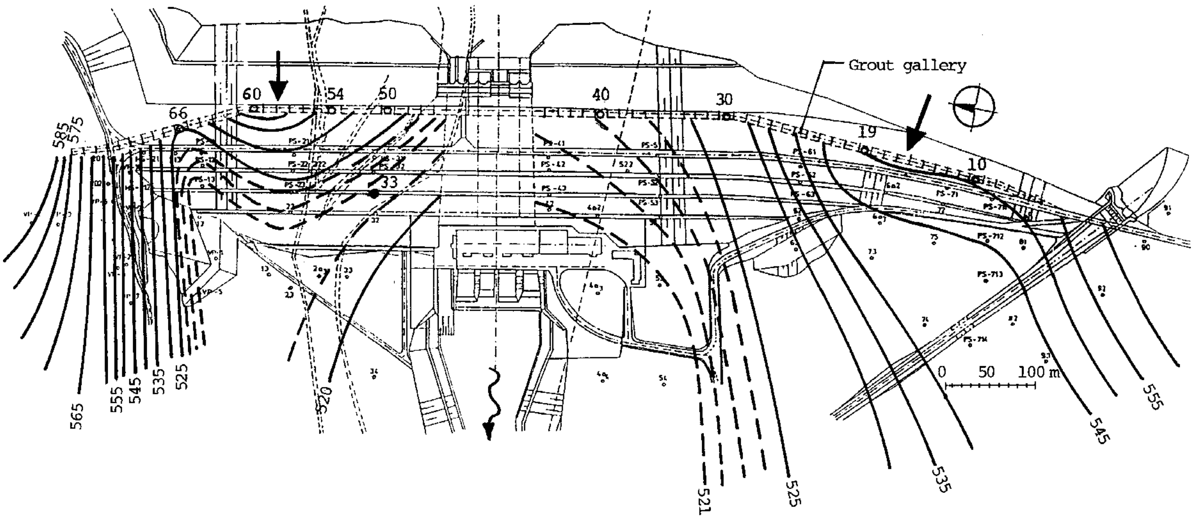

Figure 3. Groundwater level isolines in 2002 with two positions of concentrated seepages ⟵

In order to measure vertical water flow in a borehole a set of equipment can be used. An immersion probe is put into the borehole; is connected to a battery powered measurement equipment placed together with tracer jet control (NaCl solution) on the surface.

Concentration dependency can be watched directly on the computer screen and the time in which the maximum concentration takes place (t_{max}) can be determined. Estimation of the vertical velocity average value requires a laboratory calibration to set the computational time and vertical discharge shall be estimated from the continuity equation:

$$q_v = v_v\, A = \frac{l_v\, \pi\left(d^2 - d_s^2\right)}{4 \times 0.266\, t_{\max}^{1.474}} \tag{1}$$

where v_v is the vertical velocity, A - cross section test tube area, l_v - vertical distance, t_{max} - peak time, d - inner diameter of the tube, d_s - outer diameter of the probe.

The measurements are repeated in appropriate depth intervals so that the whole watered part of the borehole was equally covered and the vertical water flow as a depth function could be graphically depicted.

Filtration velocity in the surrounding medium (approximately in the horizontal direction) can be calculated from the vertical water flow measurement in a borehole based on the following equation:

$$v_f = \frac{\Delta q_v}{\overline{\alpha}\, d\, \Delta h} \tag{2}$$

where Δq_v is the increase or the decrease of the water discharge in the part of the borehole with the height of Δh, $\overline{\alpha}$ - borehole drainage influence coefficient for vertical flow (approximately $\overline{\alpha} \cong 20$), d - tube inner diameter.

Filtration velocity calculations according to the formula (2) are made with personal computers, results being graphically interpreted as depth dependencies. The average filtration velocity value of each borehole is given by the formula:

$$\overline{v}_f = \frac{\sum v_f \ \Delta h}{\sum \Delta h} \tag{3}$$

More intensive vertical flow in a borehole can be measured easier by borehole flowmeter.

Dilution method is used in observation boreholes with a very low water column. The tracer (NaCl) is usually introduced to water as powder. An immersion electrode probe together with battery conductometric equipment is used to monitor the dilution process. Filtration velocity is calculated by the formula:

$$v_f = \frac{\pi \ d}{4 \ \alpha \ t} \ln \frac{c_o - c_p}{c - c_p} \tag{4}$$

where d is the observation tube inner diameter, α - borehole drainage influence coefficient for the dilution method ($\alpha \cong 2$), c_o - initial concentration, c - concentration at time t, c_p - the natural concentration. The average filtration velocity values are again calculated by the formula (3).

Formula (4) assumes that solution dilution is caused by water flowing perpendicularly to the borehole axis. If there is some water flow in the direction of the borehole, the basic assumptions of the evaluation formula validity are not met and such results cannot be used. In order to eliminate the vertical flow there are devices which protect with inflatable sealings the measured area in the borehole against interference vertical flow influence available at some working places (Drost, 1970).

5. Filtration velocities regime

The first ones took place before reservoir filling in the year 1970, observation boreholes were gradually complemented and during the last measurement in 2002 there were 106 boreholes available.

The Figure 4 illustrates results from the borehole No. 54, located in grouting gallery, with permeable part in the depth from 20 to 44,4 m, under the grouting curtain. Most intensive ascent vertical flow in 2002, was measured at the open head of borehole and the maximum value of the filtration velocity 2.4×10^{-4} m/s had been determined in the depth of 43 m; average filtration velocity of the whole permeable part of the borehole was 8.0×10^{-5} m/s. At the close head of borehole was vertical flow descent and was less intensive; average filtration velocity was 2.5×10^{-6} m/s only.

Intensive flow in 2002 was obtain in borehole No. 33; maximum value of the filtration velocity of 1.9×10^{-4} m/s had been determined by the dilution method in the depth of 28.5 m, the average filtration velocity of the whole watered part of the borehole was 1.3×10^{-4} m/s.

In the valley part of the dam the average filtration velocity values in the gravel subsoil increase with the direction of water flow (Figure 1). Such phenomenon in the area can be interpreted as an underflow of a short grout curtain. These amounts are negligibly small from the water loss point of view. The most important are the hydrodynamic effects in the dam subsoil.

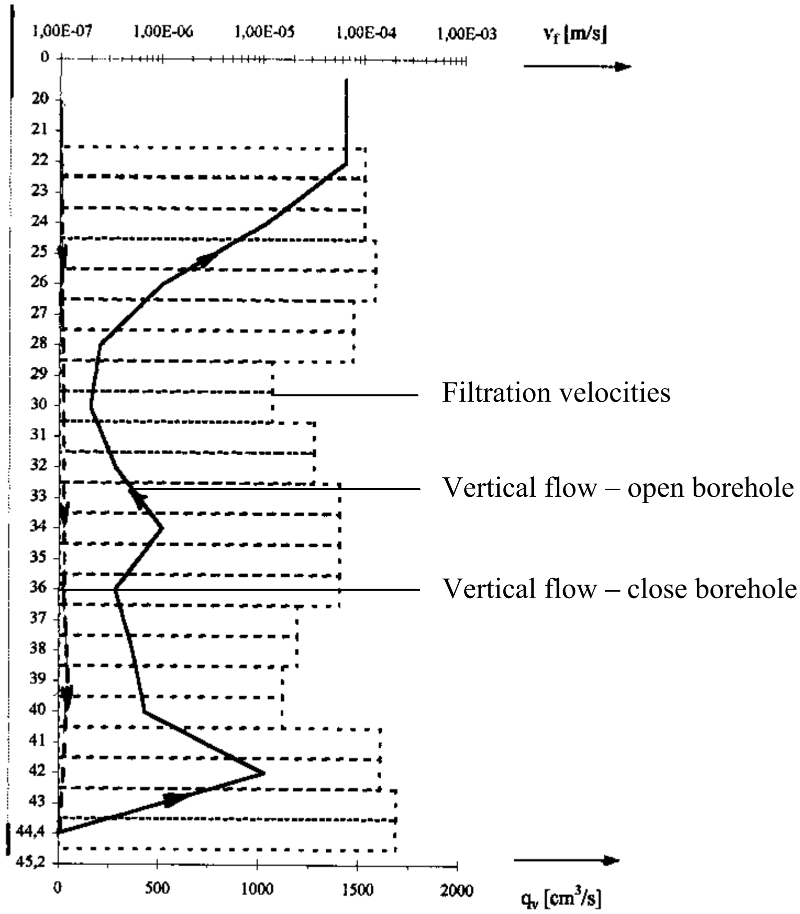

Figure 4. Dependencies of vertical discharges (q_v) and filtration velocities (v_f) with respect to the depth, measured in the observation borehole No. 54 (in grouting gallery) at the open and close head, in the year 2002.

6. Filtration stability

The measured maximum filtration velocities values are for these purposes compared with the critical values for washing out of sand particles from gravel soil pores - piping (Hulla and Cábel, 1997) or of fine-grained particles from the rock fissures (Ronzhin, 1974). Development of maximum filtration velocities in quarternary gravel soils and paleogenous flysch rocks under the dam is presented together with the critical velocities in the Figure 5. The results show that no situation that would be dangerous for the stability of the subsoil, as well as the whole dam had occurred during the filling of the dam, and it's operation up till now, even in spite of the fact that the curtain tightness criteria judged on the base on unsuitable water pressure tests had not been met.

7. Seepage discharges

Seepage discharge, calculated from the one borehole tracer measurements, being about 0.020 m^3/s for currently full reservoir. Initial calculations of the project engineers a value of 0.070 m^3/s had been expected (Figure 6).

In the years 1976-1978 reduction of seepage was obtained by additional grouting in the tectonic failured positions. Between 1978 and 1983 was positively expressed seepage reduction by natural processes due to colmatage.

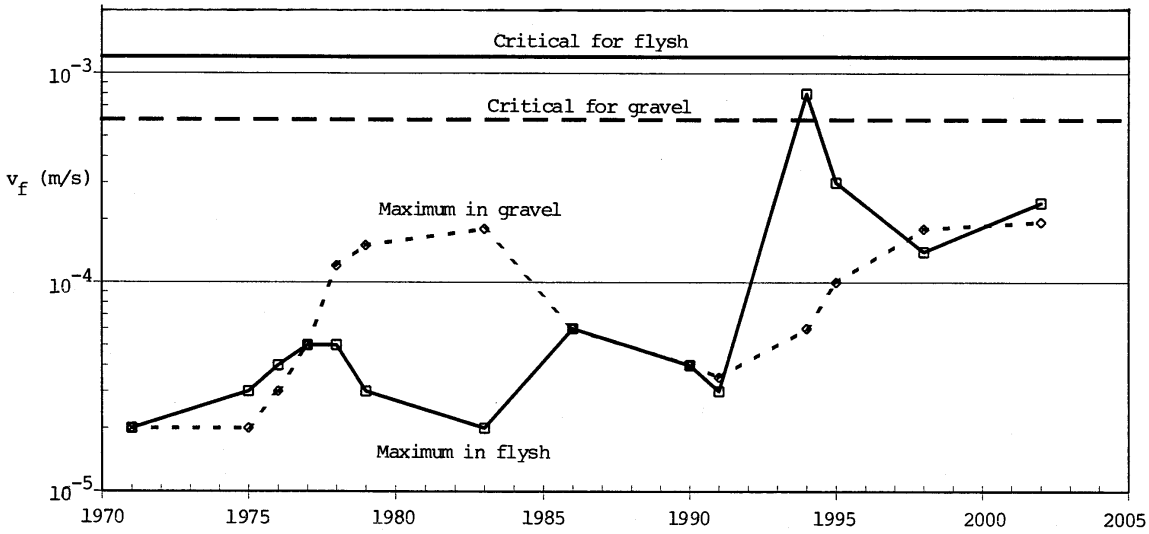

Figure 5. Development of the maximum filtration velocities in subsoils of the Liptovská Mara dam, and critical velocities for the stability of the sand particles in the gravel soil pores, and for fine-grained particles in rock fissures; v_f – filtration velocity.

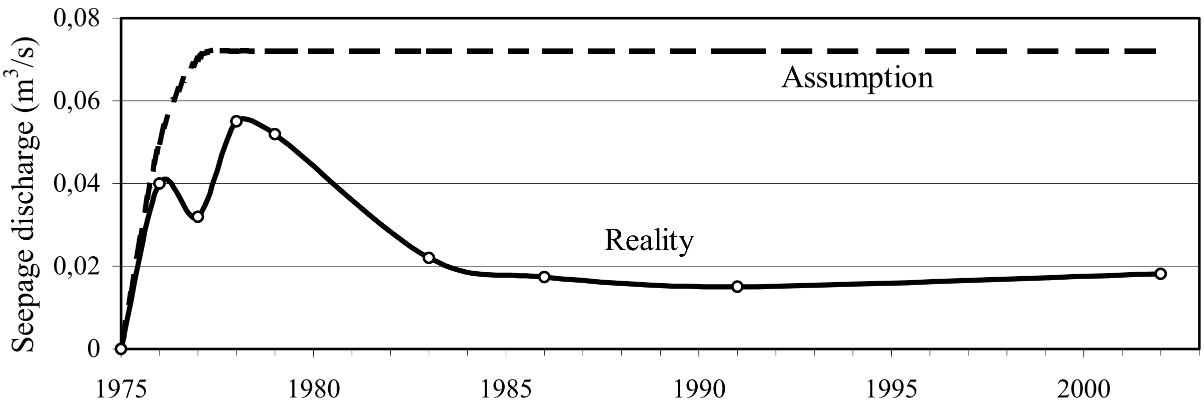

Figure 6. Seepage developments – project assumption and reality.

8. Observation boreholes aging

Processes of the observations objects aging immediate connected by their creation, by medium stability in their surroundings, demonstrate by fine particles sedimentation into the boreholes, influencing of water level and flow velocities development. We have possibility to study processes of observation boreholes aging in the subsoil of Liptovská Mara dam during 32 years.

Time development of maximal and average filtration velocities, groundwater levels and depth of the observation borehole No. 53, which has permeable part located in flysh subrock of the dam, are given in Figure 7.

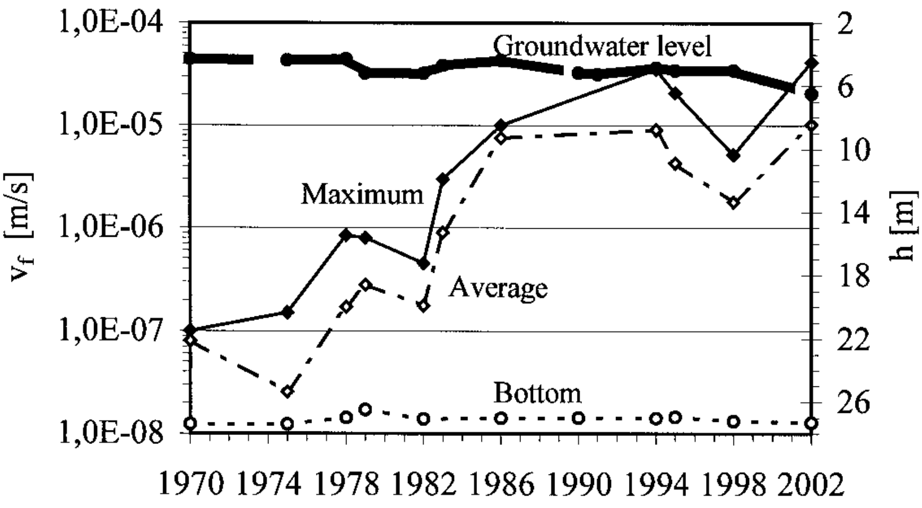

Figure 7. Time development of maximum and average filtration velocities (v_f), groundwater level and depth (h) of boreholes No. 53 in the flysh subrock of the Liptovská Mara dam.

Water level development has moderate sinking tendency, depth of borehole is practicaly unchangeable. Development of maximum and average filtration velocities has prevailingly ascending tendency, with short periods of decline, which has not connection with water levels. Maximal filtration velocities were always smaller than critical value for washing of fissure filling ($v_{crit} = 1.5\times10^{-3}$ m/s). Creation of the borehole No. 53 is possible to evaluate positive, aging process till now is not negative express.

For all observation boreholes with permeable parts in fissured flysh subrock of the Liptovská Mara dam is time development of maximum and medians filtration velocities given in Figure 8.

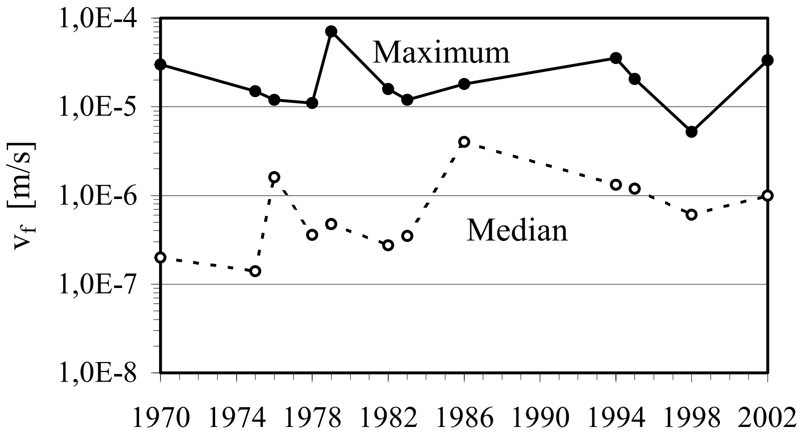

Figure 8. Time development of maximal filtration velocities (v_f) and medians in observation boreholes wit permeable parts in flysh subrock of the Liptovská Mara dam.

Relative stable maximal velocities and ascending tendency of medians testify, that 32 years old aging process of observation boreholes is not negative till now for investigation of the velocities regime. Equally it is valid too for water levels regime, which are influenced mainly by the most permeable positions in observation boreholes and in its surrounding.

9. Conclusions

In conditions of Liptovska Mara dam, seepage regime can be analysed only with the help of observation objects on the base of water level and velocity measurements.

Several different criteria for water pressure tests can be used only by evaluating of permeability of natural rock medium and in decision making processes on the need of a curtain grouting. During curtain grouting it is enough to monitor the quality of the work on the bases of pressures and mixture consumption. At the high water presure the grout curtain can be faulted during tests and results are not correspond to the reality. Effectiveness of the completed curtain can be tested during the filling of the dam, and during its operation, the best way by monitoring of the characteristics of the seepage regime.

Maximum filtration velocities, and critical velocities for washing out of the fine-grained particles from the gravel soil pores and from the rock fissures (piping) are important for the stability of the subsoil and bedrock, as well as the whole dam.

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Problems with Sampling Groundwater Polluted by Oil Hydrocarbons

Jan Ilavsky, Danica Barlokova

Slovak Technical University, Department of Sanitary Engineering, Radlinského 11, 813 68 Bratislava, Slovakia, e-mail: ilavsky@svf.stuba.sk

Abstract

The problems encountered with sampling of underground water contaminated with oil hydrocarbons are discussed, including secondary contamination with organic compounds (phthalate acid esters) delivered from plastics when during sampling water is in contact with these materials. To avoid this connection tubes from inorganic materials (stainless steel, glass) should be made.
Secondary contamination effects were carried out using glass capillary chromatography. Water samples were extracted by microextraction with n-pentane and extracts analysed under condition as follows: column length 27 m, I.D. 0,30 mm, liquid stationary phase silicone SE-52, column temperature 25 ^oC, then programmed to 250 ^oC at 10 $^oC/min$, injection port and detector (FID) temperature 260 ^oC; carrier gas nitrogen, injection technique split/splitless, sampling size 1,5 μl.

1. Introduction

Sampling trace analyses of groundwater polluted by oil hydrocarbons is a top priority. Common practice has revealed the fact, that in this phase, samples are often polluted by which have not been previously in water before. It is necessary to consider the significant contradictions which appear in dealing with problems associated with the elimination of oil hydrocarbons in groundwater and the analytical methods applied which permit settling the content of oil hydrocarbons with a very low concentration (10^{-6} - 10^{-9} $g.1^{-1}$) and the sampling method. On the one hand, modern analytical methods and expensive technical equipment (capillary gas chromatograph, spectrophotometer, etc.), which require highly qualified operation, and on the other hand, sampling, which does not require challenging methods, expensive instruments, or demanding operation contradict each other. Therefore, among the biggest errors which may appear in analyses of oil hydrocarbons in water (mainly in the event of trace concentrations) belong errors caused by sampling and possible storage. These errors are not negligible, because they can vary to a great degree from several percentages up to 100%. It follows from this that any sophisticated analytical technique which permits the achievement of a high sensitivity and precise analyses is not, in such cases, fully used.
The method and sampling technique depend on the types of sampling water (surface or groundwater), the type of analysis, its purpose, the level of difficulty, and the depth required (bottom, surface, and the depth set beforehand).
In groundwater sampling the sampling method is affected by various factors, such as, for example, the need for observation of the content of the oil hydrocarbons in a drilling hole (well), the depth and strength of the water horizon, the drill hole capacity, etc.

From a hydraulic point of view, the withdrawal of a certain volume of water into a container or a depth sampler means to make the water move upward without aerating the sample and preventing its contact with air. This can be obtained either by water suction or water delivery. Suction pumps or piston samplers are based on the first method. In water suction the balance between the gas and liquid phases fails, the samples are aerated, and bubbles are formed in the samplers, which results in incorrect analyses. During the withdrawal of the water the "pre-purification" of the samples by means of water-gas extraction must be avoided. Therefore, it is suggested to apply another method, i.e., water delivery by submersible pump. The pump has to be constructed in such a way that it can function with the lowest possible decrease in local pressure.

The selection of samplers, which is affected by the abovementioned factors, such as availability, a good price, and reliability, is a limiting factor in arriving at a suitable method. Owing to the foregoing facts and the need for groundwater sampling in drilling holes (wells) in different horizons (without organic phase on the surface), we have focused on submersible pumps.

The selection of submersible pumps is limited by the inert property of the pump materials against the action of the oil hydrocarbons as well as any change in the chemical composition of the samples and protection against explosion. For a sampling of low groundwater contaminated by oil hydrocarbons, Rondella, an inland submersible vertical membrane pump, has been tested from different angles. After adaptation it is also suitable for sampling in narrower diameters of drilling holes. Grundfos is another suitable submersible electromagnetic vibrating pump, which has been used in sampling with the Rondella pump.

However, when sampling with submersible pumps, water samples come into contact with pump materials (construction parts of a pump, valves, connecting fittings and seals) and delivery piping, many of which are of an organic origin (plastic, rubber, etc.). These materials continuously release various types of organic matter (filling agents and impurities) into water as the residues of a production cycle, and it is not possible to remove them.

Nowadays a great attention is payed to the harmful effects of polymer additives (plasticizers, antioxidants, stabilizers etc), which are very movable inside the plastic and thus can release into the environment [1].

Phthalates (diesters of the o-phthalic acid) belong to the group of the polymer plasticizers, especially PVC, most frequently used in practice. In some conditions phthalates migrate into the surrounding space and via food, water and air enter also the human body. Phthalates were found in underground and drinking waters in many countries, often in microgram quantities [2]. Di-n-butylphthalate (DBF), di-(2-ethylhexyl)phthalate (DEHF), butylbenzylphthalate (BBF), di-n-oktylphthalate (DOF) and others belong among the most frequent contaminants.

For the purpose of the phthalate analysis capillary gas chromatography is being used as the best method, especially in the cases where sample contains a rich mixture of another compounds, i.e. petrol hydrocarbons, and non-selective (FID) detector is used. In the most frequent cases capillary columns with silicone stationary phases and column temperature gradient are being used [3,4].

As the most frequent method for isolation of phthalates from water an extraction with nonpolar organic solvents is being used, i.e. n-hexane [2,5], i-octane [6], n-pentane [7] etc.

Nowadays microextraction is being used for the routine analysis of many organic contaminants of water. In this paper we have focused our effort to the possibilities of microextraction in phthalate analysis [7]. As the extraction solvents n-pentane was used.

2. Experimental

2.1 Instrumentation

The analysis were performed with the Carlo Erba (VEGA 6000) gas chromatograph (modified for usage with capillary columns) equipped with a flame ionization detector and split/ splitless capillary injector. The chromatograph was fitted with glass capillary column (27 m x 0.30 mm i.d.) coated with silicone stationary phase SE-52.
The gas chromatographic conditions were as follows: the column temperature during injection was 25 ^oC, then programmed linearly at 10 ^oC/min to a final temperature of 250 ^oC, nitrogen was used as carrier gas and 1.5 μl samples were injected using splitless injection mode (valve time 30 s).
The injector and detector temperature were 26O ^oC. Chromatograms were integrated with HP 3392A (Hewlett-Packard) integrator.

2.2 Reagents and Solutions

The extraction solvent - n-pentane (for spectroscopy) was obtained from E.Merck (Darmstadt, Germany). Phthalates were of analytical grade from commercial sources.
The purity of the internal standard - 1-Cl-n-octadecane was 99% (Supelco, Bellefonte, Pa., USA).
Standard solutions of the compounds used in this paper were prepared in n-pentane. For extraction 0,5 ml of this solution was used.

2.3 Microextraction

For the microextraction a simple glass extraction (volumetric) flask (1) equipped with the conical stopper (5) was used. For n-pentane separation a separator of the thin solvent layer (2) was connected to the flask, and through the side arm, 3, pure water is added until the solvent is transferred into the dry capillary, 4, to the height required (Figure 1) [8]. The n-pentane extract is easily accessible and can be injected immediately by means of a syringe into a gas chromatograph.

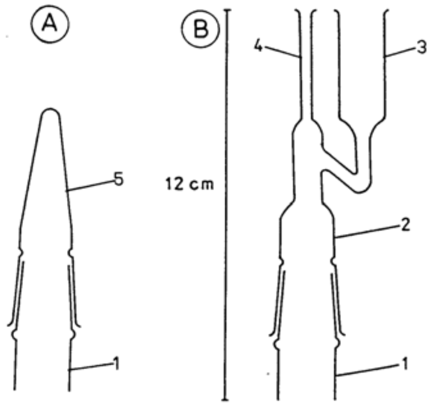

Fig. 1:

A: The glass extraction (volumetric) flask, equipped with a male joint (1) and conical stopper (5), prepared to microextraction.

B: The solvent thin layer separator (2) containing side arm for water (3), capillary for extract (4) conected to the flask (1) after extraction.

3. Results and Discussion

When using highly sensitive methods, such as, for instance, capillary gas chromatography, this secondary contamination always occurs. Therefore, it is essential to know its effect on qualitative analyses. The samples on the chromatograms of all submersible pumps with plastic or rubber delivery hose pipes show regular elution waves, which represent impurities that penetrated into the samples during sampling. In order to prove this effect, the release of impurities from the hoses that were used was tested, and drinking water was used as a model sample.

For the water analysis microextraction followed by the capillary gas chromatography can be used. One liter of water containing defined content of organic substances was subjected to microextraction (at 7 ^{o}C) with 0,5 ml of n-pentane containing internal standard (IS) by intensive manual shaking 5 min. Internal standard was used to obtain microextraction recovery of phthalates.

The chromatogram of the n-pentane extract of drinking water is in Figure 2. Figure 3, and 4 shows a chromatograms of 1 litre of a drinking water sample in which a transparent or yellow plastic hose was dipped for 2-3 minutes during stirring (10 cm long, with an external diameter of 2 cm) and Figure 5 shows a chromatogram while using a red rubber hose. The samples were obtained under the same conditions.

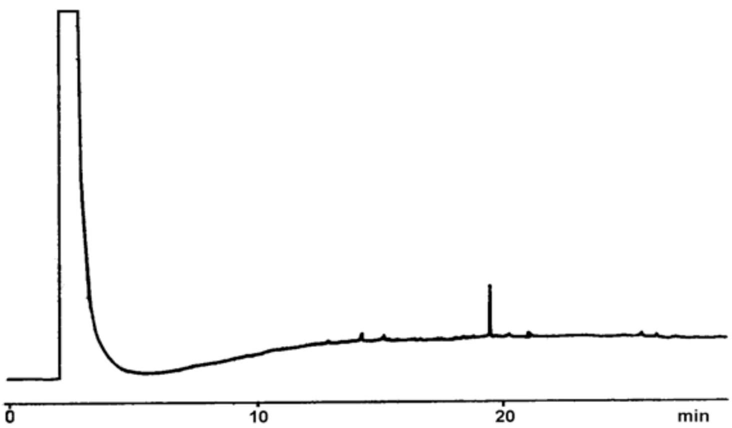

Fig. 2: The chromatogram of the n-pentane extract of drinking water sample

From comparing the chromatograms in pictures 3-5 with the chromatogram of water, it is clear that during the short contact between the hoses and water, some elements penetrated into the drinking water from the materials the hoses are made of (softeners). The lowest amount of water contamination is caused by rubber hoses (less than 0.01 mg . 1^{-1}), while the contamination caused by plastic hoses ranges from 0,01 mg – 0,1 mg . 1^{-1}.

Figure 6 demonstrates the chromatogram of a n-pentane extract of 1 litre of drinking water after passing through a plastic hose that had not been used before (15 m long with an external diameter of 2 cm), and Figure 7 shows the chromatogram after 30 minutes of the passage of water (6,0 $1.s^{-1}$). In comparing both of these chromatograms, it is evident that even after 30 minutes of the passage of water certain matter has been released into the water, which, if incorrectly determinated, can affect the results of analyses in a negative way, especially if other methods have been applied. In most cases it concerns phthalate esters, which are applied as plasticizers in the production of synthetic compounds.

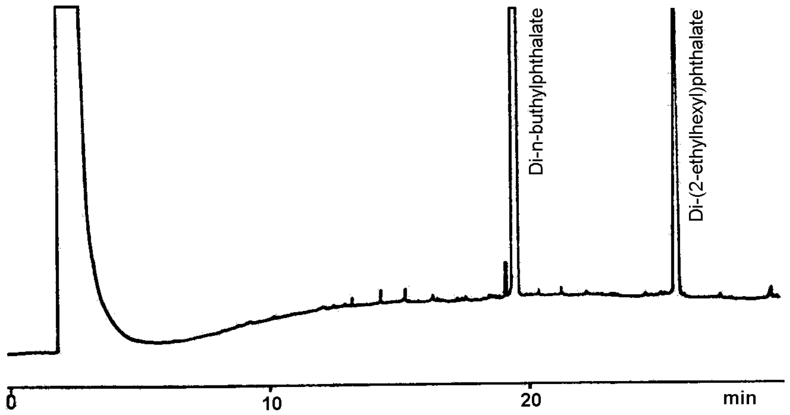

Fig. 3: The chromatogram of n-pentane extract of a drinking water sample after secondary contamination using a transparent PVC hose

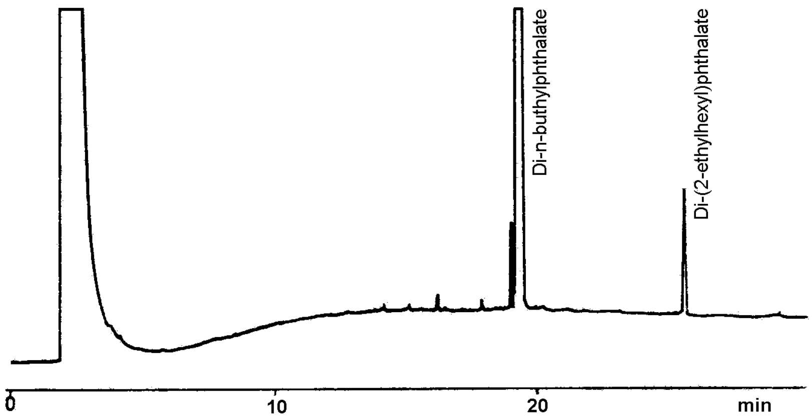

Fig. 4: The chromatogram of n-pentane extract of a drinking water sample after secondary contamination using a yellow PVC hose

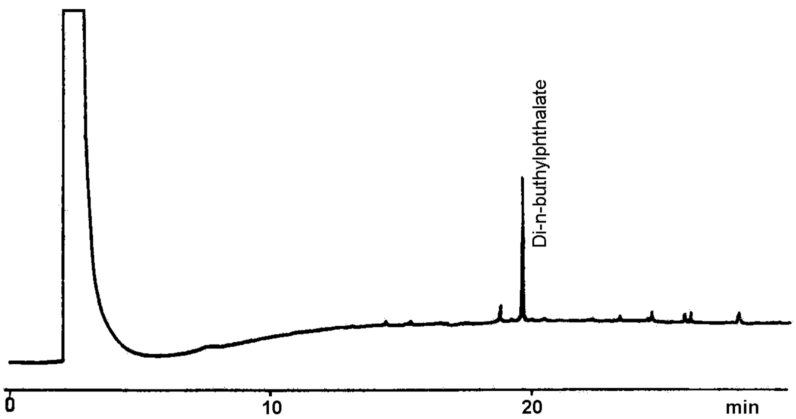

Fig. 5: The chromatogram of n-pentane extract of a drinking water sample after secondary contamination using a red rubber hose

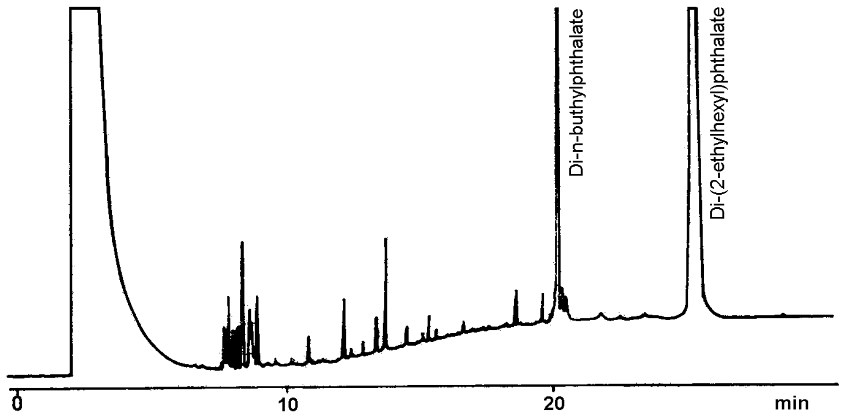

Fig. 6: The chromatogram of a n-pentane extract of drinking water after passing through a plastic hose that had not been used before

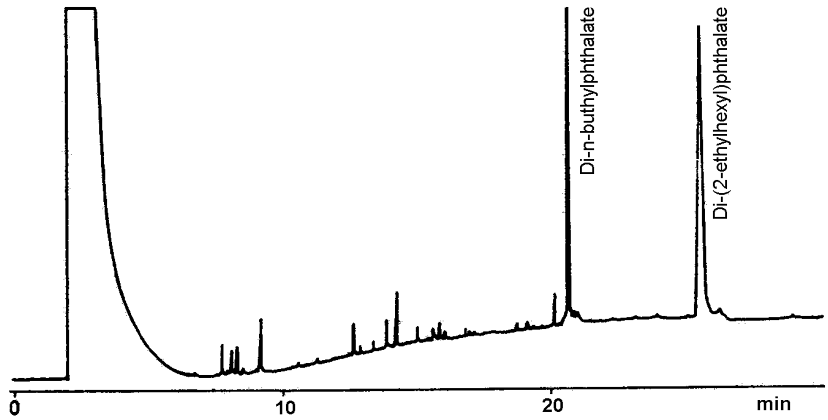

Fig. 7: The chromatogram of a n-pentane extract of drinking water after 30 minutes of the passage of water

We can conclude that n-pentane, being used for estimation of petrol hydrocarbons, is suitable for microextraction of di-n-butylphthalate and di-(2-ethylhexyl)phthalate. We mention this fact because for water sampling from wells and bores PVC hoses are being used (PVC contains DBF and DEHF) and thus the water sample is secondary contaminated with phthalates. These compounds are coextracted with later analyzed components into n-pentane and are able to influence the results of petrol hydrocarbon estimations.
As an example Figure 8 shows the chromatogram of the extract of 1 litre of water containing Diesel Oil (0,02 mg/l H_2O) by microextraction into 0,5 ml of n-pentane. Figure 9 shows the chromatogram of Diesel Oil in the presence of phthalates (5 μg/l of each) after extraction into 0,5 ml of n-pentane.

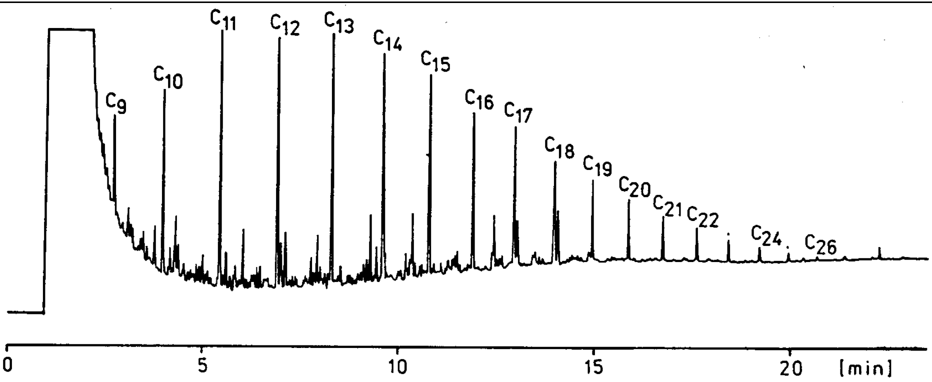

Fig. 8: Gas chromatogram of Diesel Oil (concentration 20 μg/l H_2O) after extraction of 1 litre of water with 0.5 ml of n-pentane (C_9 to C_{26} are elution peaks n-alkanes)

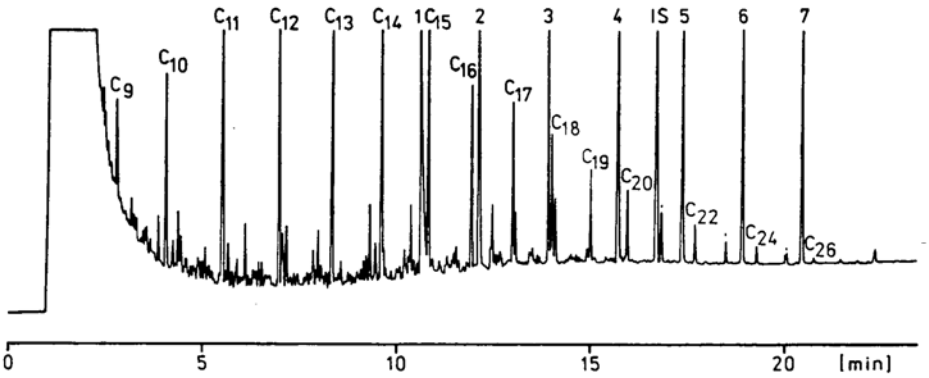

Fig. 9: Gas chromatogram of Diesel Oil (concentration 20 μg/l H_2O) in the presence of phthalates (5 μg of each) after extraction of 1 litre of water with 0.5 ml of n-pentane containing 10 μg 1-Cl-n-octadecane (IS).
1 – Di-methylphthalate, 2 – Di-ethylphthalate, 3 – Di-n-propylphthalate,
4 – Di-n-butyl-phthalate, 5 – Di-n-pentylphthalate, 6 – Di-n-hexylphthalate,
7 – Di-(2-ethylhexyl)phthalate, IS – internal standard are elution peaks phthalates

From this figures can be concluded that in using capillary gas chromatography and optimized analytical conditions phthalates do not influence the estimations, i.e. there is a possibility to estimate petrol hydrocarbons and phthalates in waters in one analysis.
In the case of the determination of solety phthalates, especially the higher ones (lowers are rarely used) in waters using the technique of microextraction into n-pentane there is a possibility to concentrate the n-pentane extract and thus quantified concentrations yet below 0,02 ug/l of water.

4. Conclusion

It result from the above considerations that the microextraction method of phthalates isolation is very rapid, simple, economically profitable and with low detection limit. The results of this study give data important for quantitative analysis of phthalates in waters (e.g. relative recoveries at different concentrations) which may be used for their routine quantitative analysis involving microextraction and capillary gas chromatography.

Just as impurities can be penetrate water during contact with organic materials in sampling, at the same time the same materials can deplete contaminated water with some of the elements analyzed. Therefore, only inert inorganic materials (such as non-corroding steel, polished hard chromium, glass, etc.) should be applied in the production of pumps and their accessories for depth sampling for analyses of organic contaminants.

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Application of Sediment Transport Formulae to Dam Breach Erosion

Jan Jandora[1], Jiří Hodák[2]

[1] Ing., Ph.D., Water Structures Institute, Faculty of Civil Engineering, Brno University of Technology, Zizkova 17, 662 37 Brno, tel: +420 5 4114 7759, email: jandora.j@fce.vutbr.cz
[2] Ing., Water Structures Institute, Faculty of Civil Engineering, Brno University of Technology, Zizkova 17, 662 37 Brno, tel: +420 5 4114 7759, email: hodak.j@fce.vutbr.cz

Abstract:
An essential part of mathematical models for breach erosion is the description of the sediment transport. The paper presents results of such used formulae that are applied to dam breach erosion. The calculations are then compared with the data obtained from physical (experimental) research performed on 0.86 m high sandy dikes at the Laboratory of the Water Structures Institute, Brno University of Technology, Czech Republic.

1. Introduction

The failure of dams can lead to catastrophic consequences, as populations tend to settle in floodplains. There are many examples of dam failures (for example the failure of the Teton dam, Idaho, USA, 1976, killed 11 people and caused major damage).
Simulations of breach formation and prediction of generated flood discharge are crucial for the risk management of dams and flood defences. An essential part of the mathematical models used for breach erosion simulation is the description of the pick-up and transport of the sediment.
Physical research of dam breach can provide valuable data for assessing the validity of present sediment transport formulae and for developing new ones.

2. Description of the test site

The models of a sandy dike (Fig. 1) were located in the Laboratory of the Water Structures Institute, Brno University of Technology, Czech Republic. Sandy material of the models had the following physical characteristics: friction angle $\varphi = 38.2^{o}$, mean particle size $D = 1.354$ mm, $D_{10} = 0.201$ mm, $D_{16} = 0.261$ mm, $D_{30} = 0.479$ mm, $D_{50} = 0.875$ mm, $D_{55} = 1.013$ mm, $D_{90} = 3.130$ mm, density of sand $\rho_{sand} = 2650$ kg/m^3. Slopes of the upstream and downstream face of the dams were 1:2, height of the dams 0.86 m, width of the dam crests 0.3 m, length of the dam crests 3.71 m (Fig. 2). The volume of reservoir was about 65 m^3.
Five experiments were done in 1998 and 1999 [11]. The first experiment was carried out to verify and obtain knowledge and experience with the process of dam breach and with measurements. Four experiments were completely evaluated [6] - discharge, 3D shape of the breach, volume of transported sediments, etc.
Each of the experiments had a bit different conditions - inflow to the reservoir (22.8 to 24.3 l/s), initiate saturation of the dam body (time of reservoir filling, rain before experiments), etc. So load transports, discharges through the breach, etc. were different in the experiments.

Fig. 1 The model of sandy dam

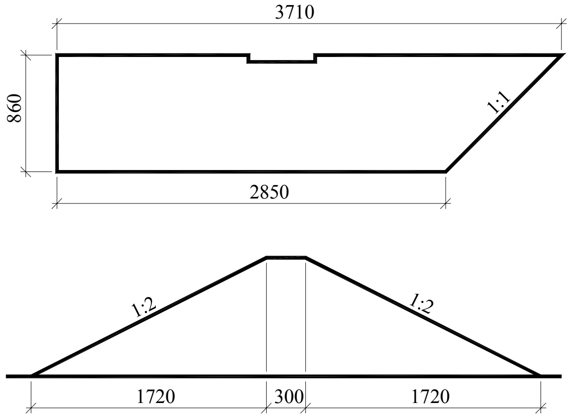

Fig. 2 Longitudinal and cross sections of the dam body

3. Evaluation of experiments

The following methods were used for evaluation of the experiments:

- video-log;
- photo;
- piezometers;
- thermistors installed in the dam body to record the progress breaching [8].

Digital 3D models of breach of 4 experiments were done [5], [6]. Figures 3 to 8 show photos, digital 3D models, volume of transported sediments and discharge of the breach experiment performed on 27th May 1999.

Fig. 3a Breach from the downstream - time 80 s

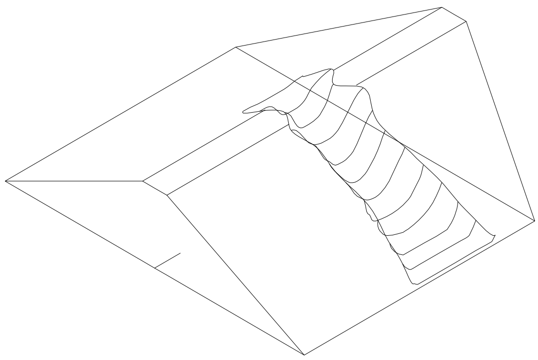

Fig. 3b 3D model - time 80 s

Fig. 4a Breach from the downstream - time 100 s

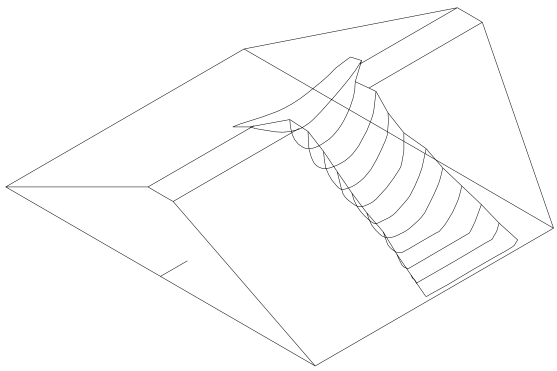

Fig. 4b 3D model - time 100 s

Fig. 5a Breach from the downstream - time 120 s

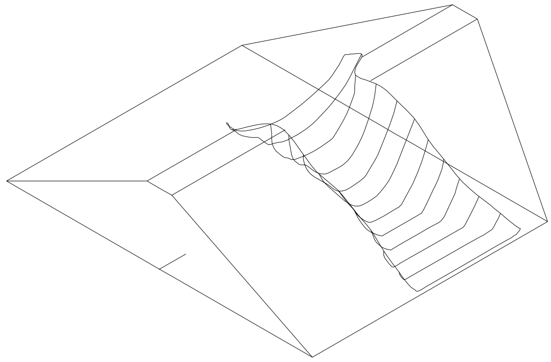

Fig. 5b 3D model - time 120 s

Fig. 6a Breach from the downstream - time 140 s

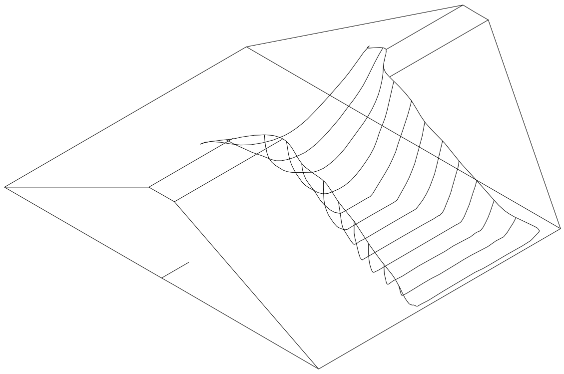

Fig. 6b 3D model - time 140 s

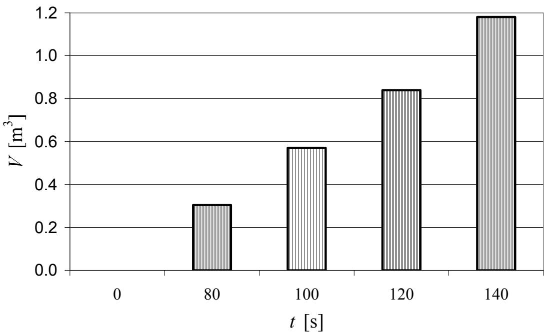

Fig. 7 Time log of the transported sediments volume

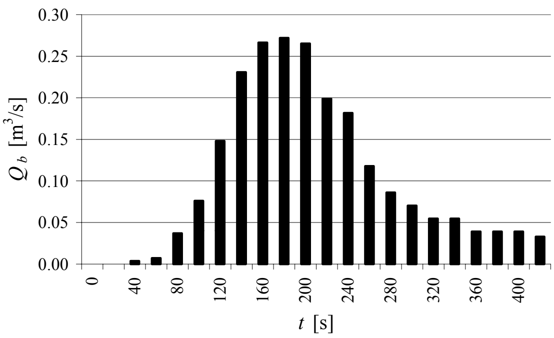

Fig. 8 Discharge through the breach

4. Sediment transport formulae

This chapter summarises sediment transport formulae that are included in the paper.

Meyer-Peter and Müller [7]

bed-load transport: $$q_b = 8 (g\,\Delta\,D^3)^{0.5}\,[\mu\theta - 0.047]^{1.5}\,; \tag{1}$$

Wilson [14]

bed-load transport: $$q_b = 11.8\,(g\,\Delta\,D^3)^{0.5}\,\theta^{1.5}\,. \tag{2}$$

Smart and Jaeggi [10]

bed-load transport: $$q_b = 4\,C^{-0.5}\,(g\,\Delta\,D^3)^{0.5}\left[\frac{D_{90}}{D_{30}}\right]^{0.2}(\tan\beta)^{0.6}\,\theta^{0.5}\left[\theta - \theta_{cr}(\beta)\right]; \tag{3}$$

$$\theta_{cr}(\beta) = \theta_{cr}\,\cos\beta\left(1 - \frac{\tan\beta}{\tan\varphi}\right). \tag{4}$$

Bathurst, Graf and Cao [3]

bed-load transport: $$q_b = 2.5 \frac{\rho}{\rho_s} (\tan \beta)^{3/2} \left[q - q_{cr}\right]; \tag{5}$$

$$q_{cr} = 0.21\, g^{0.5}\, (D_{16})^{1.5}\, (\tan \beta)^{-1.12}\,. \tag{6}$$

Rickenmann [9]

bed-load transport: $$q_b = 3.1 \frac{(g\, \varDelta\, D^3)^{0.5}}{\varDelta^{0.5}} \left[\frac{D_{90}}{D_{30}}\right]^{0.2} \theta^{0.5}\, [\theta - \theta_{cr}(\beta)]\, Fr^{0.6}\,. \tag{7}$$

Engelund and Hansen [4]

total load transport: $$q_t = 0.05\, C^{-1} \left(g\, \varDelta\, {D_{50}}^3\right)^{0.5} \theta^{2.5}\,. \tag{8}$$

Bagnold [1], [2]

total load transport: $$q_t = q_b + q_s\,; \tag{9}$$

bed-load transport: $$q_b = \frac{0.13}{\tan \varphi - \tan \beta} \frac{C\, u^3}{\varDelta\, g}\,; \tag{10}$$

suspended load transport: $$q_s = \frac{0.01}{w_s / u - \tan \beta} \frac{C\, u^3}{\varDelta\, g}\,. \tag{11}$$

Bagnold - Visser [12]

total load transport: $$q_t = q_b + q_s\,; \tag{9}$$

bed-load transport: $$q_b = \frac{0.13}{(\tan \varphi - \tan \beta) \cos \beta} \frac{C\, u^3}{\varDelta\, g}\,; \tag{12}$$

suspended load transport: $$q_s = \frac{0.01}{w_s / u - 0.01 \tan \beta} \frac{C\, u^3}{\varDelta\, g}\,. \tag{13}$$

5. Results of sediment transport calculations and their comparison with experiments

Sediment transport calculations have been done with the formulations of chapter 4 for the experimental conditions:

Comparisons of calculated and experimental values of load transport q are showed on Figures 9 to 12. Minimal and maximal ratios of calculated and experimental values of load transport from all four experiments are stated in the table 1. Average values of the ratios for each formula are included in the table as well.

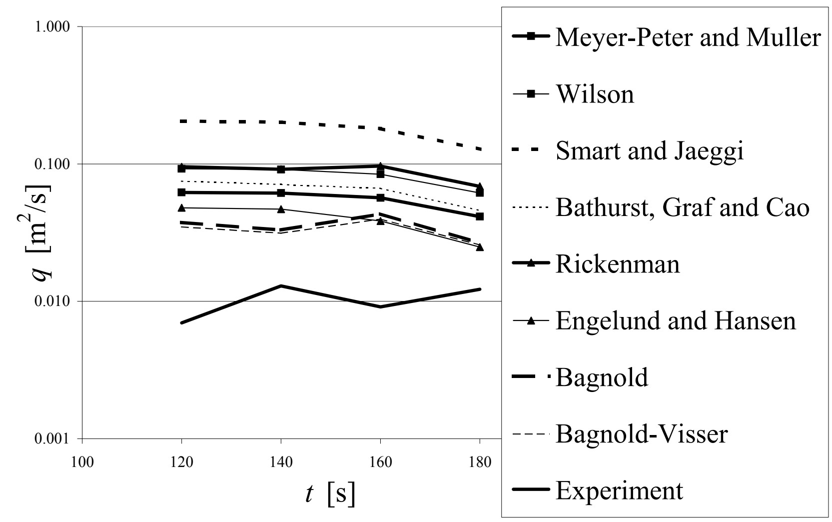

Fig. 9 Comparison of calculated and experimental values of load transport (21st May 1999)

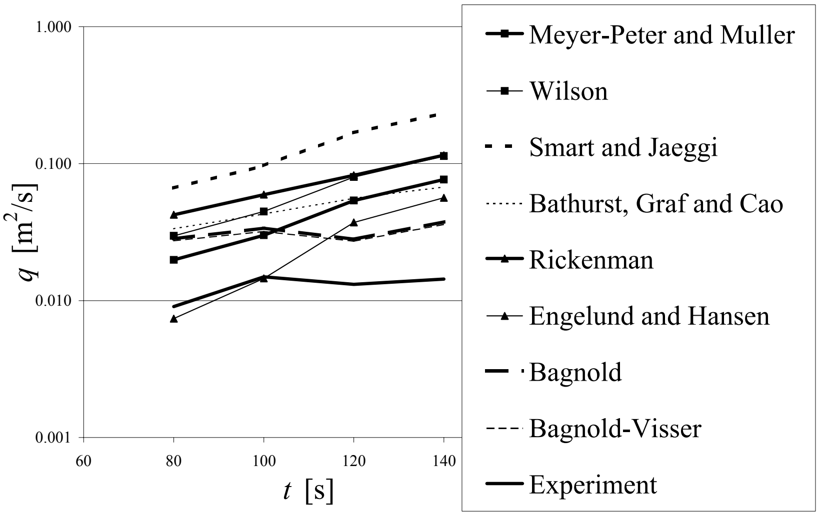

Fig. 10 Comparison of calculated and experimental values of load transport (27th May 1999)

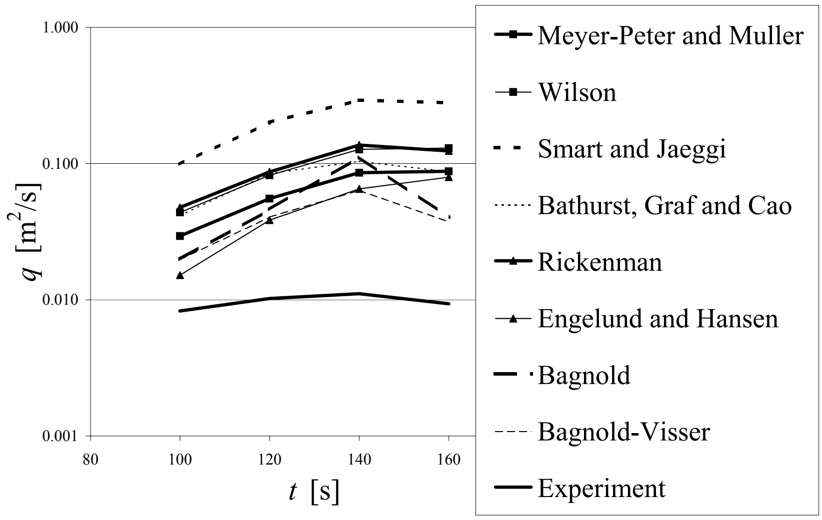

Fig. 11 Comparison of calculated and experimental values of load transport (7th June 1999)

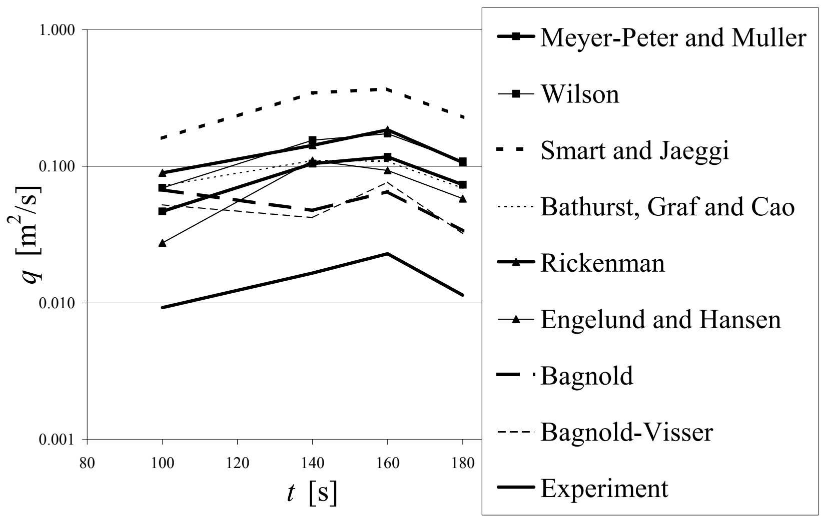

Fig. 12 Comparison of calculated and experimental values of load transport (29th July 1999)

Meyer-Peter and Müller [7]
The rates of sand transport calculated with (1) are 2.0 to 9.4 times larger than the experimental values.

Wilson [14]
The formula (2) predicts 3.0 to 13.9 times larger sand transport than the experimental values.

Smart and Jaeggi [10]
The formulae (3) and (4) predict sand transport rates that are 6.5 to 30.0 times larger than the experimental rates.

Bathurst, Graf and Cao [3]
The formulae (5) and (6) overestimate the experimental sand transport rates by a factor ranging between 2.9 and 10.7.

Rickenmann [9]
The calculated sand transport rates are 4.0 to 13.7 times larger then the observed rates.

Engelund and Hansen [4]
The formula (8) predicts sand transport rates that are 0.8 to 8.5 times larger than the measured rates.

Bagnold [1], [2]
The rates of sand transport calculated with Bagnolds' formulae (9) to (11) are 2.1 to 10.1 times larger than the experimental values.

Bagnold - Visser [12]
The agreement between calculated and experimental values is the best in comparison to the other formulae (calculations are 2.1 to 5.8 times larger than the experimental values).

Tab. 1 Maximal and minimal ratios ($\frac{q_{\text{calculation}}}{q_{\text{experiment}}}$) of calculated and measured values of load transport

	min	max	average
Meyer-Peter and Müller [7]	2.0	9.4	5.4
Wilson [14]	3.0	13.9	8.0
Smart and Jaeggi [10]	6.5	30.0	17.5
Bathurst, Graf and Cao [3]	2.9	10.7	6.3
Rickenmann [9]	4.0	13.7	8.5
Engelund and Hansen [4]	0.8	8.5	4.0
Bagnold [1], [2]	2.1	10.1	3.9
Bagnold - Visser [12]	2.1	5.8	3.4

6. Conclusion

Most of tested sediment transport formulae predict sand transport much larger than the experimental ones (Tab. 1). The Bagnold-Visser formulae (12) and (13) seem to be most suitable for such type of problem as is the dam breach of a sandy dam.

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8. List of main symbols

C	friction coefficient: $C = \left[\dfrac{\kappa}{\ln(12\, d/k)} \right]^2$	[1]
c	depth averaged sand concentration (by volume)	[1]
c_b	bed-load concentration	[1]
D	mean particle diameter	[m]
D_i	particle diameter such that i % of sediment volume has a diameter smaller than D_i	[m]
d	depth of water	[m]
Fr	Froude number: $Fr = \dfrac{u}{\sqrt{g\, d}}$	[1]
g	gravity acceleration	($g = 9.81$ m/s^2)
k	roughness factor ($k = 3\, D_{90}$ for $\theta < 1$; $k = 3\, \theta D_{90}$ for $\theta \geq 1$)	[m]

q	flow discharge (specific)	[m^2/s]
q_b	bed-load transport (specific)	[m^2/s]
q_s	suspended load transport (specific)	[m^2/s]
q_t	total load transport (specific)	[m^2/s]
u	depth averaged flow velocity	[m/s]
w_s	sediment fall velocity: $w_s = \frac{1}{18}\frac{g\,(\rho_s-\rho)\,D^2}{\upsilon\,\rho}$	[m/s]
β	inclination angle of breach slope	[^{o}]
Δ	specific density: ($\Delta = \frac{\rho_s-\rho}{\rho}$)	[1]
φ	friction angle	[^{o}]
κ	constant of von Karman	(κ = 0,54)
μ	the ripple factor (μ = 1 for a plane bed, μ = 0 for ripples and dunes)	[1]
θ	the Shields' particle mobility parameter (Sields number): $\theta = \frac{\tau}{\rho\,g\,\Delta\,D}$	[1]
θ_{cr}	critical value θ for incipient motion	[1]
ρ	water density	[kg/m^3]
ρ_s	sediment density	[kg/m^3]

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The research was supported by MSM 261100006 and by Grant Agency of the Czech Republic, project No. 103/02/P131.

So_2 And No_x Emmission From Hot-Water Boiler Working on Crude Oil During Lower and Higher Capacity

Marija Jankovska [1] , Jasmina Arnautovic[2], Marija Majer[3]

Abstract

The ways of main pollutants formation in the process of fossil fuels combustion are explained. There are presented the methods how the process of formation of SO_2 and NO_x to be decreased and also the methods of reduction of already formed SO_2 and NO_x in the flue-gas. A proper calculation has been done to the emission of SO_2 and NO_x for a concrete case with given elemental composition of the used crude oil and set working conditions of the hot-water boiler. The obtained results of the calculation are compared with the results from measurements for the same hot-water boiler, under similar working conditions, during lower and higher capacity.

1 Introduction

Development and progress of the modern civilization in all spheres have a direct connection with the development of energetics. All phases included in energetics (generation, transformations, transmission, distribution and use of various types of energy) cause, more or less, a negative impact on the environment.

Today chemical energy accumulated in the fossil fuels appears as a basic source for generation of the other necessary types of energy.

According to 1998 data, the use of the conventional sources of energy is as follows: 40% liquid fuel, 26% solid fuel, 24 % natural gas, 7% nuclear energy and 3% hydroenergy.
The use of the fossil fuels causes pollution of all the spheres of environment: lithosphere, hydrosphere and atmosphere. Steam boilers as structures in which the chemical energy of the fossil fuels transforms into heat energy, have a crucial role in pollution of the environment. Solid products of the combustion degrade and pollute the soil, underground and ground flows
and the atmosphere. Waste waters arising from the boiler cleansing, water ash-eliminators, systems for hydraulic transportation of combustion solid products to the depositing site

(1) Grad. Env. Eng., Association for Research and Cultural Collaboration between Macedonia and Croatia, Skopje, Macedonia
(2) Grad. Env. Eng., Association for Research and Cultural Collaboration between Macedonia and Croatia, Skopje, Macedonia
(3) Grad. Env. Eng., Association for Research and Cultural Collaboration between Macedonia and Croatia, Slatina, Croatia

pollute the hydrosphere. Anyway, the atmosphere is a medium which is mostly exposured to the negative impact of the steam boilers operation. All the gas products from the fossil fuels combustion in which in addition to the the basic products of combustion appear other, more or less, toxic substances, have been emitted into the atmosphere. It also receives the unused heat energy from the process, first of all, as a physical heat of the leaving gases, causing a heat pollution of the environment.

Since we can not expect a recent mass use of unconventional types of energy (solar radiation energy, wind energy, energy of high and ebb tide of the sea waves, geothermal energy), the fossil fules will be the mostly used sources of energy in the world. The price of the high-quality fuels is expected to have an increase in future, and on the other hand, the legal regulations in the field of protection of the environment becomes stronger, so the allowable emission of polluting substances in the stmosphere will be limited more and more.

In order to respond to this situation, industrially developed countries, first of all USA and Japan, develop processes how to reduce the harmful products in the atmosphere arising from steam boilers combusting fossil fuels. In that way it is possible to use low-quality fuels, but however to reduce the pollution of the environment to minimum.

2 Methods for Simultaneous Reduction of NOx and SO_2

Methods for simultaneous reduction of nitrogen and sulfuric oxides in various was combine the methods for desulfurization of the outcoming gases and methods for reduction of the nitrogen oxides (primary and secondary).

The procedure presented in Figure (1) unifies the processes of three degrees combustion, recirculation of gases and a dry desulfurizationn of combustion products.

About (80-85)% of the fuel and necessary quantity of air for its combustions have been obtained through the main burners. The other 15-20% have been supplemented by liquid fuel which together with the recirculating gases is introduced over the zone of active combustion, forming a reduction atmosphere in which NOx produced in the first zone is degraded to N_2.
So, $Ca(OH)_2$ which in the form of dry, fine particles is placed over the reduction zone is used as a medium for adsorbtion of SO_2. In this zone there is a finalization of oxidation of the combustive components of the gas fuel through introduction of secondary air. The injection of water after the heat of air, also and shelful in the SO_2 elimination.

This procedure makes possible a reduction of NOx which is bigger than 60%. The reduction of SO_2 is bigger than 50% and it is a result of the partial replace of the fuel with natural gas and of the application of dry process related to the desulfurization of the outcoming gases. If 22% of the used fuel in the process of combustion is a natural gas, and the mollar ratio Ca/S=1,8, the reduction of SO_2 is 58%. This method also makes possible to achieve a significant reduction of CO2 as a result of the partial replacement of the used fuel with gas fuel which has a lower carbon /hydrogen (C/H) ratio.

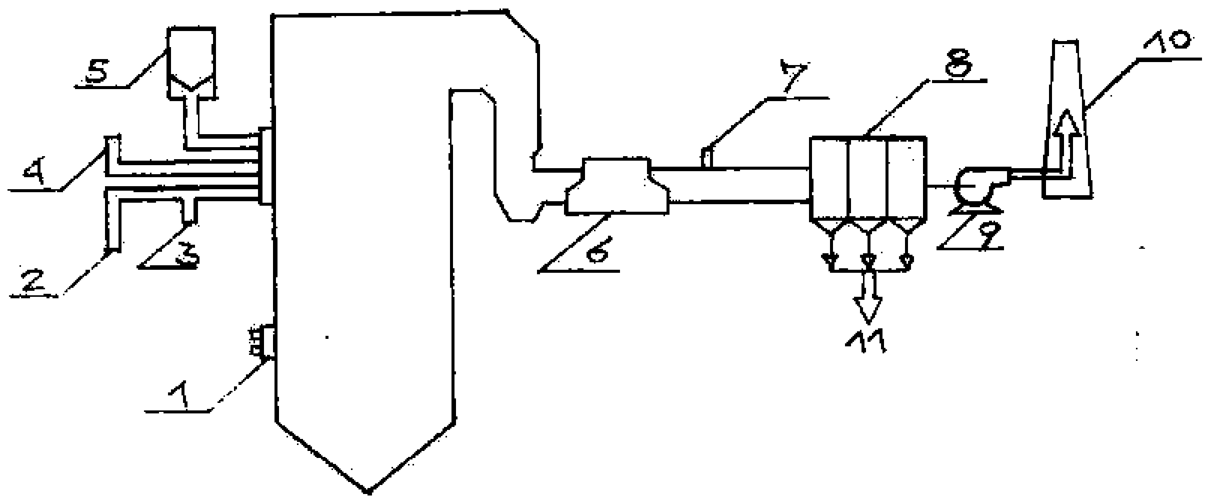

Figure 1- Scheme of a system for simultaneous reduction of NOx and SO_2
1- main burners, 2-recirculating gases, 3-natural gas, 4-secondary air, 5-adsorbent silo, 6-air heater, 7-water for increase of humidity, 8-electrostatic filter, 9-ventilator, 10-chimney and 11-towards the depositing site

Figure (2) shows the process of simultaneous reduction of Nox and SO_2 through which the reduction of their concentration is reduced more than 70%. In order to reduce NOx you should combine the processes of gradual combustion which is possible by means of specially designed burners and SNCR method in which urea is used as a medium related to selected absorbtion. A dry process od desulfurization in which pottasium and sodium compounds is used related to the elimination of SO_2 from the gases. The reduction of SO_2 also can be increased by means of increase of the fuel-gas humidification.

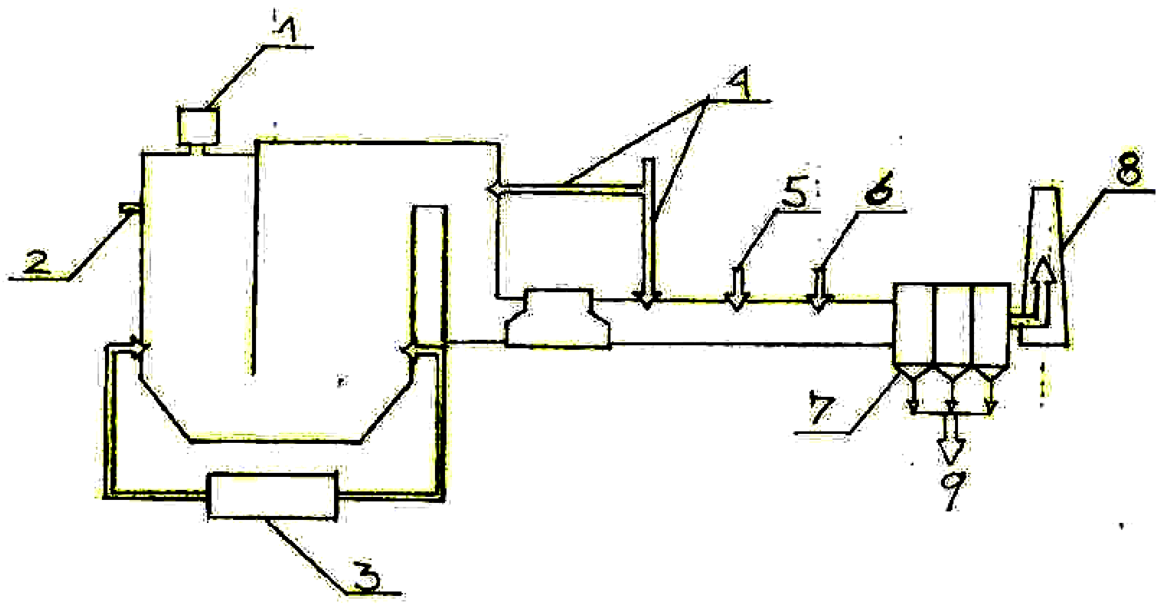

Figure 2-A scheme of the system for simultaneous reduction of NOx and SO_2
1-specially designed burners, 2-secondary air, 3-urea intake, 4-calcium compounds intake, 5-sodium compounds intake, 6-water for humidity increase, 7-ash remover, 8-chimney and 9-towards the depositing site

The fuel and a part of the air necessary for combustion are introduced into the firebox by means of specially designed burners. In this zone a reduction atmosphere is generated ($\lambda<1$). The secondary air necessary for finalization of the fuel combustion is taken in laterally forming a cover around the flame nucleus slowly mixing with it and eliminating the products of the incomplete combustion. This way make possible to achieve a (62-69)% reduction of NOx. For further increase of NOx reduction you should apply the SNCR method. The urea injection is carried out by means of injectors placed at one level, where the temperature profile in the steam boiler makes possible a normal performance of the selective- catalytic process.

Combination of two procedures makes possible for the reduction of NOx to be bigger than 80%.

calcium hydroxid, as a medium for SO_2 adsorbtion can be inserted into the economizering part of the steam boiler reaching reduction of the SO_2 concentration only of 10% or to insert into the gas canal after the air heater reaching a reduction <40%. Much bigger reduction of the SO_2 concentration in the gas you can achieve using sodium compounds. Namely, usage of sodium bicarbonate $(NaHC)_3$ before the air heater or using sodium sesquicarbonate ($Na2CO_3x\ 2H_2O$) before the ash-eliminator which reults in a 70% reduction of SO_2.

It is to be said that joint combination of the SNCR method and the method of dry desulfurization of gases results in their mutual sinergy interaction in the reduction of NO_2 and NH_3 concentrations in the outcoming gas.

The method of simultaneous reduction of NOx and SO_2 shown in Figure (3) is characterized by the fact that it doesn't use absorbents for SO_2 elimination and as a result it doesn't create secondary products.

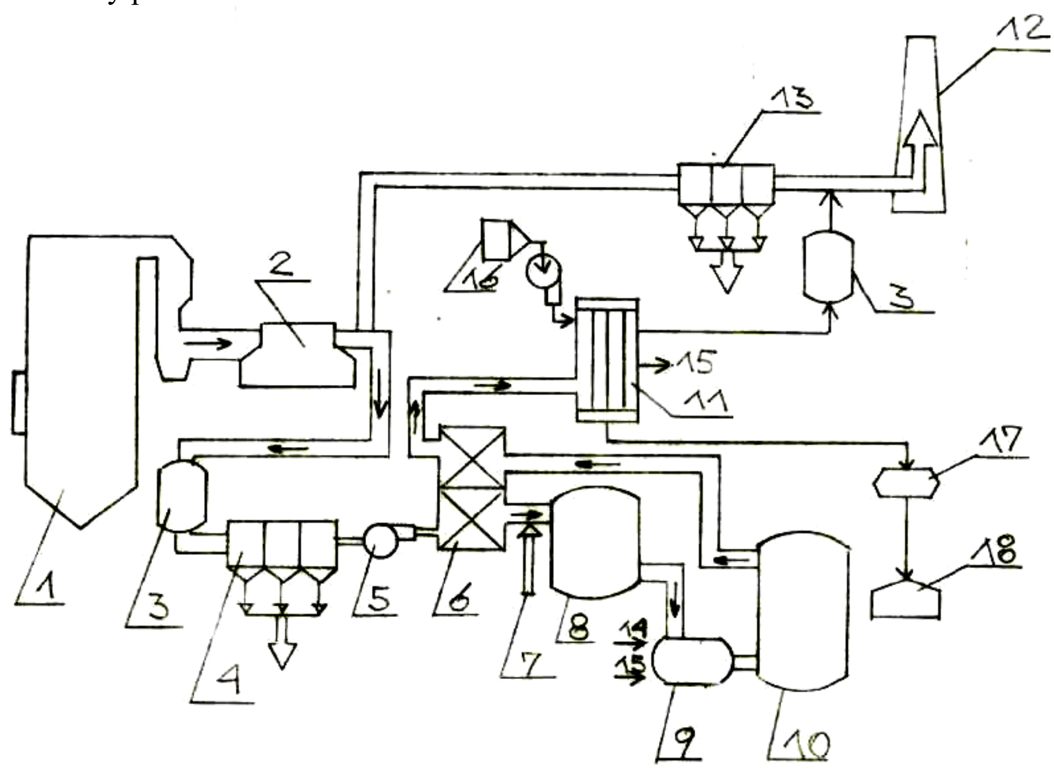

Figure 3-Scheme of the system for reduction of SO_2 and NOx in the coming out gas
1-steam boiler, 2-air heater, 3-auxilliary burner, 4-bag filter, 5-gas ventilator, 6-heat convertor, 7-ammonia supply, 8-catalyzer, 9-auxilliary fire-box, 10-reactor for catalytic oxidation of SO_2, 11-condenser, 12-chimney, 13-electrostatic filter, 14-natural gas, 15-heated air, 16-cold air, 17-acid collector and 18-acid storing tank

Gases coming out from the air heater (2) are inserted into high efficency filter (4), they are heated in the heat convertor (6) and gop to the catalyzer (8) where with a small quantity of ammonia Nox are reduced up to N and steam. After that, gas is heated in the auxilliary fire-box (9) using natural gas and heated air which go to reactor (10) in which SO2 oxidates into SO3. After the reactor the gas is cooled in the heat convertor (6) passing through the condenser (11) where SO3 hydrolizes in concentrated sulfuric acid. Cleansed gas goes tto the atmosphere through the chimney (12).

Nox and SO2 eliminating reactions have been exothermal and the generated heat warms up the gas. This heat and the heat which is delivered to the gas in the first auxilliary fire-box is recuperated in the condenser through heating of the air used for combustion of the natural gas in the second auxilliary fire-box.

3 Indirect Methods for Reduction of the Harmful Components Impact on the Environment

These methods are used in cases when as a result of technical and technological reasons it is impossible to carry out any sanation measure in the framework of the process of combustion, fire-boxes and the steam boiler itself. In that case because of the impossibility to act towards the emission reduction, you should make efforts to reduce the emission as much as possible. **This method is related with the limitation of production and CO_2 emission in the atmosphere which in some West European countries has been regulated by legal regulations.**

Most commonly used procedures are as follows:
- centralized supply of the necessary heat energy and technological steam up to broad circle of users;
- combined processes of production of various types of energy (electrical, heat , technological)
- higher chimnies building;

Through the construction of centralized structures- sources of energy it is possible to provide for the structures to have bigger heat capacity i greater dimensions, creating conditions for usage of some previously mentioned procedures and whose price is not a significant item (neither technical nor economical) in the total financial construction of the energy source. Also the specific price of cost of any type of generated energy and the specific production of pollutants per unit of produced energy (or unit of final product with the energy users) will be low, leaving a possibility for subsequent investments to reduce the energy source impact on the environment.

The combined processes of production of various types of energy reduces the specific production of all pollutants (specific per unit of generated energy). The rapid development of the technology related to the making of elements of such structures makes possible to achieve extremly high coefficients of useful effect of the structures with simultaneous big reduction of pollutant concentrations in the combustion products.

Construction of high chimnies as a principle is the most indirect method whose aim is to intervene in spreading of pollutants in as big area as posible during their emission in the atmosphere and in that way to provide for low emission characteristics of the source of pollution.

The development of the technology for production and segment construction of chimnies and with a bigger cross section of the exit opening it is possible to reach a height of the same up to 60 m. which is, of course, a matter to count on in future. The price of such chimnies is significantly reduced recently, so this procedure has already been applicable for a small installed capacity in the boiler rooms, too.

4 General Views

In order to provide for an efficient reduction of SO_2 and NOx emissions it is necessary for the method for their reduction to be mutually combinmed. Each of these methods solves the problem partially and the desired results are left out. For example, through usage of high quality liquid fuel the emission of sulfuric oxides is reduced but not the emission of nitrogen oxides, as well. It can be increased if the share of hydrogen in the fuel is increased (the relation C/H is reduced creating higher temperatures in the flame nucleus so increasing the production of NOx, as well.

Using fire-boxes of a special construction, the problem of the nitrogen oxides can be solve for ever, but the problem of the sulfuric oxides is to be solved additionally.

The application of methods for reduction of polluting substances in the comming out gas is associated with a large number of limiting factors. For example, missing space necessary for installing of such type of structure can appear as a limiting factor in already constructed energetic structures. These methods also require certain reconstructionms of the steam boiler structure itself.

The price apears as a crucial limiting factor, too. With the convential structures of this type is in the amount from 1 500 000-2 500 000 USD/100 MW. With energetic structures with total installed capacity less than 100 MW this specific price is significantly increased (even twofold) which could lead to the conclusion that this way of solving problems with polluters is justified only with the energetic structures with higher installed capacity. Additional exploitation costs for reagents, maintaining and exploitation of the structure as well as for removal, transportation and depositing of the secondary products. On the other hand, the existence of such a structure makes possible for usage of lower quality fuel containing sulfur up to 5% (in energetic structures where methods for desulfurization are not used, the share of sulfur in the fuel should be maximum up to 2%). In that way, through the difference in the price between a low quality and high quality fuel the investment will be returned during the period of a few years.

Depositing of secondary products (sulphates and sulphites) appear as additional problem with the nonregenerative processes of desulfurization. The most contemporary technological solutions of this process in USA and Japan forecast processes of secondary products oxidation, as well, so that as a final product of the process you can produce gypsum for sale on the market. In addition to this benefit, these structures require much more smaller space in comparison with the conventiopnal structures of this type and their price is almost twofold lower.

5 Conclusion

In order to provide for an efficient reduction of SO_2 and NOx emission it is necessary to combine the procedures for their reduction. The problem related to reduction of emission should be solved for each case separately because only in that way you can come to the most appropriate solution which gives the desirable results. The choice of procedures and methods for solution of the problem depends on the conditions for energetic structure (steam boiler), on the conditions in the energetic structure as a whole, as well as on the conditions offered on the market related to fuels. The chosen procedure besides the fulfilment of standards related to impact on the environment is should be economically justified, as well. The construction of special structures for cleansing of the flue-gas from SO_2 and NOx is justified only with energetic structures whose capacity is bigger than 100 MW. These structures include ash-remover which removes the solid products of the combustion from the flue-gas so that you can solve this problem, too. Using such a structure, as already said, it is possible to use low quality fuels at low price as well as the possibility for possible commercial utilization of the secondary products.

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Flood Problems in Gdańsk

Teresa M. Jarzębińska

Gdańsk University of Technology, Faculty of Hydro and Environmental Engineering, Department of Hydraulic Structures and Water Resources Management, Narutowicza 11/12, 80-952 Gdańsk, Poland, tjarz@pg.gda.pl

Abstract
Gdańsk is situated on the Baltic coast at the mouth of the Vistula River. Gdańsk is a very important economic, cultural, scientific and industrial centre. The city is the most flood endangered urban agglomeration in Poland. Flood danger may come from a storm on the Baltic Sea, from the main channel of the Vistula in case of very high discharge and breach of flood dykes, and from the moraine hills situated to the south of the city in case of heavy precipitation. Up till now it has been considered that the highest danger may come from the two first directions. In 2001 it appeared, however, that catastrophic rain on the area of Radunia Channel catchment caused flood which resulted in considerable economic losses in Gdańsk. During winter 2002/03 it appeared also that serious flood danger can be caused by sedimentation cone in the mouth of Vistula. This place is prone to the formation of ice jams, which will result in the increase of water elevation along the section of main Vistula channel. In the paper the flood hazard of the city of Gdańsk, causes, run and consequences of the flood from July 2001 are presented. Proposal of actions for the future are discussed and the means presently undertaken to limit this flood danger.

1 Introduction

Gdańsk is the city of more than a thousand years history. It is situated on the Baltic coast at the mouth of the Vistula River (Fig. 1). Gdańsk has 450 thousand inhabitants and is a very important economic, cultural, scientific and industrial centre. The city is situated on the lowland area and is one of the most flood endangered urban agglomerations in Poland. Any flood in Gdańsk, results in considerable economic losses. Its lowland position causes that removal of floodwaters is very difficult and takes a long time.
Vistula is the largest river of Poland, second largest (after Newa) river of the Baltic Sea catchment. The length of Vistula is 1047 km. The total Vistula catchment is 194 000 km^2 of which 169 000 km^2 (87%) is within Polish boundaries. The catchment of Vistula which is in Poland occupies 54% of the countries territory. The average discharge of Vistula to the sea (1951-1995) is 1080 m^3/s which results in an annual outflow of 32 km^3. In the dry and wet year these values amount from 20 to 44 km^3.
The Vistula delta was always under severe threat of floods, which became more and more dangerous in connection with economic development of the area and constantly increasing population. Majority of floods resulted from unfavorable ice conditions which even during low discharges formed ice jams and severe floods. The main engineering action for diminishing flood damages was the construction of flood dykes and polders. These were inundated by flood waters thus preventing other valuable regions from flooding. This situation was considerably worsened by unfavorable hydrodynamic, ice and meteorological conditions over the area of Gdańsk Bay and Vistula Lagoon. From XIV to XVIII century flood dykes of Gdańsk Vistula, Elbląg Vistula and Nogat were breached nearly 150 times[4]. In XIX century this happened more than 60 times. Essential change in the layout of Vistula

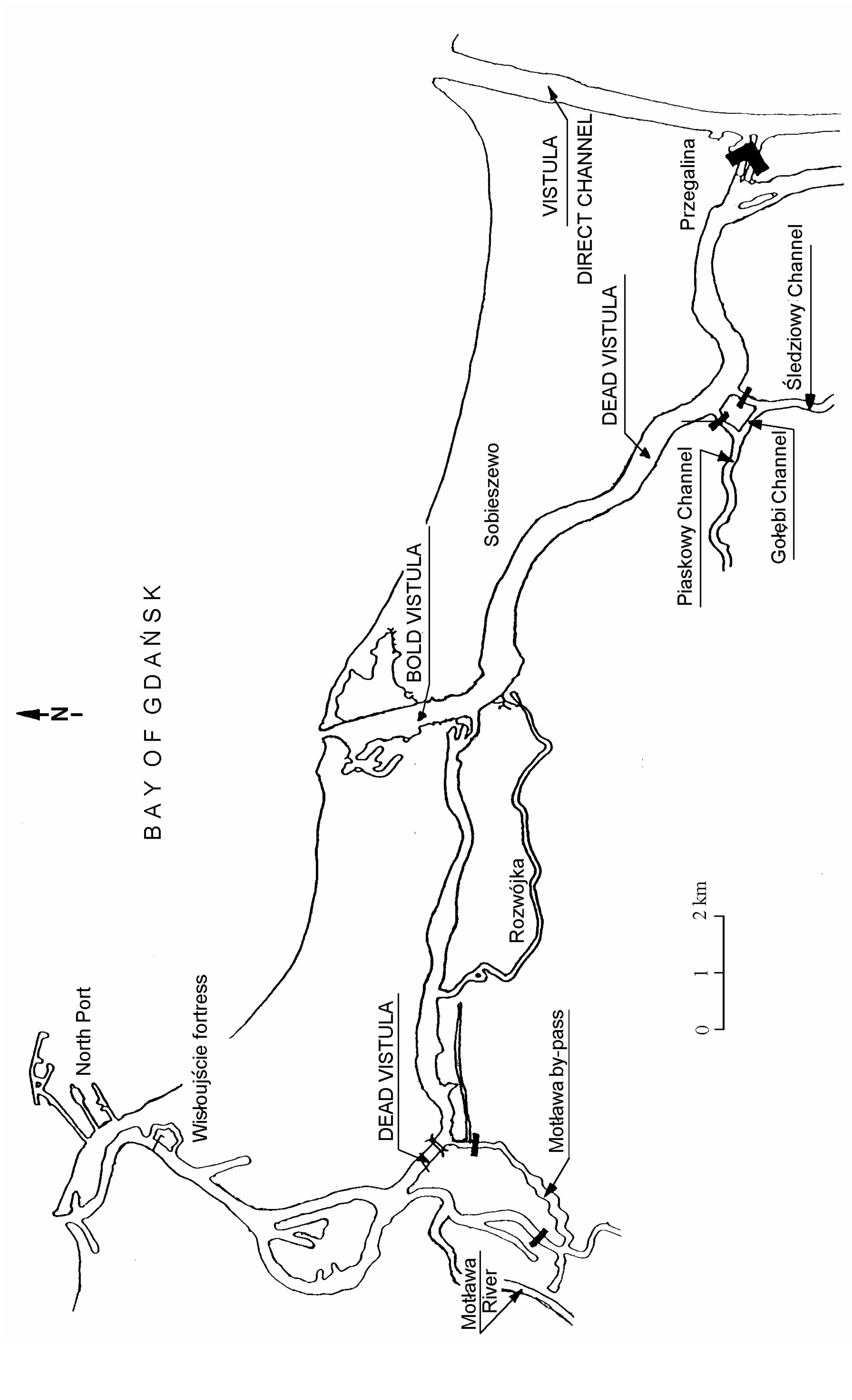

Fig. 1: Hydrographical network of the Dead Vistula catchment[1]

channels appeared in 1840 when flood waters after forming of big ice jam broke the belt of dunes near Górki creating new outlet to the sea, later called Bold Vistula (Wisła Śmiała). The shortening of river length resulted in increased discharge and hydraulic degeneration of Elbląg Vistula. Catastrophic floods from 1855 and 1888 resulted mainly in the decision of artificial channel of Vistula discharging waters directly to the sea. In 1895 the final section of the Vistula was transformed into a direct channel to the sea protected on both sides by flood dykes (Fig. 1). After the formation of direct Vistula channel to the sea the area of Żuławy on both sides of Vistula are protected by means of flood dykes. Since that time, there has been no serious flood in Gdańsk. There was man made flood in 1945 when flood dykes were breached by explosives and the whole area of Żuławy was flooded. However, flood danger still exists[2].

2 Flood danger of the city of Gdańsk

Gdańsk is situated on the lowland and depression terrains at the foot of moraine hills.The area of Gdańsk 262 km^2 is borders from the North by the Bay of Gdańsk and Dead Vistula and from the East reaches main Vistula Channel. The city is under the influence of Motława River and its tributaries. The largest tributary is Radunia River. Specific location of Gdańsk causes that it is endangered to all kinds of floods. They may come from various sources (Fig. 1):

- from the main Vistula channel – these are the main floods caused by the destruction of flood dykes of the estuarine section of the river as the result of high water stages (overflow over the crest of flood dyke) or their long time of duration erosion of main body of the dyke, and ice jam floods;
- from the Baltic Sea, and in particular from the Bay of Gdańsk – storm floods and ice jams which appear on the sedimentation cone at the mouth of Vistula River to the bay;
- from the Dead Vistula and its tributaries – unfavorable superposition of spring flood (snow melting and precipitation) and storm may result in overflowing of water above flood dykes of the river;
- from the Radunia Channel and streams from the upper mountain slope – mountain streams cause flood, which has a very severe run and causes significant material losses, and breaching of the channel dyke results in flooding of city terrains being in depression
- inside polder flood danger, which may appear in case of long lasting rains, when the inflow to the pump stations exceed the capacity of drainage facilities;

the source of danger may be also the failure of technical facilities on the streams in the given region (storm gates, flood gates, navigation locks, pump stations, weirs, reservoirs, siphons, culverts etc.)[3].

Flood in Gdańsk in July 2001

3.1 Precipitation regime in Gdańsk

The precipitation measuring network in Gdańsk consists of only three stations operated by the National Meteorological Service. They are unfortunately located on the city boundaries (Fig. 2), and it is, therefore, very difficult to estimate spatial distribution of precipitation over the whole area of Gdańsk. Precipitation in Gdańsk is highly non-uniform in space and time. There were frequent intensive rainstorms, which, however, covered only small area. The average annual precipitation in Gdańsk is about 600 mm and that for July is 68 mm. In recent 4 decades it was observed that maximum daily precipitation for a particular year occurred in July. Up till 1980 the maximum values of precipitation for July were much lower than those for the year. The maximum daily precipitation, which appeared up till 2001, was in July 1980 and amounted to 80 mm. Since 1980 the maximum values of precipitation for July coincide

with those for the year. This may indicate a certain change in climate. However, the multiyear precipitation records indicate that in general, maximum values of daily precipitation did not exceed 60 mm. This happened only twice - in 1980 and in 1984. The value of daily precipitation in 2001 substantially exceeded previous records (W. Szpakowski – privat communication).
On the 9th July the observed amounts of precipitation were as follows:

Table 1

The amounts of precipitation on the 9th July 2001

measuring station	Precipitation during 4 hours (14.00-18.00)	Daily precipitation on 9th July	Average precipitation for July
Rębiechowo airport	72 mm	124 mm	68 mm
North Port	80 mm	118 mm	66 mm
Świbno	9 mm	72 mm	68 mm
GUT	90 mm (15.00-18.00)	132 mm	

The daily amount of precipitation in the whole city was 120 mm, this being estimated to have a probability of 0.5 to 0.3 % (once in 200 - 300 years)[5].

3.2 Damaged objects and flooded areas

As the result of intensive precipitation, the following objects were destroyed or inundated (Fig. 2):

- the dike of Radunia Channel was breached in 6 places, which resulted in flooding of the area of the city situated in the depression on the side of the channel and the main road,
- the main embankment of the small Srebrzysko reservoir on the Strzyża Stream was breached, which resulted in a severe flood along the main street and flooding of the crossing on the road between Gdańsk and Gdynia.
- The embankment of Kłodawa River was breached over the length of 30 m,
- two main roads approaching Gdańsk from the West turned into torrential rivers,
- Gdańsk main railway station was flooded, which caused one week's break in traffic.

Losses in the city infrastructure were estimated to about 50 mln. USD. More than 300 families were affected by the flood (damaged houses, loss of property). More than 5000 people received special status of persons affected by floods, who deserve social assistance[5].

3.2.1 Damage of right-hand dike of Radunia Channel

In Pruszcz Gdański node Radunia River divides into two branches: the Old Radunia River and the Radunua Channel (RCh). The Old Radunia River flows through Żulawy 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 in Gdansk and defensive moats. The length of the RCh is 13.5 km^2 and its catchment totally on the left hand side amounts to 55 km^2. The whole catchment was turned into housing schemes. Channel discharge amounts to 7 m^3/s and its capacity was estimed at ca. 21 m^3/s. The Old Radunia River flows within flood dykes, has no tributaries and has practically transition character for flood discharges. However the RCh has many tributaries from the left part of the catchment. These are: Siedlecki Stream, Oruński Stream, collector Ø = 1000 mm, storm collector Ø 1400 mm, Maćkowy Stream, St. Adalbert Stream, and Rotmanka Stream. All these streams flow into Radunia Channel from the left bank, perpendicular to the channel (Fig. 3). At present the Channel is partly regarded as retention reservoir of the total capacity

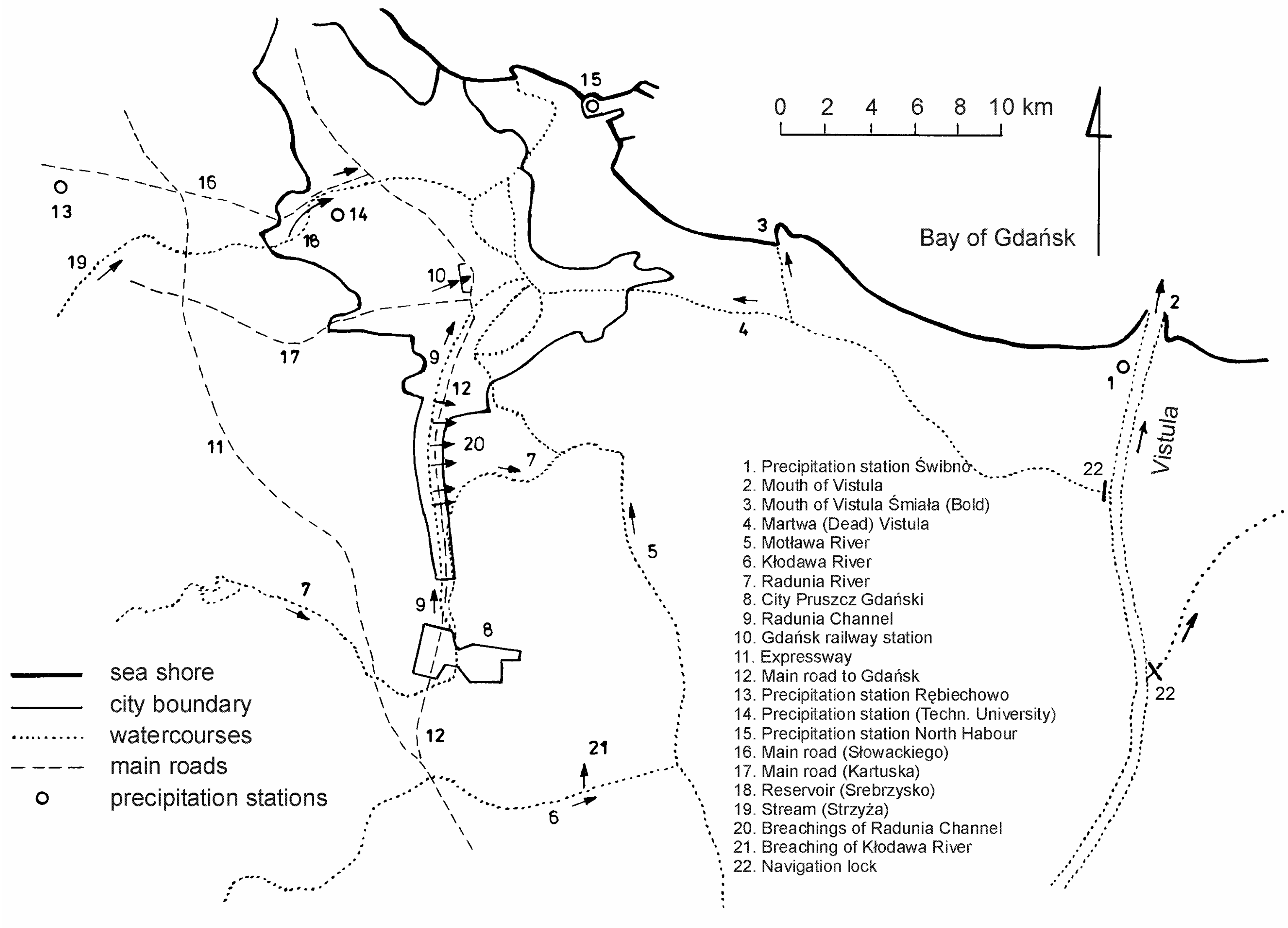

Fig. 2: Scheme of the Gdańsk area (the precipitation stations and damaged structures are marked)

about 100 000 m^2. Water surface in the Channel is about 1.5 m above the terrain situated along the right bank.
As the result of severe water inflow to the Channelin July 2001 its right side dyke was breached in 6 places between km 4.150 and 9.000. The total length of the breaches in flood dyke was about 200 m. As the result of breaches of the dyke 3 residential districts were flooded. Estimated calculations indicated that the inflow to the Channel Q = 120 m^3/s and the volume of water which discharged into the Channel during 4 hours amounted to 1.76 mln m^3. Such inflow the Channel could not receive and transfer in a safe way.

3.2.2 Damage of Srebrzysko Reservoir
Retention Reservoir Srebrzysko is situated on the Strzyża Stream in the district Gdańsk Wrzeszcz. The Stream flows partly as open channel and in some parts as pressure conduit. It conveys its waters to the Dead Vistula about 5 km downstream from the reservoir. The capacity of the reservoir is about 30 000 m^3. The reservoir was designed for the $Q_{10\%} = 5$ m^3/s. On the 9 July the overtopping of the reservoir occurred and about hour 23 its head part was breached. Rapid emptying of the reservoir resulted in the discharge of large amounts of water along the Słowackiego Street, which became a torrential stream. This was indicated by by thorn out pavement slabs and asphalt layer from the street. As the consequence the communication node on the Alee Grunwaldzka and numerous basements of the houses along the Strzyża Stream were flooded. For many hours the main traffic line between Wrzeszcz and Oliwa was stopped.

3.2.3 Breaching of left-hand dike of Kłodawa River
Kłodawa is the second largest, after Radunia effluent of Motława. In its final section Kłodawa has flood dykes on both sides. The breaching of flood dyke happened after midnight of 11 th July 2001 at the distance about 3.6 km from Motława. Running water rapidly made the gape in the dyke of the length 35 m. The flood dyke was eroded in about 70%. The water running through the breach flooded agricultural land of the neighboring villages.

4 Action for the future

Identification of flood danger in Gdańsk is the first step on the way to undertake real investment actions, which aim to increase the security of the city and its inhabitants. Before taking any action concerning construction of hydraulic structures, it is necessary to increase the amount of measuring points of precipitation and water stages within the city area. These should be automatic measuring points with direct transfer of information to the center. It is also necessary to develop a mathematical model including the whole area of Gdańsk, taking into account all streams, and making possible the analysis of various variants of dangers and protection against them.
The studies should include:

- analysis of actual hydrologic characteristics of all streams,
- the assessment of the possibility of safe transfer of flood waters at the existing state of river network,
- determination of the maps of flooded areas (extent and depth) for the discharges of various probabilities,
- future concepts to improve flood protection of the city (technical and non technical means of protection),
- development of the monitoring and warning systems of inhabitants against flood danger.

The first stage of the above mentioned tasks is the “Model of the distribution of water in the Gdańsk node in order to secure flood protection of the Dead Vistula”. This model is presently developed by the interdisciplinary team of scientists and designers.

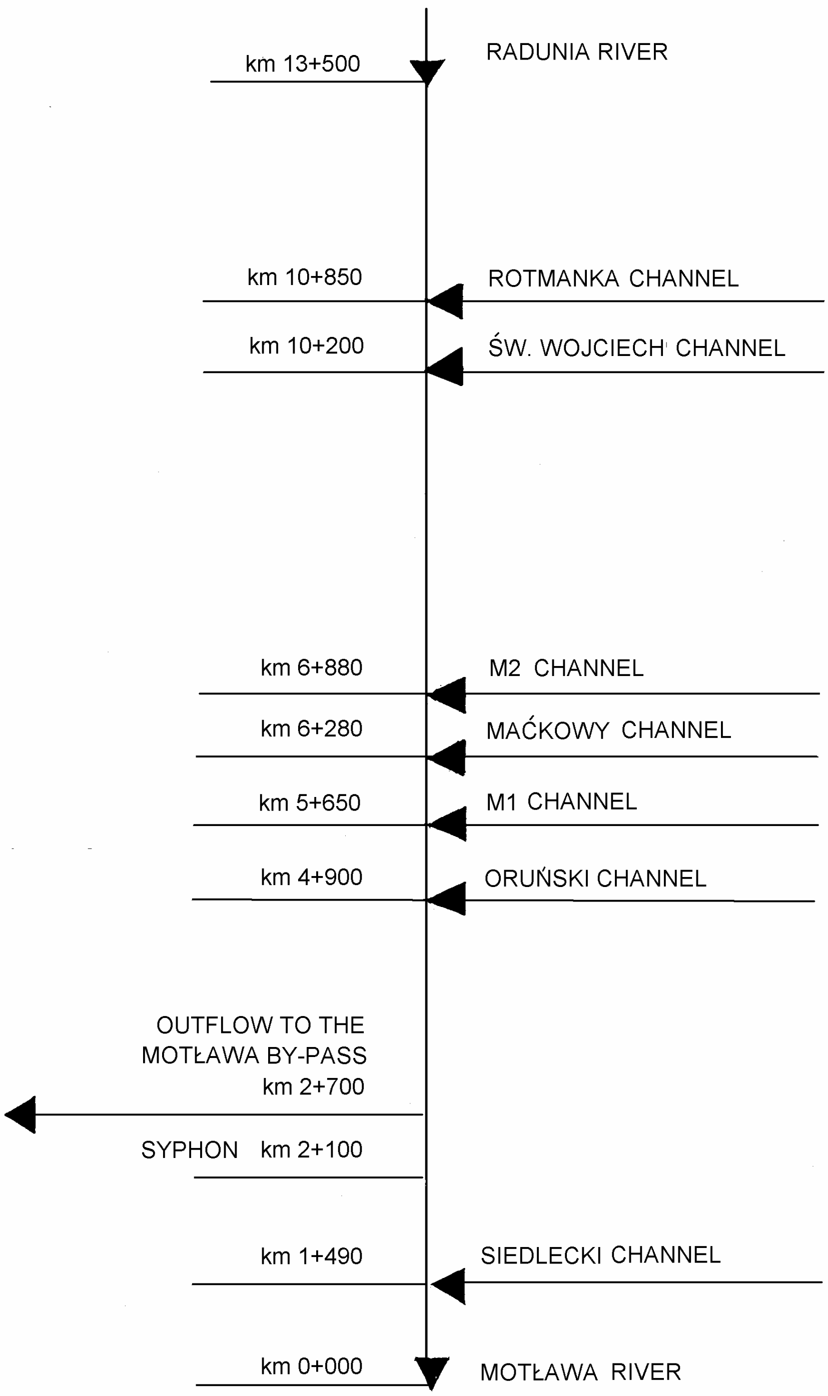

Fig. 3: Scheme of the Radunia Channel and its tributaries

5 Conclusions

The history of Gdańsk indicates that:

- floods are the natural and very often appearing hydrological phenomenon on this area;
- human beings for many years ignored the threatening danger, and with his activity significantly increased the scale of the danger. Urbanization of the upper terraces of Gdańsk, liquidation of retention reservoirs on the streams significantly deteriorated the conditions of retention in the catchments of the streams;
- determination of the frequency of the appearance, magnitude and the range of danger is not possible. Also we cannot estimate from which direction the danger will come;
- the city of thousand years history, which always lived on water, cannot be shifted in space. Therefore we must realize that flood danger will always exist. Because we cannot protect ourselves against flood we have to learn how to live with it, taking into account our knowledge and technical and organizational possibilities.

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The Changes of Groundwater Quality on the „Czarny Dwór" Intake in the Light of Polish-Swedish Investigations

Beata JAWORSKA-SZULC
Małgorzata PRUSZKOWSKA
Maria PRZEWŁÓCKA

Technical University of Gdańsk, Faculty of Hydro and Environmental Engineering, Department of Hydrogeology and Engineering Geology, Gdańsk, Poland, bejaw@pg.gda.pl, mpru@pg.gda.pl, mprzew@pg.gda.pl

Abstract

The „Czarny Dwór" groundwater intake is one of the main sources of water for Gdańsk. It is situated on the Marrine Terrace and exploits mainly Quaternary aquifer, but also tertiary and cretaceous. The water is generally of good quality although some undesirable changes were observed in the quaternary aquifer during almost 40 years of exploitation. The changes are caused by developing urbanization, former high pumping rates and natural conditions such as vicinity of sea and type of unconfined aquifer. They are expressed in an increased amount of Cl^-, SO_4^{2-}, N/NH_4^+ ions and total hardness.

Introduction

The "Czarny Dwór" groundwater intake is one of the most significant well fields in the Gdańsk water supply from almost 40 years. The line of wells, 3200 m long is situated 600 –1500 m from the sea shore. The location of the wells on the Marine Terrace is connected with high quality and availability of water from Quaternary deposits. The abundance of water in this area was noticed already at the beginning of XX th century by German hydrogeologist Thiem. The development of the surrounding urban area though, has brought some threats to the water extracted from the Quaternary aquifer. Groundwater risk assessment and protection of the water was the aim of different investigations and studies carried out for many years.

From 2002 a team from Royal Technical University of Stockholm together with polish hydrogeologists and students from Technical University of Gdańsk has also contributed in those studies. The first year of cooperation was concentrated on the assessment of salinization from the Baltic Sea, mainly using geophysical methods and also on the influence of the allotment gardens situated in close vicinity of the intake till last year.

Hydrogeological conditions

There are three aquifers containing water of good quality on the Marine Terrace: Quaternary, Tertiary and Cretaceous.

The Cretaceous aquifer is connected with widespread hydrogeological structure called Gdańsk Artesian Basin. It consists of fine – grained, quartz – glauconite sands of Coniacian and Santonian. The aquifer, 150 m thick occure from about 150 m below ground and their filtration coefficient is 0.2 – 2.3 m/h. The recharge zone of the aquifer includes the area of the Kashubian Lake District where the head of the aquifer is approximately 150 m asl there. The discharge zone of the basin is located on the terrains of the Vistula River Delta and the Bay of

Gdańsk together with the Marine Terrace. The original piezometric surface drops to 15 m a.s.l. there.

The Tertiary aquifer in this area consists of varigrained sands in some places with gravel and phosphate concretions of Oligocene. The top is about 60 –75 m a.s.l., the thickness reaches 30 m and the filtration coefficient varies from 0.4 to 1.2 m/h. The recharge zone is on the Kashubian Lake District where the original piezometric surface amounts to 120 m a.s.l. and the zone of discharge is the same as for the Cretaceous aquifer with the natural head of piezometric level at about 8 m a.s.l.

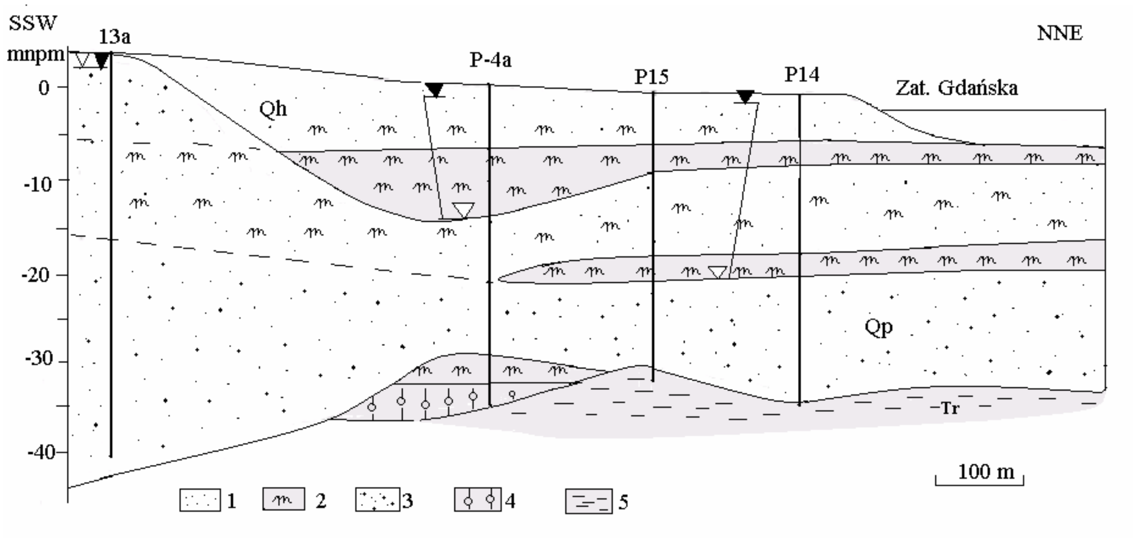

Fig.1 The hydrogeological crossection through the Czarny Dwór groundwater intake.
1 – sand, 2, 5 – silt, 3 – sand with gravel, 4 – boulder clay

The Quaternary aquifer is composed of fluvioglacial sands and gravel originating from Middle Polish glaciation. They are covered by Holocene sands of outwash cones, and along the coast by marine sands. Those deposits are interbeddeed with silts and silty sands in the part of area adjoining the sea. The aquifer is 25 – 35 m thick and the filtration coefficient amounts from 0.2 to 6.1 m/h. In natural conditions the water table, or in places the piezometric surface, varied from 2 to 4 m a.s.l. In some parts of the area the Quaternary aquifer is covered by peat but in a considerable region there is no any confining layers at the roof, so it is rather exposed to contamination. The recharge is mainly lateral, by a direct inflow from the Kashubian Lake District, and also in some extent through atmospheric precipitation and the ascension of water from Tertiary aquifer. In natural conditions the groundwater table along the coast was about 2 m a.s.l. which means that the drainage was taking place a few kilometers offshore [1].

The natural circulation of groundwater was disturbed by exploitation. The most intensive output in history took place in 1984 – 85 when in the centre of depression cone the water table was lowered to approximately 3.5 m a.s.l. At present, the water table rises in all three exploited aquifers. The groundwater level in Quaternary aquifer reaches in some places the level of terrain like it was in natural conditions.

Threats to the groundwater quality and the metodics of investigations

One of the threats to the exploited Quaternary water is close vicinity of the sea. The intensive exploitation in the period of 1965 – 1985 resulted in developing of extensive depression cone spreading under a part of the Bay of Gdańsk. This situation caused intrusion of salt water to the Quaternary aquifer in 80-ties and 90-ties. Another factor unfavourable for the quality of groundwater is lack of confining layers in the upper part of the aquifer in a 1 km wide area surrounding the line of wells, which means exposure to potential contaminant from the surface.

An obvious threat to the well field is also a development of the surrounding urban area. The risk is connected with expending road net of heavy traffic, development of sewage system and other underground installation, existence of such objects as waste disposal plant, hospital, transport base, petrol stations, garages and factories and also allotment gardens which had existed for 20 years in the closest surroundings of the intake. The gardens may had constituted a risk of pollution due to excessive use of fertilizers and insecticides by the unprofessional owners. Besides it was stated in the past, that on the area of the gardens there existed some incorrectly constructed shallow wells used as cesspools or compost tanks. Because of the risks the decision on the gardens liquidation was taken recently. From last year tidying up works are carried on and the action to improve the recreational value of this part of the area is taken, together with land reclamation works necessary because of the present high groundwater level.

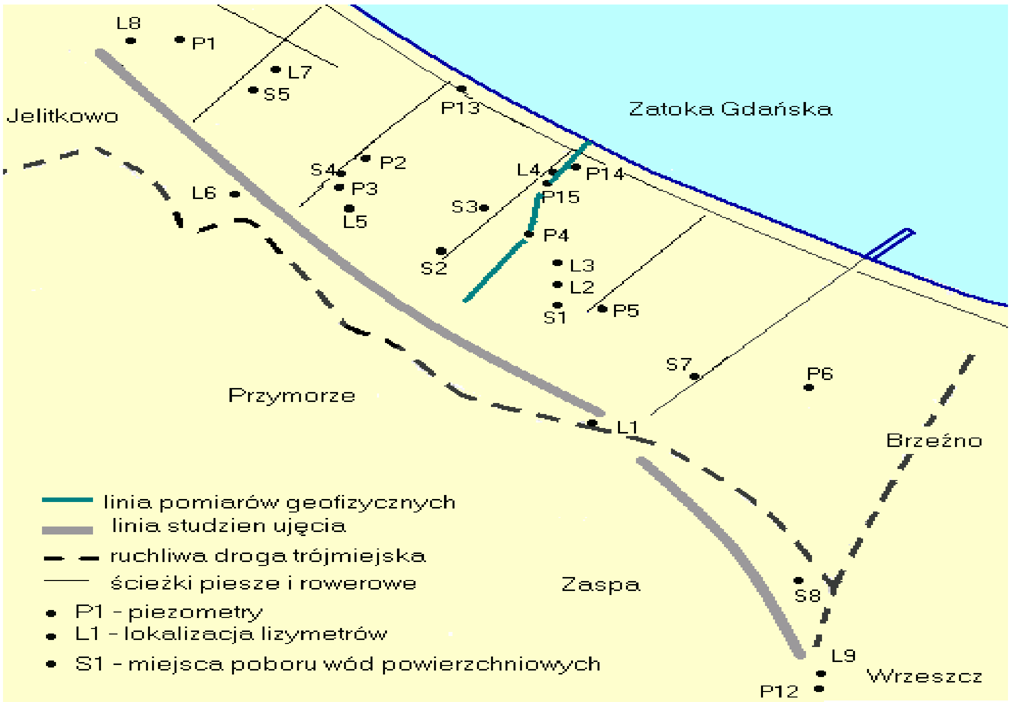

Fig.2 The scheme of the intake with lokalization of sampled points.

In April 2002, when the field work was carried on, there was still a disorder connected with destroyed gardens, arbours, fences and with the risen groundwater causing pounding of the terrain. In the days 24th – 27th of April some samples were taken in order to make chemical analysi. These were 9 samples of groundwater taken from piezometers located between the line of wells and the sea shore, 7 samples of surface water from drainage ditches and floods and 5 samples of soil water taken from lysimeters. In the samples taken to laboratory the following parameters and ions were measured: colour, alkalinity, Cl^-, SO_4^{2-}, N/NO_3^-, Ca^{2+}, Mg^{2+}, Na^+, K^+, Fe^{2+}, Mn^{2+}. Electrical conductivity and pH was measured in the field. The points of sampling and the location of the intake is shown on Fig.2 . The Fig. 3 and 4 shows concentrations of chosen parameters in all three types of water.

In order to detect possible salt water intrusion in the study area, a geophysical method - Vertical Electrical Sounding (VES) was applied. Thanks to differences in resistance between salt and fresh water it is possible to detect boundaries of salt water occurance using geophysical methods. The profiles were made at eight different locations (Fig.2) and the Schlumberg's electrodes system was adopted in the measurements.

The results of geoelectrical sounding

Tab. 1 Values of the apparent resistivity at the 8 measured points

Głębokość m	S13a	P4a	P15	P14-P15	P14	Bicycle path	beach	linia brzegowa	zakresy oporności
1.5	3638	173	880	663	1652	3146	588	13	0-50
2.2	3758	119	649	621	886	2530	654	12	51-200
3.2	3580	86	455	422	920	1671	594	11	201-500
4.6	2661	78	256	239	296	920	453	11	501-5000
6.8	1445	81	136	98	134	325	260	13	5001-
10	549	78	103	94	56	157	157	16	Ωm
15	353	78	177	113	85	177	127	21	
22	91	91	106	85	91	91	182	23	
32	45	97	109	106	97	97	193	32	
46	106	126	146	233	798	333	1197	40	
68	160	203	65	-	-	857	1743	-	
100	628	7854	361	-	-	198	2199	-	

The figures shown in Tab.1 as apparent resistivity are expressed in Ω m. In all the profiles (except for the measurements obtained at the beach) there is a clear reduction of the resistivity values at the depths of 0.5 – 4 meters, which is due to the level of groundwater table. At the depth of approximately 7 –11 meters in all the profiles except of the one at the beach front there is an increase of resistivity values, which may be explained by the fact that there is a change in the layers from sand to sand mixed with silt at this depth. The main result of the measurements do not show any salt-water intrusion.

The results of chemical water analysis

The analysis on the samples were performed in the laboratory of Royal Technical University in Stockholm. Figures 3 and 4 show the results concerning SO_4^{2-}, Cl^- and Na^+. All the determined ions and compounds are compiled in Tab.2, where the minimum and maximum values are shown in numerator and the average value in denominator. Symptomatic is high and variable concentration of sulphates in all kinds of waters but especially in surface water (31 – 610 mg/dm^3) This can be explained by both natural factor and human activity. Sulphates can be formed naturally in an aeration zone in the presence of organic matter. In this case the organic matter is present mainly in peat covering more or less continuously the exploited aquifer. Concerning antropogenic origin one must keep in mind the recent history of the area. The activity in the allotment gardens included probably using different fertilizers such as potassium sulphate or dung and compost. Besides a heating plant using coal existed for many years in the close vicinity of the intake. Sulphur compounds emitted to the atmosphere can infiltrate to the ground with precipitation, where depending on the red-ox conditions, they were present in different form. According to S. Witczak [3] coal burning is one of serious sources of sulphates in shallow groundwater.

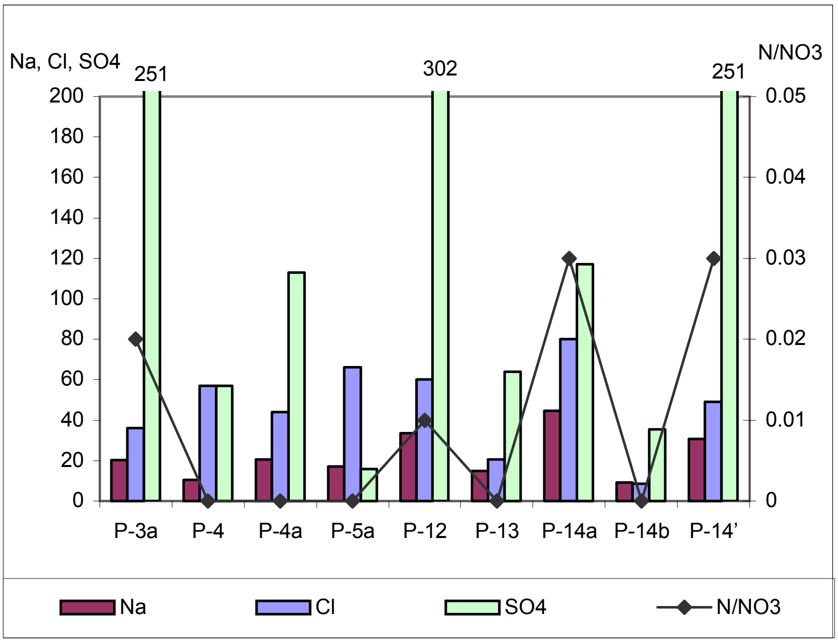

Fig. 3 Concentrations of Na^+, Cl^-, SO_4, and N/NO_3 in groundwater (April, 2002)

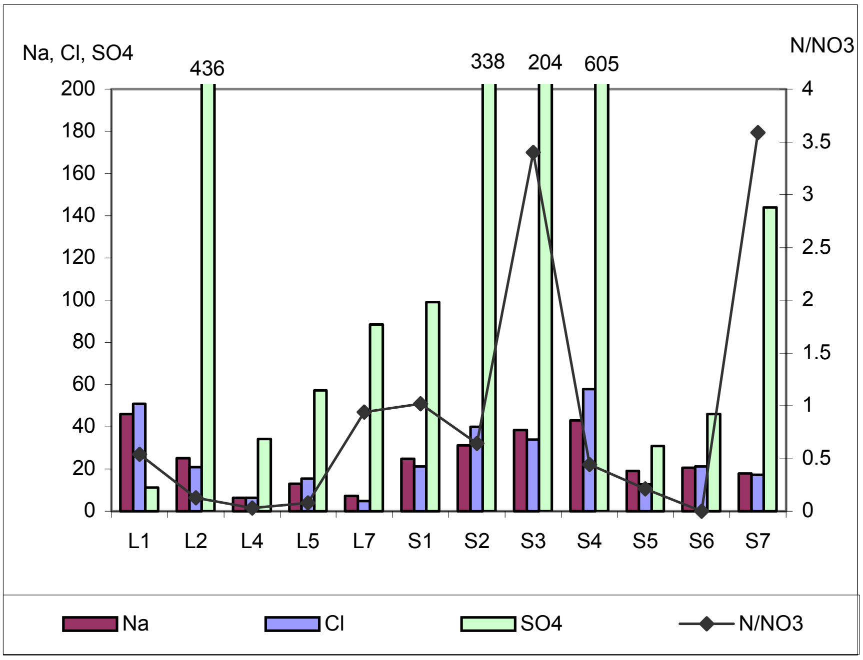

Fig. 4 Concentrations of Na^+, Cl^-, SO_4, and N/NO_3 in surface and soil water.(2002)

The influence of the former allotment gardens is visible mainly in the surface water composition. One of examples can be an increased concentration of potassium (max 40 mg/dm^3) and the other - nitrate nitrogen (max 3.59 mg/dm^3, average 1.33 mg/dm^3) Those ions occur in typical amounts in the groundwater though. It is quite understandable taking into consideration the current hydrodynamic conditions on the concerned area. The hydraulic head in the exploited water bearing strata is higher than this in the shallower water. Consequently the contaminants from the shallow water don't flow into the main aquifer. The situation can be changed in the event of increased yield of the intake. This situation took place in the past.

Tab. 2 Minimum, maximum and average values in waters taken from piezometers, lysimetersand surface water

	piezometers	lysimeters	Surface water		piezometers	lysimeters	wody pow.
cond.	$\frac{409-1085}{664}$	$\frac{268-994}{574}$	$\frac{431-1307}{835}$	HCO^3	$\frac{145-285}{215}$	$\frac{138-233}{190}$	$\frac{120-305}{226}$
pH	$\frac{6.87-7.59}{7.27}$	$\frac{6.64-7.33}{6.93}$	$\frac{6.7-8.05}{7.15}$	Cl^-	$\frac{8.5-80}{46.8}$	$\frac{4.7-50.9}{19.7}$	$\frac{12.8-58}{29.2}$
Na^+	$\frac{9.1-44.5}{22.3}$	$\frac{6.43-46.0}{19.6}$	$\frac{17.8-42.9}{27.9}$	N/NO_3	$\frac{0-0.03}{0.01}$	$\frac{0.029-0.94}{0.344}$	$\frac{0-3.59}{1.33}$
K^+	$\frac{2.3-5}{4.04}$	$\frac{0.57-5.7}{2.55}$	$\frac{0.65-43}{11.1}$	SO_4^{2-}	$\frac{15.9-302}{134}$	$\frac{11.3-436}{125.5}$	$\frac{31-605}{210}$
Ca^{2+}	$\frac{58-155}{97.4}$	$\frac{40-160}{103}$	$\frac{63-227}{130}$	Fe^{2+}	$\frac{0.91-6.8}{2.45}$	$\frac{0.04-75.4}{28.19}$	$\frac{0.22-36}{11.03}$
Mg^{2+}	$\frac{9.8-22.8}{16.7}$	$\frac{1.5-19.1}{10.1}$	$\frac{4.6-14.8}{10.1}$	Mn^{2+}	$\frac{0.07-0.327}{0.167}$	$\frac{0.004-0.87}{0.389}$	$\frac{0.186-1.16}{0.576}$

Figures presented in the Tab 2 also shows wide variations of Fe^{2+} and Mn^2 amounts in soil and surface water. Although the concentrations are sometimes really high (max 75.4 mg/dm^3 in soil water and max 36 mg/dm^3 in surface water) it shouldn't be admitted as an anomalous phenomenon. Such high amounts of those ions are sometimes reported in shallow groundwater in the presence of organic matter in sediments [2].

Figures 5 and 6 present variations of SO_4^{2-}, Cl^-, N/NO_3 and N/NH_4 in the wells of the intake in different periods of time, as a background for the investigations conducted in April 2002. The chosen ions can be admitted as indicators of human influence on the chemistry of groundwater. The sequence of numbers on the axe of abscissae responds to their actual location in the field. The periods of time are: 1978 (more or less natural or slightly changed composition of groundwater), 1992 (the year of highest amount of Cl^- ion in piezometers as a result of overexploitation) and 2002 (current situation).

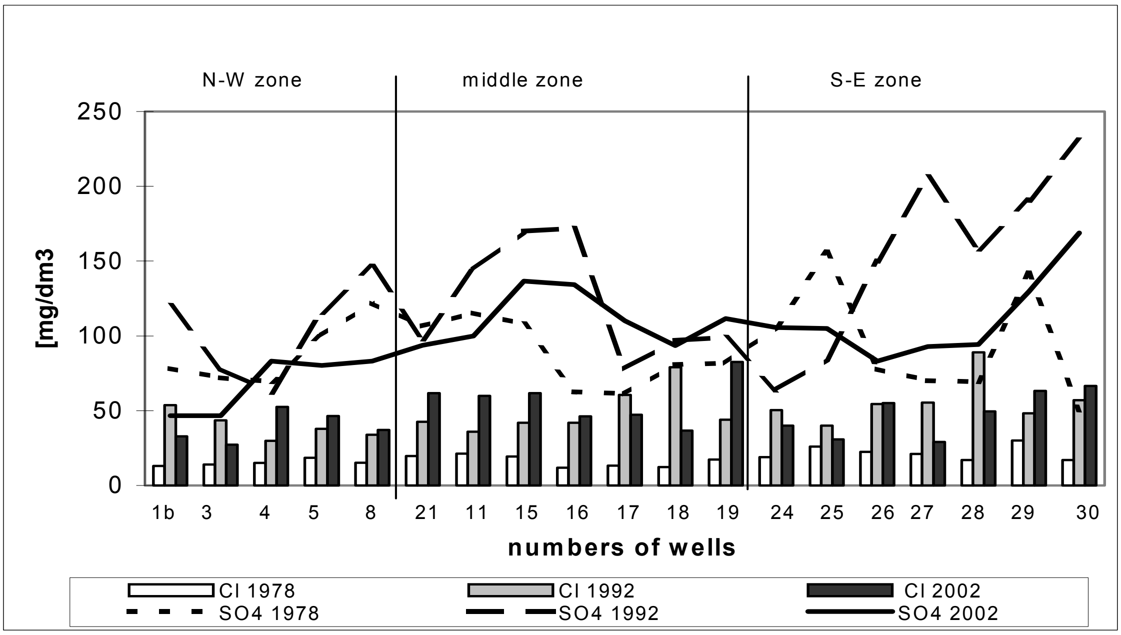

Fig.5 The changes of N/NO_3 i N/NH_4 concentration along the line of wells in different periods of time.

The graphs show changes in chemical composition of exploited water. The lowest concentration of majority of ions was in 1978. It is also visible that the amount of some compounds, especially ammonia nitrogen and sulphates varies in wide limits. Most significant changes concern the S-E part of the intake where such objects as sewage disposal plant, hospital, bus base, highway, allotment gardens are concentrated in close vicinity of the wells. Those variations goes together with an increased amount of Cl^- ion, which rather shouldn't be admitted in this part of area as the effect of salt water intrusion, but as the effect of human activity.

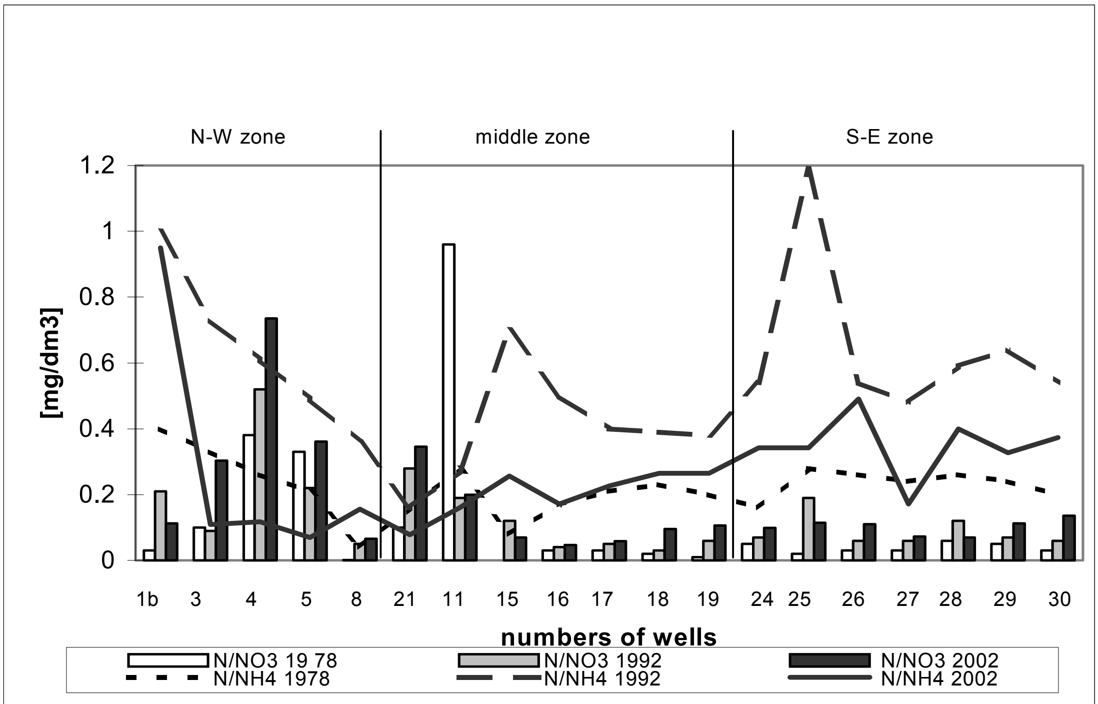

Fig.6 The changes of Cl^- i SO_4^{2-} along the line of wells in differents periods of time

Conclusions

The results of geophysical investigations show that there is no salt water intrusion into the Quaternary aquifer any more. In all the profiles at the coastal area fresh waters were ascertained.

The chemical analysis indicate the human influence on the groundwater composition, especially in the S-E part of the intake where some potential sources of contamination are in close vicinity. Those changes don't disqualify the exploited water to the drinking purposes. They only show some processes that take place in the aquifer. Distinctive feature of all kinds of waters is wide variation of ammonia nitrogen, nitrate nitrogen and sulphates, which can be related to the way of the land use. The influence of the allotment gardens is clearly visible in the chemical composition of surface and soil water (increased amount of sulphates, potassium and nitrate nitrogen) but not in groundwater. It is quite obvious in the current hydrodynamic conditions, when the exploited aquifer shows higher hydraulic head than the shallow groundwater. The situation can be changed in case of increased exploitation.

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Parameters of Turbulence at a River Harbour Entrance

Cedomil J. Jugovic[1], Willibald Loiskandl[2]

BOKU – University of Natural Resources and Applied Life Sciences, **Vienna**, [1] Department of Water Management, Hydrology and Hydraulic Engineering, cedomil.jugovic@boku.ac.at, [2]Department of Hydraulics and Rural Water Management, willibald.loiskandl@boku.ac.at, Muthgasse 18, A-1190 Vienna, Austria

Abstract
River ports are usually exposed to continuous sedimentation of suspended solids in their basins. The maintenance by dredging is very costly. Hence an intensive need to find solutions which would minimise the siltation of harbour basins exists. The objective of this study was to determine and analyse governing turbulent transport parameters of the flow pattern at the harbour entrance and to find an optimal solution for a yacht harbour on the river Danube in Austria to minimise siltation. The case study also serves as basis for a discussion of measuring turbulent parameters in a classical hydraulic scale model.

1 Introduction

Most practical free surface flow problems are turbulent. Turbulence is also responsible for viscous fluid friction losses. Natural flow processes are three dimensional, it is consequentially reasonable but not always possible and in same cases not necessary to perform a multidimensional analysis [1]. because they are characterised by significantly different geometric scales for flow in transverse or in longitudinal direction. The mathematical description as well as measurements on physical models provide a certain reproduction level of nature in form of one-, two- or three-dimensional description and build up the basis for the development of simulation models.
The case study is a good example of turbulent driven flow of water and sediments transport. Yacht ports situated on the Danube river in Austria suffer from continuous sedimentation, hence periodical dredging is needed. The costs of dredging is an economic concern of municipal and harbour management authorities. The sedimentation of a harbour is a result of a net sediment transport through the harbour entrance into the harbour basin. The driving mechanism of sediment transport through the entrance is the water exchange between river and harbour, caused by the a large velocity gradient between the river flow along the entrance and the flow in the harbour. This velocity difference results in a mixing zone at the harbour entrance which drives a recirculating flow in the harbour. River sediments which are transported into the harbour tend to settle down in the harbour basin. In such a case the flow in the harbour is solely driven by the turbulent transfer of momentum though the entrance. Contemporary turbulence models do not reproduce accurately the flow phenomena, mainly because several turbulent length scales are involved. Hydraulic scale models are still very valuable for the appropriate harbour entrance design. The objective of this study was to derive modifications of the harbour entrance to reduce siltation rates substantially.

2 Turbulence modelling

Turbulence is an irregular motion which in general makes its appearance in fluids when they flow past solid surfaces or even when neighbouring streams of the same fluid flow past or over one another [2]. The hypothesis of Bousinesq concerning the relation between the Reynolds stresses and the local mean velocity gradients, assumes the turbulent transport to be of the gradient type. In order to investigate the „additional stresses" in the contact area between the river and the harbour, where the mean velocity profile is changing rapidly, measurements of mean velocities $\overline{v_1}, \overline{v_2}, \overline{v_3}$, turbulent or fluctuating velocities v_1', v_2', v_3' and turbulent shear stresses $-\overline{v_1' v_2'}$ have been provided. The instantaneous velocity v_i is divided into a mean velocity $\overline{v_i}$ and a turbulent velocity component v_i'.

$$v_i = \overline{v_i} + v_i' \qquad (1)$$

For the turbulent velocity field the normal conservation laws hold. The conservation of mass for an incompressible fluid is:

$$\frac{\partial \overline{v_i}}{\partial x_i} = 0 \ ; \qquad \frac{\partial v_i'}{\partial x_i} = 0 \qquad \text{respectively.} \qquad (2)$$

The conservation of momentum leads to the so-called Reynolds equation:

$$\frac{\partial \overline{v_i}}{\partial t} + \overline{v_j}\frac{\partial}{\partial x_j}\overline{v_i} = -\frac{1}{\rho}\frac{\partial p}{\partial x_i} + \nu \frac{\partial^2 \overline{v_i}}{\partial x_j \partial x_j} - \frac{\partial}{\partial x_j}\overline{v_j' v_j'} \qquad (3)$$

The first term on the left hand side is the change of the mean velocity with time, the second term is the convection term. The first term on the right hand side gives the influence of a force caused by the pressure p, the second term is the viscosity term with ν the kinematic viscosity and the last term contains the turbulent correlations, commonly described as turbulence stresses. By multiplying them with the density (ρ), Reynolds stress tensor of turbulence is obtained[3]:

$$\begin{pmatrix} \tau'_x & \tau'_{xy} & \tau'_{xz} \\ \tau'_{yx} & \tau'_y & \tau'_{yz} \\ \tau'_{zx} & \tau'_{zy} & \tau'_z \end{pmatrix} = -\rho \begin{pmatrix} \overline{v'^2_x} & \overline{v'_x v'_y} & \overline{v'_x v'_z} \\ \overline{v'_y v'_x} & \overline{v'^2_y} & \overline{v'_y v'_z} \\ \overline{v'_z v'_x} & \overline{v'_z v'_y} & \overline{v'^2_z} \end{pmatrix} \qquad (4)$$

The „additional stresses" due to turbulent motion are acting in addition to the stresses

$$\tau_{ij} = \eta \frac{\partial v_i}{\partial x_j} \ ; \ \eta \text{ - dynamic viscosity}$$

which are caused by the water density and stream deformation. The turbulence stress term $\overline{v_i' v_j'}$ is an extra unknown in equation (3). It is impossible to solve this set of equations without further information about this term. Consequently the value of this term must be known or they have to be expressed by known quantities.. The increased computer capacity has added greatly to the possibility of calculating $\overline{v_i' v_j'}$. However for daily life problems the effort is still to large.

Although some of these models give remarkably good results in calculating turbulent flow fields, there are several difficulties. In the models a certain number of constants have to be introduced which are often assumed to be universal. The universal character however, has not been proven. It might well be that a constant determined in the case of the decay of isotropic turbulence may have a different value for turbulence with different structure, Another difficulty is the fact that although the equations for the unknown terms are formally correct, the physical idea behind the equations is by no means clear. For example, it is very difficult to get a physical idea of terms like dissipation in the transport equation used in some models. However, all the models have one basic restriction: the turbulent

flow field should be in a nearly equilibrium and in a nearly isotropic state. Consequently these models will be insufficient to describe strongly disturbed turbulent flow fields. Detailed description of turbulence is only possible through measuring of turbulent correlations.
In the last years a lot of activity was put in the implementation of new measuring techniques and instrumentation in the area of hydraulic scale modelling [4]. Today, by the standard model investigation it has become also possible to survey turbulent flow performances, not a characteristic so far of the conventional scale model tests.

3 Hydraulic scale model

The model was constructed for the yacht harbour Spitz on the Danube river (flow-km 2018,2; left bank). The harbour is situated on the outer bank of a long light right flow curve. The model scale was 1:40 / 25. For the definition of model margins, beside hydraulic and hydrological data, results of the flow field measurement in situ, at a Danube discharge of 2800 m³/s, were used. The flow width of the Danube river influencing the flow situation at the harbour entrance was about 1/5 of the total width (~ 300 m) (Fig. 1).

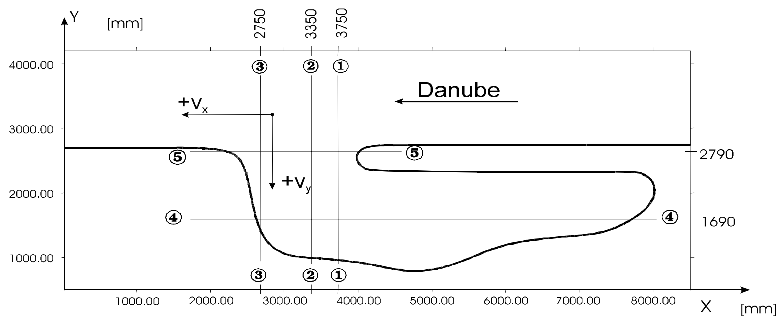

Fig. 1: Plan view of the harbour Spitz-scale model (present situation)

Following parameters have been varied:
- Construction elements – short groins in different positions
- R. Danube discharge: 2000 m³/s (130 - days exceed duration)
  2800 m³/s (50 - days exceed duration)
  4000 m³/s (6 - days exceed duration)
- Free discharge / back water conditions
- Dike extension, shifting outside / inside
- Downstream dike position 90° / 30° to flow direction

Different forms of the harbour entrance were tested:
- V0 Present situation of the harbour Spitz (harbour type 1)
- V1a - V1n Installation of short groins and bottom sills (Fig. 2)
- V2a - V2f 4,5 m Dike shifting in river direction; different positions of short groins (Fig. 2)
- V3a - V3g 7,5 m Dike shifting in river direction; different positions of short groins
- V5a - V5d Harbour type 2, downstream harbour dike, 30° to river flow direction (Fig. 2)
- V6a - V7d Harbour dike extended for 20 m; 40 m
- V8a - V8d Harbour dike extended for 20 m and shifted 4,5 m in river direction
- V9a - V9d Harbour dike extended for 20 m and shifted 4,5 m in harbour direction (Fig. 2)

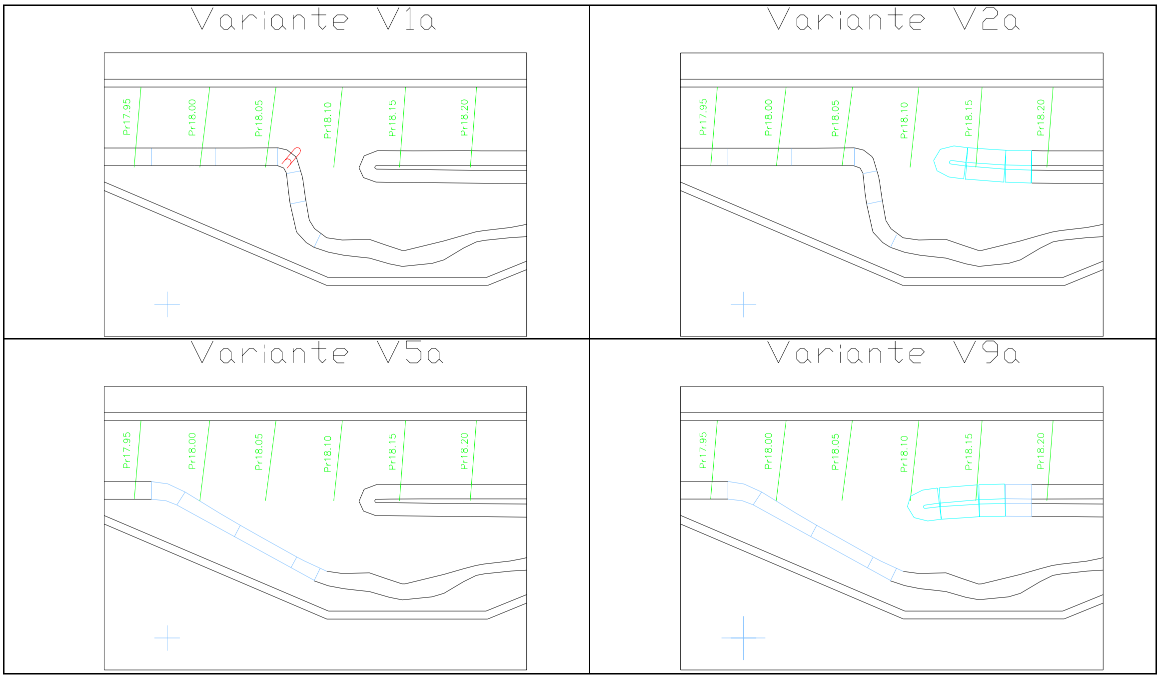

Fig. 2: Tested construction changes of harbour geometry

4 Experimental procedure and results

The difficulties of scaling suspended sediments for the needs of hydraulic model are already known. Therefore, some alternative experimental procedures have been undertaken [5]. Through detailed flow–velocity measurements with help of 3-D acoustic-Doppler current- meter, some turbulent flow parameters, especially in the mixing zone between the river and the harbour, have been determined and analysed [6].

4.1 Turbulence intensity (T_u)

The time average values for time T are:

$$\overline{v_i} = \tfrac{1}{T}\int_{t_0}^{t_0+T} v_i \, dt \qquad (5)$$

The time should be long enough to ensure that mean values of velocities (Fig.3) are time independent. The average value of the fluctuating velocity is given as RMS (root mean square) turbulence value

$$v_i' = \left[\tfrac{1}{T}\int_{t_0}^{t_0+T} \left(v_i - \overline{v_i}\right)^2 dt\right]^{0.5} \qquad (6)$$

and is equal to the time-series standard deviation of instantaneous velocity, e.g. for x-velocity component

$$\mathrm{RMS}\left[v_x'\right] = \sqrt{\left(v_x'\right)^2} = \sqrt{\frac{\sum v_x^{\,2} - \left(\sum v_x\right)^2 / n}{n-1}} \qquad (7)$$

where n represents the number of measuring values in a sample [7].

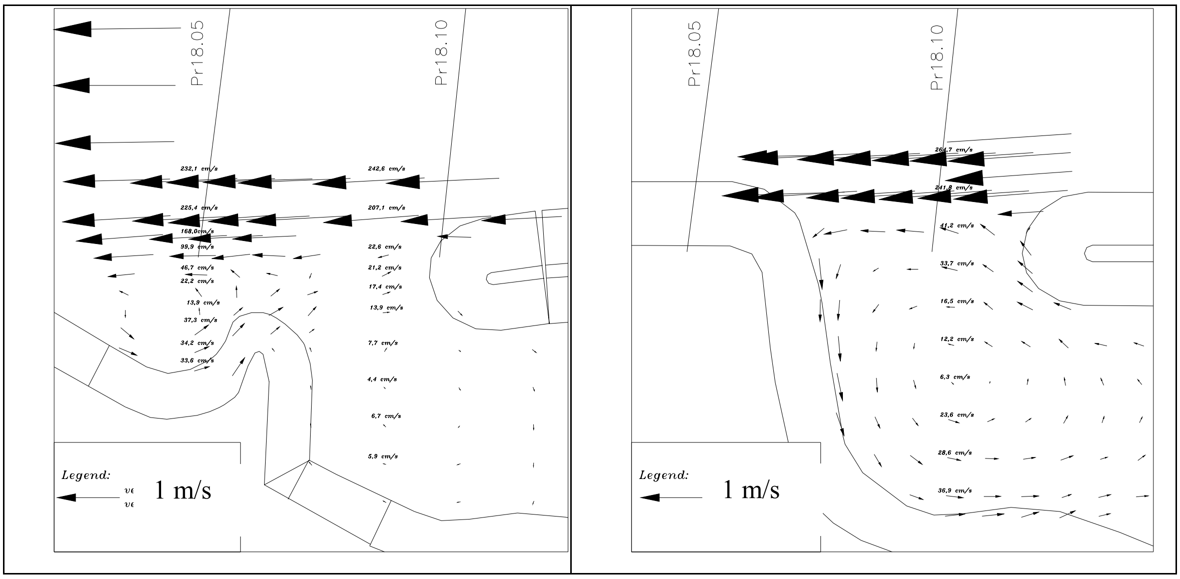

Fig. 3: Mean velocity flow field in the river/harbour area

The turbulent velocities always appear as a superposition of their components. The turbulence intensity distribution (Fig. 4) is determined through vector summation of the average values divided with the mean channel velocity ($\overline{v_k}$).

$$T_u = \frac{\left(\overline{v_x'^2} + \overline{v_y'^2} + \overline{v_z'^2}\right)^{0.5}}{\overline{v_k}} \qquad (8)$$

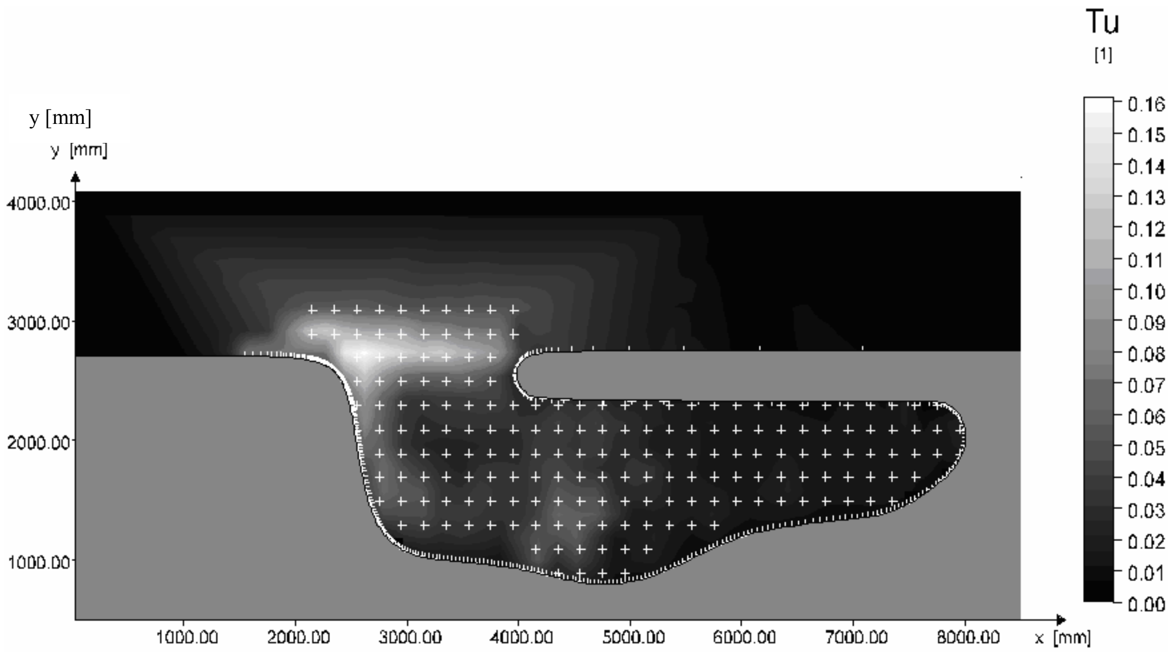

Fig. 4: Turbulence intensity distribution

4.2 Turbulent exchange coefficient (A_y)

The velocity flow field in the mixing zone (harbour entrance) is characterised by a significant longitudinal velocity gradient in cross direction y, d $\overline{v}_x$ / dy (Fig. 5) and by the maximal turbulent velocity correlation $\overline{v_x' v_y'}$ (Fig. 6) [8].

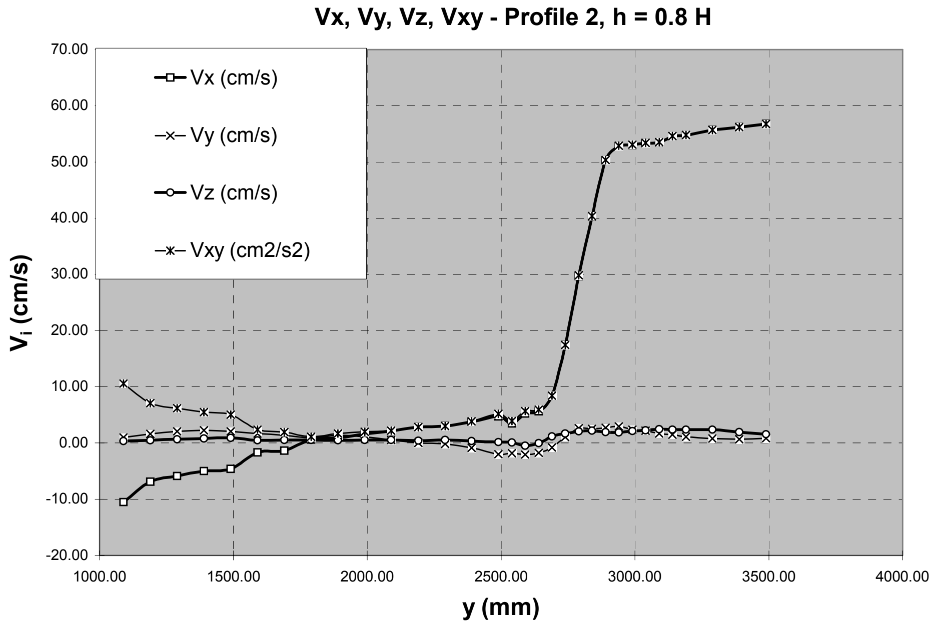

Fig. 5: Velocity gradient d $\overline{v}_x$ / dy - Profile 2

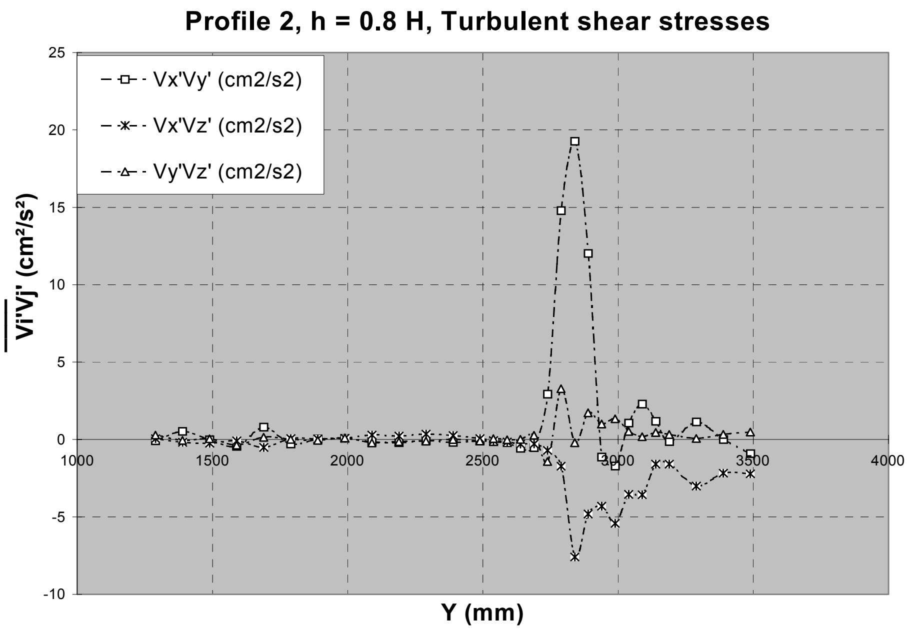

Fig. 6: Turbulent shear stresses $\overline{v_x' v_y'}$ - Profile 2

The covariance is used as a measure of the correlation between the two variables in the analysis of the turbulent shear stresses.

$$\mathrm{COV-XY} = \overline{v_x' v_y'} = \frac{\sum v_x v_y}{n-1} - \frac{\sum v_x - \sum v_y}{n(n-1)} \tag{9}$$

The shear stress components $\tau_{xy}' = \tau_{yx}' = \overline{v_x' v_y'}$ represent the transport of the x-momentum through the area perpendicular to the Y-axis, e.g. harbour entrance (Profile 5 in Fig. 8). On the basis of the turbulent flow measurement, it is possible to calculate the turbulent exchange parameter (A_y)

$$A_y = \frac{\overline{v_x' v_y'}}{dv_x / d_y} \tag{10}$$

at the harbour entrance.

The local variable turbulent exchange parameter A_y may be calculated as a ratio between the maximal shear stresses and maximal velocity gradient. Maximal values for both parameters occur in the profile of the harbour entrance. The intensive mixing flow in the dividing plane river-harbour show different local shear stress intensities along the harbour entrance (Fig. 7).

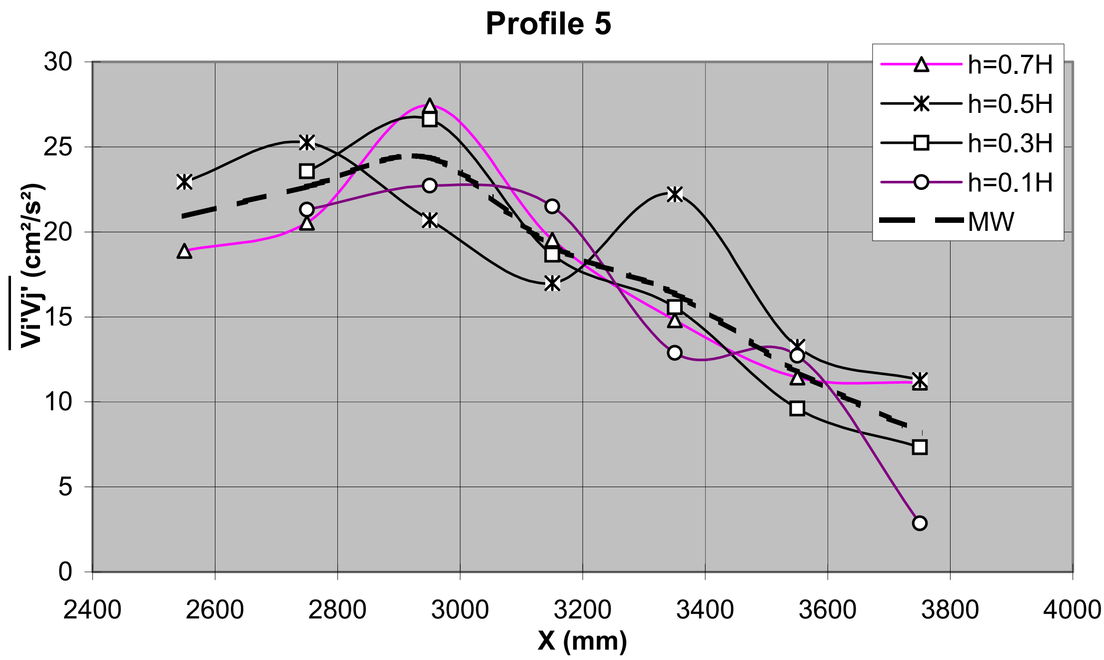

Fig. 7: Turbulent velocity correlation $\overline{v_x' v_y'}$

On the contrary the velocity gradients in three cross sections (P1, P2, P3) indicate negligible differences among each other. Hence, it follows that the variation of the turbulent exchange parameter A_y is manly predominated by turbulent velocity correlations $\overline{v_x' v_y'}$ (Fig. 8).

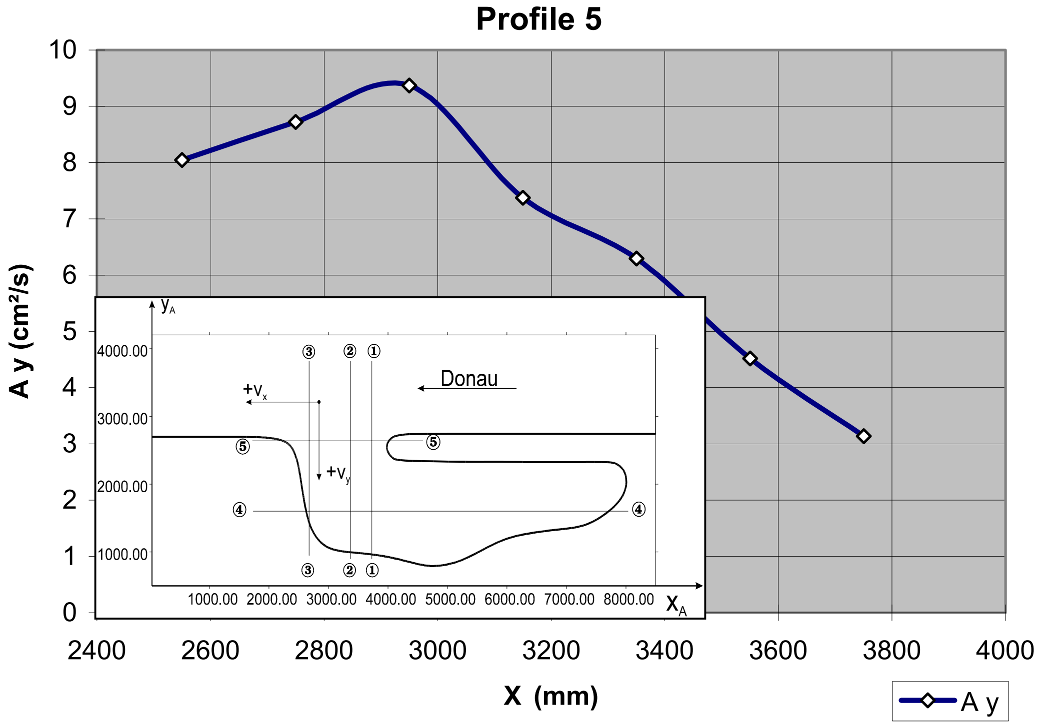

Fig 8.: Distribution of exchange coefficient A_y over the harbour entrance

5. Conclusion

In this study the influence of turbulent flow parameters at the harbour entrance on water exchange between the river and the harbour has been examined. The complex flow patterns involved are not easy accessible by computation , therefore a hydraulic model tests was performed. The exchange of water occurs mainly due to the turbulent diffusion. In the mixing zone between the main flow stream and the harbour the turbulence is caused through shear forces due to velocity gradients. Based on accurate long-time velocity measurements, and statistic analysis of velocity-component signals, the following conclusions can be made:

- One large circular gyre is formed in the entrance area of the harbour. The velocities are decreasing in the direction to the gyre centre. The flow pattern does not vary significantly over water depth.
- The geometry of the harbour entrance has considerable influence on the turbulent flow characteristics.
- The maximum of the mean-velocity gradient $d\overline{v_x}/d_y$, turbulence intensity (T_u), Reynolds stresses $\overline{v_x' v_y'}$ and consequently turbulent exchange parameter (A_y) occur at the interface river/harbour, located downstream from the centre of the entrance.

The role of laboratory tests is shifting from problem solving to process studies, among which turbulence is of primary importance. This requires quite different measuring techniques, both with regard to the quantities to be measured and the resolution in time and space. Covering the flow field by computerised long-time velocity measurements with high frequency is one of the main tasks of hydraulic scale modelling today.

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Floods on small streams – their reasons and possibilities of protection against them

Jozef Kamenský, Barbora Fialíková

Slovak University of Technology, Department of Hydraulic Engineering, Radlinskeho 11, 813 68 Bratislava, Slovakia, kamensky@svf.stuba.sk , fialik@svf,stuba.sk

Abstract
There are historical floods on the small and medium streams of Slovakia relatively often in the last time. These floods, which have character of local disaster and make large damages especially in the adjoining villages and towns. They occurred predominantly during summer months and they are caused by extreme precipitation. The contribution shows course of floods, which were in the June 1999 on the river Myjava in the catchment of the river Morava and on the streams of Krupinica and Litava in the catchment of the river Ipeľ. It analyses the reasons of their origin. From the analysis of the precipitation data, runoff from the catchment, capacities of stream channels and retention ability of hydraulic structures built on them come out the technical designs to flood protection in relatively high dense settled valleys of these streams.

1 Introduction

There were a lot of relatively big floods on the small streams in the Slovakia during last four years. Their catchment area is between 50 and 200 km^2 and they are also characterised by high dense settled valleys of these streams. These floods are typical by enormous heavy increase and decrease of discharges in time. They have a short duration and in the reason of urbanisation of their valleys they caused large damages on the belongings of inhabitants, state property and on the roads.

Department of Hydraulic Engineering, Faculty of Civil Engineering, SUT, made on the base of the request of injured organisations and villages, hydraulic analyses of two different flood situations, which caused overflowing of region towns Myjava and Sahy and adjoining villages. Both of these floods occurred in June 1999. This article gives an information about results of these analyses and about measures, which are prepared on the base of these analyses.

Region town Myjava is situated on the river Myjava, which springs in Biele Karpaty Mountain. River Myjava has length of more than 80 km and discharges itself into the river Morava by the village Kuty. The whole catchment area of the Myjava has an area of 800 km^2. Area of interest, which was injured by this analysis, is the upper part of the catchment area approximately till the tributary Brezovsky potok. The size of catchment area till this tributary is 150 km^2. It is characterised by rather big lateral slope of thalweg from 1,1 till 1,4 % and by the heterogeneity of arrangement of forests with the area about 30 %. Greatest part of catchment area is composed of meadows and fruit. The catchment area is characterised by relatively high dense settled of bank zone with town Myjava and villages Horna Myjava, Tura Luka, Podbranc, Prietrz, Osuske a Jablonica.

On the river Myjava are located and operated three smaller hydraulic structures. It is the water reservoir Armaturka with the retention volume of 11 000 m^3, hydraulic structure Brestovec with the retention volume of 62 000 m^3 and hydraulic structure Star Myjava with the retention volume of 69 000 m^3. These uncontrolled volumes could decrease together peak discharge maximally less than 6 m^3s^{-1}.
The second important locality, where the floods caused damages and danger for inhabitants, is the region town Sahy by adjoining villages. The town is situated on the confluence of the rivers Ipel and Krupinica. River Krupinica is one of its most important tributaries on the Slovak area.
This river springs in the Javorie Mountains and takes away the water from the catchment area with the size 551 km^2. Its most important tributary is the river Litava, which represents 2/5 from the whole catchment area of the river Krupinica. Both streams are trained only partially in the villages and there are no hydraulic structures on them. Catchment area is in the upper part of these rivers systematically forested and in the lower parts only sparsely, forest create 35 % of the area. Flood in June 1999 occurred also on another right bank tributary of the river Ipel, the river Olvar with the catchment area 58 km^2.

2 Description of flood situations

Flood on the upper part of the river Myjava rose on 22.6.1999. It was caused by extremely profuse precipitation. Preview of daily precipitation measured at the hydraulic structure Brestovec during June is on the Fig.1. From the graphic data results that June had abnormally high precipitation. Before the flood, there was a period with rather high precipitation. In the decade before the day when the flood began, there were six days with precipitation over 10 mm. In this period fell down rainfall of 84.4 mm.

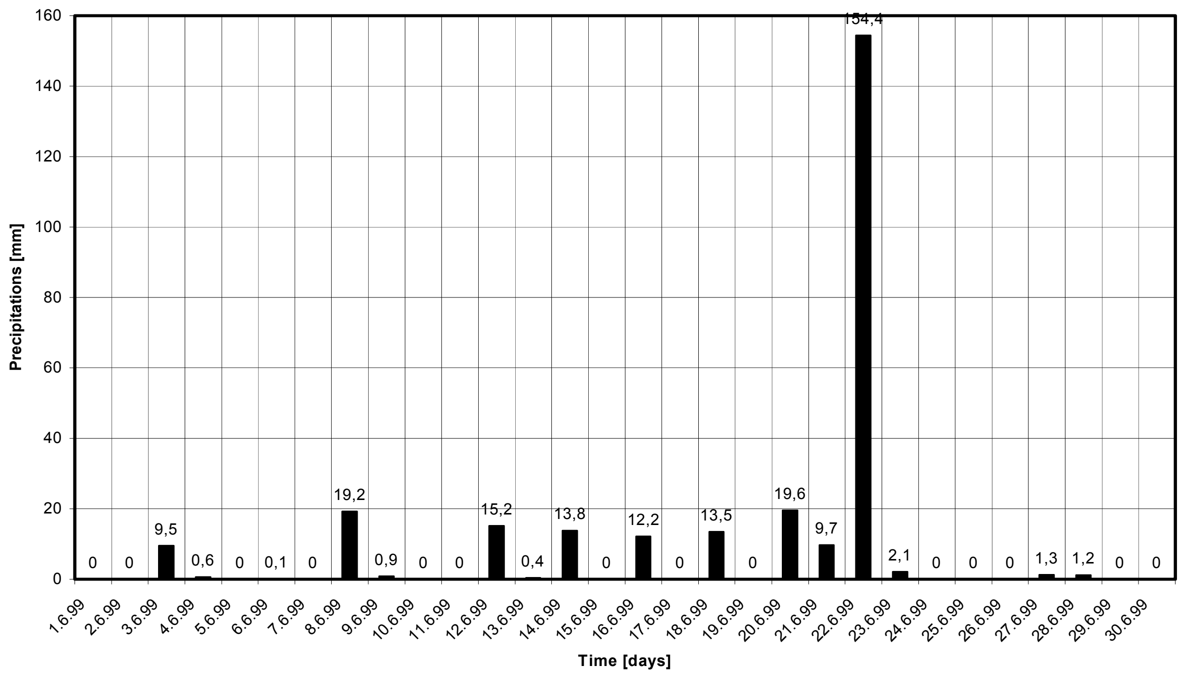

Fig. 1: Daily precipitation at reservoir Brestovec in June 1999

Into this statement on 22.6. came really enormous precipitation introduced by the daily precipitation total 154.4 mm, which suddenly increased tributaries from the catchment into streams and caused flood situation. Time distribution of precipitation during the critical day, which is on the Fig.2, is also not homogeneous. The maximums of the hour precipitation total were in the time between 7 and 9 am. In this time fell down rainfall of 43.36 mm. These extremely high hour precipitation total caused, that equipment, which normally work without problems, were overload and they were no match for taking away the water from the town Myjava.

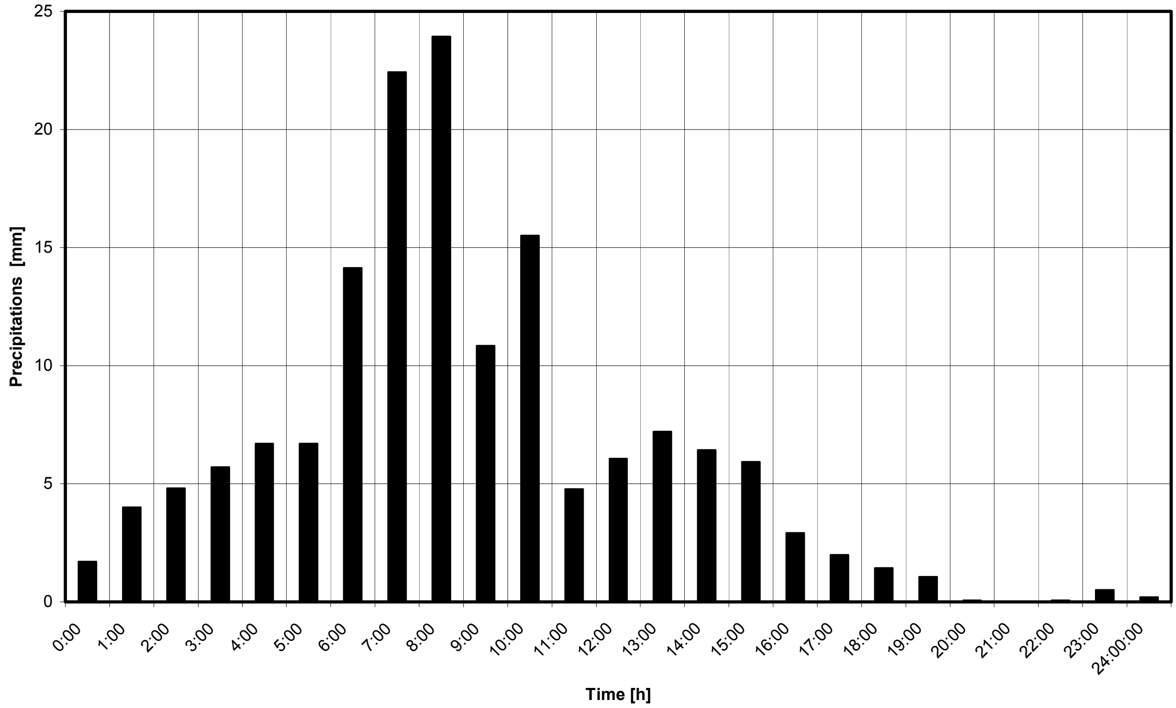

Fig. 2: Hourly precipitation at reservoir Brestovec on June, 22, 1999

Because precipitation total was measured also on the upper hydraulic structure Stara Myjava, it is possible to say that in the upper part of the catchment area of the river Myjava was the precipitation on 22.6. not as high in the part of the town Myjava and in the upper part above the town. We can assume that the main part of the discharge came into the river Myjava from its right bank tributaries and also from the beaded surfaces of the town. We can say that the extremely high, time and space unevenly distributed precipitation caused spilling out of the discharge from the river bed, which had higher sizes than officially stated $Q_{1\%}$.

Also in the second case of flood, which overflowed part of the town Sahy, was the primary and base reason extremely high precipitation before flood and during the flood. Data about daily precipitation total on Vinica and Dudince stations are graphic described on the Fig. 3. Vinica station is situated 12-km north-east of the town Sahy near the catchment area of the river Olvar and the data was used for the statement of precipitation and run-off from this catchment area. Dudince station is situated 10-km north of the town Sahy was relevant station for the analysis of the run-off from catchment areas of rivers Krupinica and Litava.

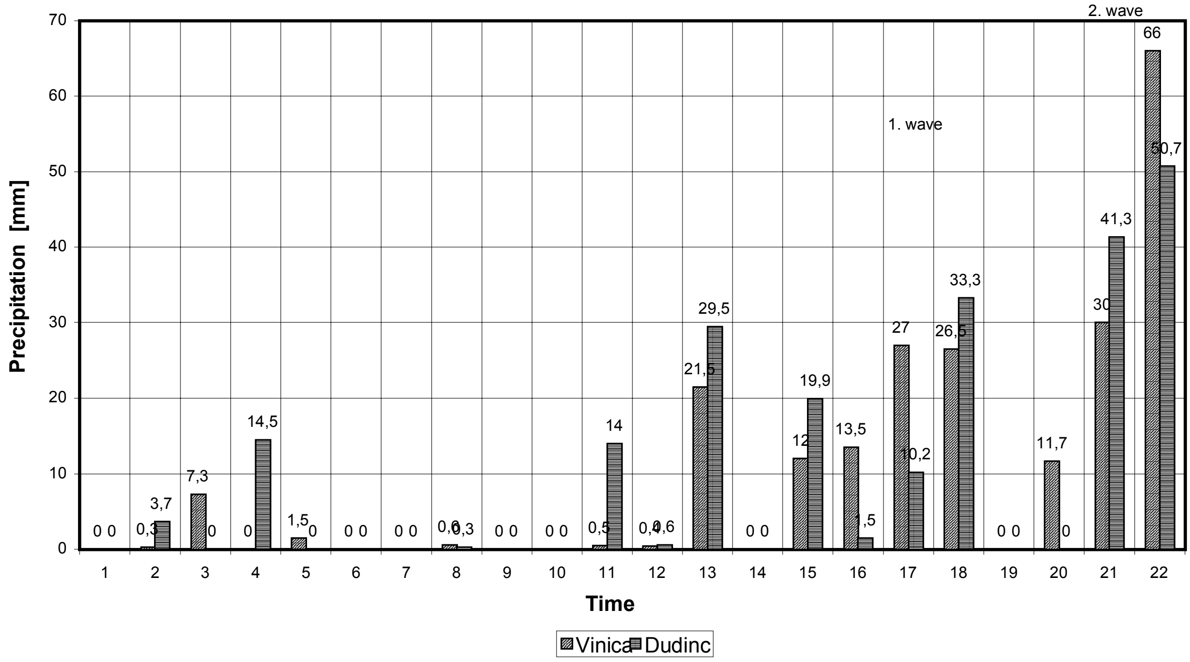

Fig. 3: Daily precipitation on Vinica and Dudince stations in June

Daily precipitation total observed in stations Slovenske Darmoty and Vyskovce, which are documented on the Fig.4, are just trying to describe the whole situation in wider area and to show two waves of the precipitation. In first wave, in the period from 17.6. to 20.6., there were precipitation from storm. Second more important wave of precipitation, in the period from 21.6. to 25.6., was caused by up-and-down cold front, which attacked with its precipitation the region of Stiavnicke vrchy mountain and Krupinska vrchovina mountain.

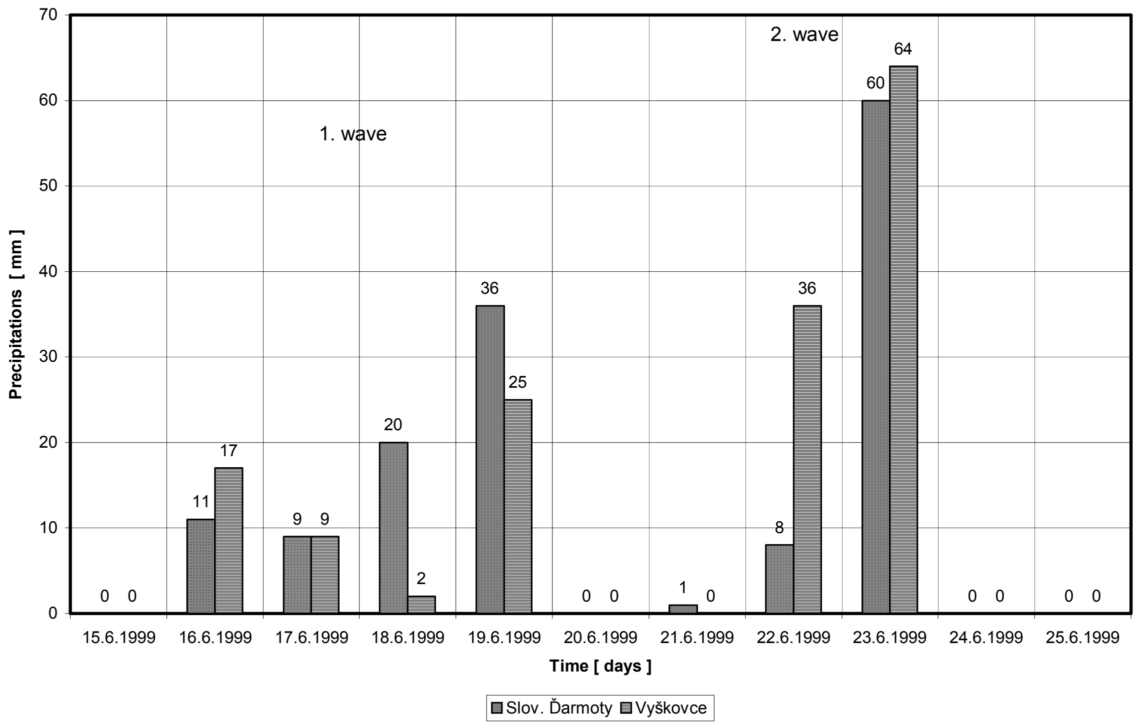

Fig. 4: Daily precipitation on Slovenske Darmoty and Vyskovce stations in June 1999

The second wave of precipitation caused the increase of discharges in streams that way that the river Olvar spilt out from its riverbed and the great part of the village Tesmak was overflowed. In rivers Krupinica and Litava rose time-synchronised floods with the culmination of the same standing, which caused spilling out the riverbed of Krupinice under the Litava tributary and overflowing of the village Taban situated in the place of confluence of rivers Ipel and Krupinica.

3 Analysis of reasons of floods

Extreme precipitation in catchment areas of both streams, where the floods rose, activated abrupt increase of discharges, what we can see on flood waves on figures5, 6 and 7.

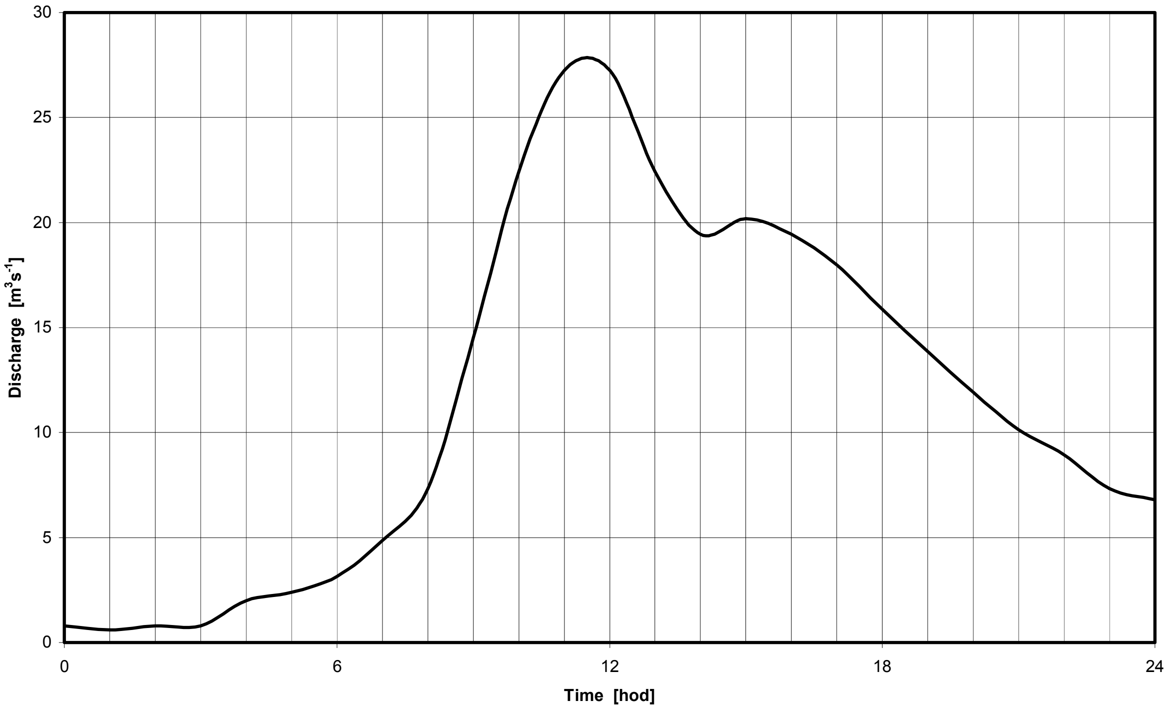

Fig. 5: Discharges on security spillway at reservoir Brestovec on June, 22, 1999

Time preview of discharges in the river Myjava on 22.6 in the form of flood wave, obtained from measurements of water levels and spillway heights on hydraulic structure Brestovec is on the fig. 5. From the graph of the flood wave results that the peak discharge $Q_{max} = 27.6\ m^3s^{-1}$ occurred round noon and it rose from normal discharge in 7 hours.

Because the controller of the river fixed and later measured peak discharges and cross sections on the river on rather long stretch, it was possible on the base of the results obtained from 1-D mathematical model of the river Myjava from hydraulic structure Stara Myjava until the end of the town approximately state peak discharges, which increased its size in the consequence of right bank tributaries and from the town. Discharges downstream the town in the village Tura Luka already reached values about 70 m^3s^{-1}, it is the double of the real discharge capacity of the river Myjava on this stretch.

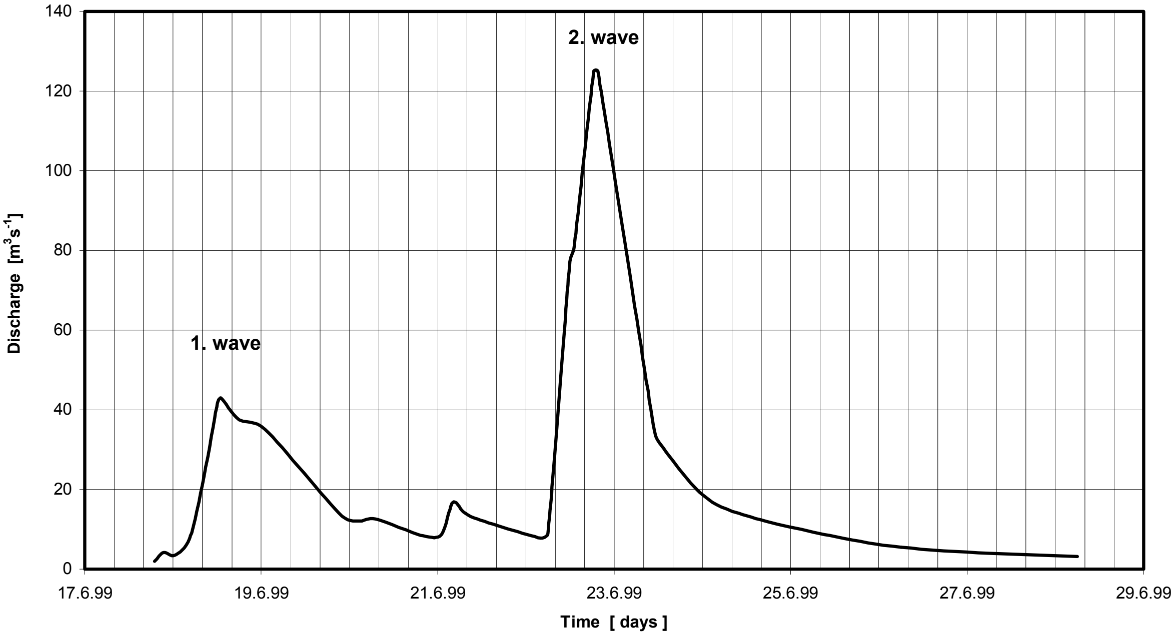

Fig. 6: Discharges of Krupinica river in Plastovce

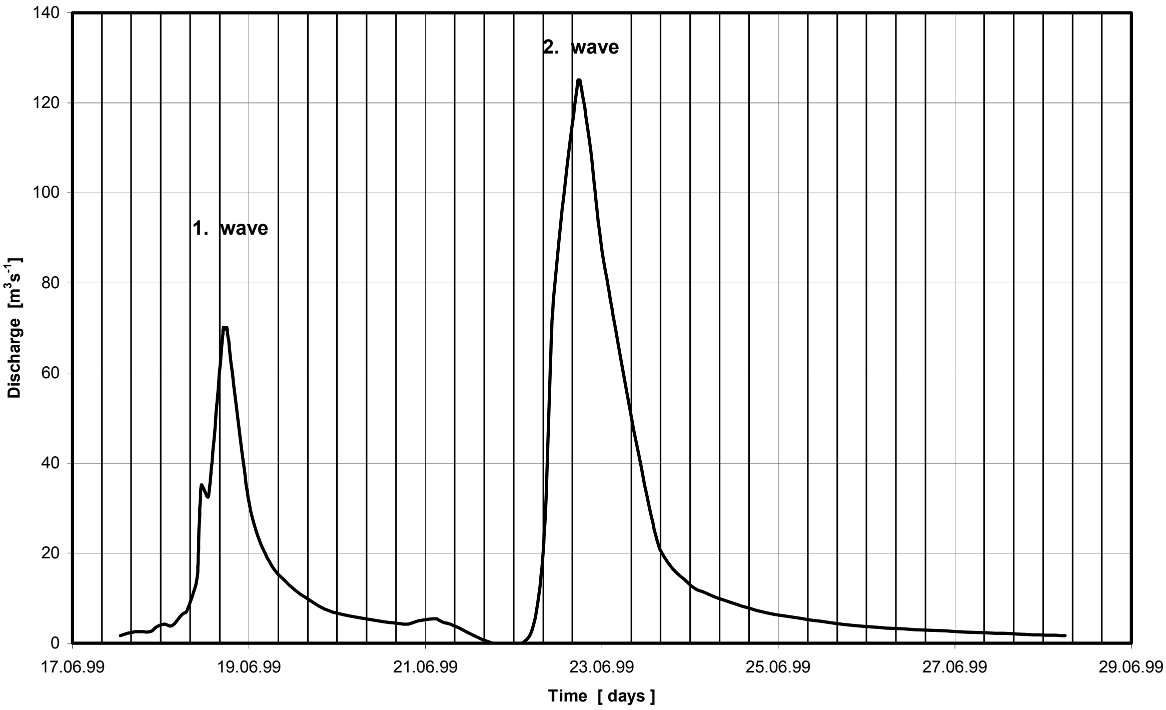

Fig. 7: Discharges on Litava river in Plastovce

Flood wave running on rivers Krupinica and Litava were constructed from water level data and from stage discharge level curves in measurement cross sections situated upstream the confluence of both rivers. We can see from graphs on figures 6 and 7 two independent flood waves, caused by two precipitation waves and we can also see two extremely fast increases of discharges on both rivers in both waves. Flood situation of unexpected extensions was caused by current occurrence of culmination on both rivers, what caused that the value of discharge reached $Q_{max} = 240\ m^3s^{-1} \cong Q_{0.2\%}$ under the confluence of rivers.

By both floods we can characterise their corporate marks and also corporate occasions. To main reasons belong:

- Auxiliary high precipitation total in whole month especially in the period from 2. till 22. June, which were not reached in the whole century and which represent for the region Sahy 300% of normal monthly precipitation total.
- Substantial precipitation with high intensity occurred in one day, daily precipitation total was about 100 mm in Sahy region and 154 mm in Myjava region.
- As a result of former precipitation from first waves it came to the saturation of soil horizont, which caused cardinal increase of runoff coefficients, from effected comparisons of the precipitation value and flood waves volume result that average runoff coefficient values, which are in both regions about 0,3, increased about 40% to values 0,4 and 0,45.
- In both cases discharge capacities appeared often inside villages and towns negative influenced by intervention into the discharge cross section by building of bridges, crossing the wiring, etc.
- On untrained rivers (Krupinica, Litava, Olvar) rises the danger of washing away the dirtiness, limbs an d sometimes also their arrestment in cross sections and the formation of immeability barriers in the river.

To these general reasons we could allocate for every of these cases real specific reasons, to which belong falling off the flow capacity by inadequate operation, limited options of forecasting, reporting and warning hydrological service, limitations in existing legislation in water management and in environment etc.

We can also see noticeable increase of flood damages, which is by the way also caused by building of family houses, even whole neighbour units (neighbour unit Taban in Sahy) realised in regions first endangered by floods.

4 Devices of technical actions and conclusions

From detailed climatic, hydrologic and hydrodynamic analysis of flood running and reasons in regions of towns Myjava and Sahy result submissions for the realisation of technical and organising actions on rivers and catchment areas, which could increase flood-protection of the area especially inside villages and towns in lowlands of rivers Myjava, Krupinica, Olvar.

Organisation arrangement cover especially accomplished knowledge of present discharge capacities with detailed knowledge of the situation inside villages and towns. It depends on an execution of hydrometric and geodetic measurements coupled together with fixing of peak levels and measuring of peak discharges. Such results could enable formation and ideal verification of 1-D mathematical models of non-uniform flow, by the application of which we could forecast running of water level with enough precision for various scripts of flood situations. Application results enable detailed processing of submissions of technical actions in critical stretches of rivers.

Components of organisation actions should be also enhancement of forecasting, reporting and warning hydrological service with operative information flux to injured organisations. In this context it seems to be necessary to compress the mesh of hydrological stations on rivers and to supply mode stations with information about development of water levels. Experience showed us that it would be necessary to realise also reform of legislation in such way, so that the controller of the river could be responsible for its state without useless draughts and he could realise the necessary volume of technical actions on the river during the whole year.

Submissions of technical actions have to go out from superior state knowledge of present discharge capacities of the river bed and it will be designed so that they could first protect goods and safety of inhabitants and organisations in of-shore river areas.

On rivers Krupinica, Litava a Olvar they are preparing the systematic river training with adequate solution of flow capacity, in which the river is crossed by communications and wiring. We can also calculate with using the terrain for preparing the space for flood storage reservoirs, which could decrease peak discharges to the considerable value in the case of flood.

There are designed local regulations of the riverbed in the villages on the upper part of the river Myjava and there is also project of seven flood storage reservoirs with volume between 130000 and 1442000 m^3. These flood storage reservoirs are situated directly on the stream, they consist of a dike and a functional object. Its outlet is designed in the way that its capacity is not higher than the discharge capacity in the part below this outlet and emergency spillway is designed for execution of peak discharges.

Part of technical actions, which were already realised, is also detailed cleaning of riverbeds and removing plants, which interfere the cross section. Systematic maintenance of riverbeds and regular control of their state, especially in villages and towns, belongs to necessary activities of the river controller. Some reduction of flood damages could be reached by regulation of construction in those places of the valley, where rivers can spill out of the riverbed. By designing and realisation of arrangements it is necessary to remember, that some reasons of floods – extreme precipitation and their non-uniform distribution, rise of fortuity and unfavourable cumulating the flood waves – are of objective character, they will for sure repeat and it is not real to insure absolute protection against floods.

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Multi-criteria Optimization Methods in Water Management

Barbara Karleusa[a], Boris Berakovic[b], Nevenka Ozanic[c]

[a] University of Rijeka, Faculty of Civil Engineering, V. C. Emina 5, 51000 Rijeka, Croatia, barbara.karleusa@gradri.hr
[b] University of Zagreb, Faculty of Civil Engineering, Kaciceva 26, 10000 Zagreb, Croatia, boris.berakovic@zg.htnet.hr
[c] University of Rijeka, Faculty of Civil Engineering, V. C. Emina 5, 51000 Rijeka, Croatia, nozanic@gradri.hr

Abstract
Today, beside the economical aspect that was the most important criterion until 30-years ago, the protection of environment (and water as a part of it) and the social impact of water management projects have a very important role in decision-making process in water management.
The complexity of problems in water management is the result of complex and numerous needs, goals and different measure criteria for the evaluation of those solutions. This complexity leads to developing multi-criteria optimization methods that allow evaluating solutions and choosing the optimal among them on the base of different measure criteria.
This paper deals with the use of multi-criteria optimization methods PROMETHEE, ELECTRE and AHP, on which computer softwares PROMCALC & GAIA V.3.2., ELECTRE TRI 2.0 a and EXPERT CHOICE Pro 9.5. are based, for choosing the best reservoir configuration for irrigation of agricultural areas in western Istra in Croatia. The choice is made among twelve generated solutions (alternatives), by evaluating them on the base of ten different measure criteria according to a previously defined testing plan.
In the paper is concluded that chosen multi-criteria optimization methods are, using critically evaluated input data, appropriate and useful for decision making (choice of optimal solution) in water management problems and that further research and study on improving the process of solving problems in water management should be done in the direction of better defining criteria and measures, complex generated solutions and improving and developing new multi-criteria optimization methods.

1 Introduction

During 1970-ties as a fundamental concept in development the sustainable development was introduced. The concept of sustainable development includes the use of resources (in our case water as a renewable resource) permanently fulfilling human basic and existential needs. Such approach leads to new and better quality solutions in water management.
The criteria on which present and new solutions are created and assessed derive from the basic postulates of sustainable development.
The criteria can be classified in few basic groups:
- fulfilling water demands, water protection and protection of water,
- environmental and social acceptability
- and economic valuability.

A larger number of complex criteria led to the use of multi-criteria optimization methods for choosing best solutions in water management strategies.
In this paper for choosing the best solution for irrigating western Istra agricultural areas (Croatia, Fig. 1) multi-criteria optimization methods: PROMETHEE - Preference Ranking Organization METHod for Enrichment Evaluations, ELECTRE - ELimination and (Et) Choice Translating Reality and AHP - Analytic Hierarchy Process, were used. A detailed review of solving the mentioned problem is shown in the work[1].

Fig. 1: Study area

2 Methods

PROMETHEE are multi-criteria outranking methods used for partial (PROMEHEE I) or total preorder of alternatives (PROMETHEE II)[2][3][4]. In PROMETHEE methods the notion of criterion is extended by introducing the preference function giving the preference of the decision maker for an alternative *a* regard to *b* for each criterion. The preference value is between 0 and 1. The smaller the function, the greater the indifference of decision maker, the closer to 1 the greater his preference. In case of strict preference the value of the preference function is 1.
Six different types of criterion functions, for which the decision maker has to define maximum two parameters (indifference and preference thresholds), cover most of cases that can happen in practice: usual criterion, quasi criterion, criterion with linear preference, level criterion, criterion with linear preference and indifference area and Gaussian criterion. A valued outranking graph is considered by using a preference index. In this paper the PROMETHEE II method and criterion functions: usual criterion and level criteria, were used. On the base of PROMETHEE method PROMCALC & GAIA V.3.2. software for MS-Dos system (that was used for this case study) and newest version Decision Lab 2000 software for Windows systems were developed.
ELECTRE are multi-criteria optimization methods that enable the choice, ranking and sorting of alternative problem solutions (depending of the ELECTRE version) taking into account decision-makers criteria and preferences[5][6]. The ELECTRE method was developed for a partial order in the set of solutions based on the preferences of the decision maker. A graph with nodes that represent possible solutions and a kernel that defines preferred solutions can be constructed. ELECTRE method is useful in case when criterion functions are weakly defined. The first developed method was ELECTRE I and on the base of it ELECTRE II, III, IV and TRI followed. The ELECTRE TRI is a multi-criteria sorting method that assigns

alternatives to predefined categories. The assignment of an alternative *a* results from the comparison of *a* with the profiles that define the limits of the categories. The built outranking relation validates or invalidates the assertion of the alternative *a* to a predefined category. The ELECTRE TRI method gives the decision-maker a possibility to use pseudo-criteria with indifference and preference thresholds. In this study case ELECTRE TRI 2.0a software for Windows was used. The criteria used were true (usual) criteria and pseudo-criteria with the same thresholds used for defining level criteria for PROMETHEE method.
AHP is a priority method applicable to problems that can be represented by a hierarchical structure [7][8]. The top of the hierarchy is the goal, one level lower are criteria and even lower sub-criteria. The lowest level is represented by alternatives. AHP method is conceived on estimating relative priorities (weights) of criteria and alternatives on which pair-wise comparison criteria matrix and pair-wise comparison alternatives matrices (one matrix for each criterion) are generated. The matrices are normalized in order to calculate the priority weightings for criteria (the priority vector for criteria) and for alternatives on the base of each criterion (the priority vector of alternatives for each criterion). Combining the criterion priorities and the priorities of each alternative relative to each criterion develop an overall priority ranking of the alternative which is termed as the priority matrix. On the base of AHP method the EXPERT CHOICE Pro 9.5 software for Windows was developed.

3 Analyses

The introduced methods were employed for selecting the optimal reservoirs configuration that would solve the problem of irrigating agricultural areas in western Istra (peninsula in Croatia) shown on Fig. 1 and Fig. 2.
Supplying necessary water for irrigation is possible to achieve using water from: the catchment area of river Mirna (in which beside the already built reservoir Botonega, three more reservoirs Recina (R1), Bracana (R2) and Blaskici (R3) can be built), the catchment area of Pazinski potok/Pazin stream (on which reservoir Rakov potok/Rak stream (R4) is planed), the catchment areas of Zrenjske visoravni/Zrenj high-planes (reservoirs Momjan (R6) and Bazuje (R7)) and spring Blaz (reservoir Marcana (R8))[9].
Taking into account the technical-technologic feasibility, functionality, environmental and social limitations/restrictions and acceptable economic costs twelve alternatives that enclose different combinations of reservoirs (shown on Fig. 2): Recina (R1), Bracana (R2), Blaskici (R3), system Rakov potok-Beram (R4+R5), system Momjan-Bazuje (R6+R7) and Marcana (R8) were generated. Building of reservoirs was conceived in four or five phases depending of the alternative including that the first phase in all alternatives would be using water directly from the flow of the river Mirna and local springs.
The configurations of reservoirs that make alternatives (A) are:

- A1: Recina (R1), Bracana (R2) and Blaskici (R3);
- A2: Recina (R1), Bracana (R2), Rakov potok-Beram (R4+R5) and Momjan-Bazuje (R6+R7);
- A3: Recina (R1), Bracana (R2), Marcana (R8) and Momjan-Bazuje (R6+R7);
- A4: Recina (R1), Blaskici (R3), Rakov potok-Beram (R4+R5) and Momjan-Bazuje (R6+R7);
- A5: Recina (R1), Blaskici (R3), Marcana (R8) and Momjan-Bazuje (R6+R7);
- A6: Bracana (R2), Blaskici (R3) and Momjan-Bazuje (R6+R7);
- A7: Bracana (R2), Blaskici (R3) and Rakov potok-Beram (R4+R5);
- A8: Bracana (R2), Blaskici (R3) and Marcana (R8);

- A9: Bracana (R2), Rakov potok-Beram (R4+R5), Marcana (R8) and Momjan-Bazuje (R6+R7);
- A10: Blaskici (R3), Rakov potok-Beram (R4+R5) and Marcana (R8);
- A11: Recina (R1), Bracana (R2), Rakov potok-Beram (R4+R5) and Marcana (R8);
- A12: Recina (R1), Blaskici (R3), Rakov potok-Beram (R4+R5) and Marcana (R8).

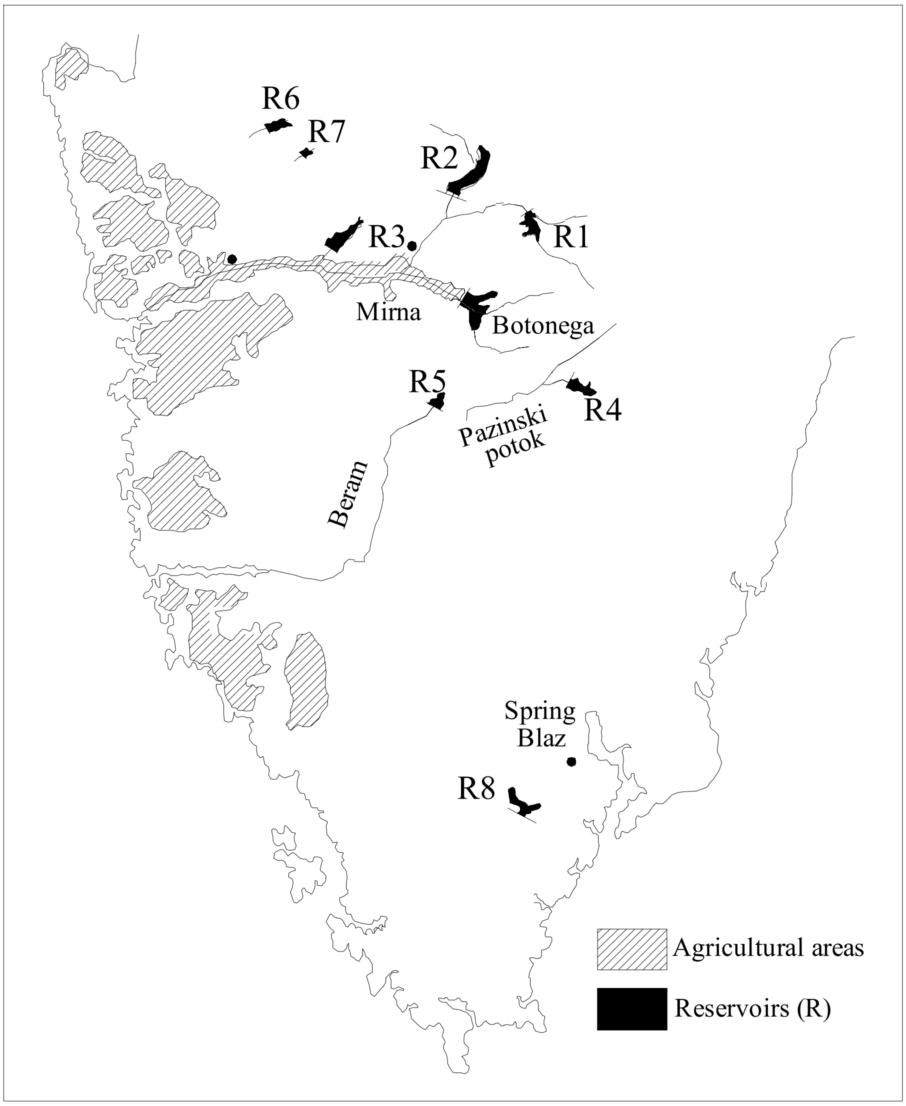

Fig. 2: Location of agricultural areas and reservoirs in western Istra

The assessment of alternatives was done on the base of ten different, commonly used criteria and corresponding measures established in the context of integral and universal analysis of the problem. The next criteria were chosen:

- **A,** irrigated area in ha,
- **FCV,** flood control volume (volume of flood wave that could be retained by the reservoirs included in the alternative) in m^3,
- economic criteria:
  - **C,** cost of m^3 of water for irrigation in euros,
  - **B,** net-benefit (with the rate of interest of 8%)) in euros,
  - **B/C**, benefit-cost ratio (with the rate of interest of 8%),

  - **IRR,** internal rate of return in %,
- environmental criteria:
  - **E/A,** the effect on the area that will be inundated by the reservoir (assessment using the sum of areas inundated by the reservoirs included in the alternative) in m^3,
  - **E/L,** the effect on the region landscape (assessment of bad looking of sides of reservoirs when the water level is low using the sum of dam highs included in the alternative) in m,
- social criteria:
  - **S/F,** negative impact of flooding villages by reservoirs (assessment by giving the estimation 1 if reservoirs included in the alternative cause flooding a community, otherwise the estimation is 0),
  - **S/R,** recreation (assessment by giving to every reservoir that can be used for recreation the estimation of 1 otherwise 0, summing estimations given to all reservoirs included in the alternative gives the final assessment).

The assessment of every alternative for each criterion is shown in Tab. 1.

Alternative	**A** (ha)	**FCV** (m^3*10^6)	**C** (eur)	**B** (za 8%) (mil.eur)	**B/C** (za 8%)	**IRR** (%)	**E/A** (m^2*10^3)	**E/L** (m)	**S/F**	**S/R**
1	18499	8,48	0,50	41,61	1,229	11,65%	33660	142	1	2
2	17475	11,72	0,42	64,94	1,434	14,08%	50370	170,8	0	2
3	17475	9,42	0,43	59,96	1,388	13,75%	47500	149,3	0	3
4	17950	6,47	0,52	24,57	1,134	9,81%	32470	193,8	1	1
5	17950	4,17	0,53	20,10	1,107	9,54%	29600	172,3	1	2
6	17735	6,19	0,50	39,49	1,211	11,19%	33350	146,3	1	1
7	18526	7,55	0,49	39,62	1,204	11,05%	42160	131,5	1	1
8	18526	5,25	0,51	36,78	1,186	10,93%	39290	110	1	2
9	17502	8,49	0,42	55,70	1,335	12,76%	56000	138,8	0	2
10	17977	3,24	0,54	11,04	1,055	8,76%	38100	161,8	1	1
11	18266	10,78	0,45	57,09	1,348	13,26%	56310	134,5	0	3
12	18741	5,53	0,54	17,94	1,091	9,33%	38410	157,5	1	2

Tab. 1: The assessment of the alternatives on the base of the chosen criteria

Criteria are often interdependent that is why their uncritical use can lead to unconsciously preferring certain solutions so it is necessary to conduct an examination of interdependency of criteria before approaching multi-criteria optimization.

Depending on the correlation coefficient R (or correlation coefficient raised to a square R^2) the interdependency/correlation between criteria can be[10]:

- for $1 < R < 0{,}75$ or $1 < R^2 < 0{,}5625$ the correlation is very pronounced,
- for $0{,}75 < R < 0{,}5$ or $0{,}5625 < R^2 < 0{,}25$ the correlation is pronounced,
- for $R < 0{,}5$ or $R^2 < 0{,}25$ the correlation is very weakly pronounced.

The results of the research on interdependency of chosen criteria are shown in Tab. 2 in which YES means a very pronounced correlation/interdependency, YES/NO a pronounced correlation/interdependency, NO a weakly pronounced correlation/interdependency, and the number written in parentheses means the correlation coefficient coefficient raised to a square (R^2).

Ranking of alternatives is performed using chosen computer softwares PROMCALC & GAIA V.3.2., ELECTRE TRI 2.0a and EXPERT CHOICE Pro 9.5. on the base of test plans that include different combinations of criteria and weights given to criteria (the weights are: 0 – the criterion is not taken into account, 1 – small weight of criterion, 2 – medium weight of criterion, 3 – large/big weight of criterion). Because of the obvious interdependence of economic criteria net-benefit (**B**), benefit-cost ratio (**B/C**) and internal rate of return (**IRR**) the last one was chosen to represent them.

	A	FCV	C	B	B/C	IRR	E/A	E/L	S/F	S/R
A	-	NO (0,1029)	**YES/NO (0,3693)**	NO	NO	NO (0,2331)	NO (0,1047)	NO (0,1391)	**YES/NO (0,3672)**	NO (0,0092)
FCV	NO (0,1029)	-	**YES (0,7429)**	**YES**	**YES**	**YES (0,8604)**	**YES/NO (0,5211)**	NO (0,0128)	**YES (0,6439)**	**YES/NO (0,3022)**
C	**YES/NO (0,3693)**	**YES (0,7429)**	-	**YES (0,9061)**	**YES (0,9298)**	**YES (0,9017)**	**YES (0,7302)**	NO (0,0463)	**YES (0,8737)**	**YES/NO (0,2966)**
B	NO	**YES**	**YES (0,9061)**	-	**YES (0,9774)**	**YES (0,9869)**	**YES**	NO	**YES**	**YES/NO**
B/C	NO	**YES**	**YES (0,9298)**	**YES (0,9774)**	-	**YES (0,9919)**	**YES**	NO	**YES**	**YES/NO**
IRR	NO (0,2331)	**YES (0,8604)**	**YES (0,9017)**	**YES (0,9869)**	**YES (0,9919)**	-	**YES (0,5753)**	NO (0,0785)	**YES (0,7561)**	**YES/NO (0,3906)**
E/A	NO (0,1047)	**YES/NO (0,5211)**	**YES (0,7302)**	**YES**	**YES**	**YES (0,5753)**	-	NO (0,1174)	**YES (0,8035)**	**YES/NO (0,3156)**
E/L	NO (0,1391)	NO (0,0128)	NO (0,0463)	NO	NO	NO (0,0785)	NO (0,1174)	-	NO (0,0062)	NO (0,0755)
S/F	**YES/NO (0,3672)**	**YES (0,6439)**	**YES (0,8737)**	**YES**	**YES**	**YES (0,7561)**	**YES (0,8035)**	NO (0,0062)	-	**YES/NO (0,4706)**
SK/R	NO (0,0092)	**YES/NO (0,3022)**	**YES/NO (0,2966)**	**YES/NO**	**YES/NO**	**YES/NO (0,3906)**	NO (0,0755)	NO (0,0755)	**YES/NO (0,4706)**	-

Tab. 2: Results of research on interdependency of chosen criteria

Test plans can be shown as:
Plan A – using the following criteria and alternating their weights:

A1 = A+C+IRR,
A2 = A+C+IRR+FCV,
A3 = A+C+IRR+FCV+E/A+E/L,
A4 = A+C+IRR+FCV+E/A+E/L+S/F and
A5 = A+C+IRR+FCV+E/A+E/L+S/F+S/R

Plan B – using only independent criteria:

B1 = A+C+E/L+S/R and
B2 = A+IKS+E/L+S/R

4 Results

All three methods gave almost equal results depending on defined criteria and criterion weights. PROMETHEE and ELECTRE TRI methods resulted suitable for selecting the best reservoir configuration in the analysed case of irrigating western Istra agricultural areas because the assessment of alternatives by the chosen criteria and criterion weights is expressed by exact values. It is verified that AHP method is more suitable for cases of multi-criteria optimization where the alternatives comparison as well as the criteria comparison is given by descriptive grades: equally important (grade 1), weakly more important (grade 3), strongly more important (grade 5),…., up to the grade of 9 or by intermediate values (grades

2, 4, 6,…), while the opposite assessment is shown by grades with reciprocal values (1/2, 1/3, …). PROMETHEE II and ELECTRE TRI methods have the possibility of using different preference functions (types of criteria), while AHP has not. Using PROMETHEE method an influence among the alternatives on the final ranking was noted while it does not occur if ELECTRE TRI and AHP methods are used PROMCALC&GAIA V.3.2. and ELECTRE TRI 2.0a softwares are able to compare more alternatives than EXPERT CHOICE Pro 9.5. software. In using all three softwares it is noticed that PROMCALC&GAIA V.3.2. is simpler than ELECTRE TRI 2.0a and EXPERT CHOICE Pro 9.5 that are quite complex to use in the beginning. Advantages and disadvantages of the chosen multi-criteria optimization methods and softwares are shown in Tab. 3.

METHOD/SOFTWARE	ADVANTAGES	DISADVANTAGES
PROMETHEE II	- gives the total preorder of alternatives - the possibility of using different preference/criteria functions (6 types of criteria)	- the influence among the alternatives on the final ranking - exact values of alternative assessment is needed as input data
PROMCALC&GAIA V.3.2	- the simplicity of inputting data - easy to survey - a larger number of alternatives and criteria than EXPERT CHOICE Pro 9.5 software	
ELECTRE TRI	- gives as a result alternatives sorted in categories - there is no influence among the alternatives on the final ranking - the possibility of using different preference/criterion functions (pseudo-criteria)	- exact values of alternative assessment is needed as input data
ELECTRE TRI 2.0a	- a larger number of alternatives and criteria than EXPERT CHOICE Pro 9.5 software	- the complexness of inputting data - difficult to survey the inputted data
AHP	- the possibility of different assessment of the comparison between alternatives and criteria (verbal, graphical, numerical) - it is possible to have sub-criteria	- there is only one type of criterion function (true criteria), other are not possible
EXPERT CHOICE Pro9.5	- easy to survey the hierarchy structure	- the complexness of inputting data - relatively limited number of alternatives and criteria

Tab. 3: Advantages and disadvantages of the chosen multi-criteria optimization methods/softwares

The employment of chosen multi-criteria optimization methods gave various results. The diversity of results is a consequence of a large number of criteria whose weights were changed on the base of test plans. The results are shown in Tab. 4.

	ALL USUAL/TRUE CRITERIA (type I)		PSEUDO-CRITERIA (type I and IV)
	Test plan A	Test plan B	Test plan A
PROMETHEE II PROMCALC&GAIA V.3.2.	2, 7, 8 , 11	8, 11	2, 11
ELECTRE TRI ELECTRE TRI 2.0 a	2, 3, 9, 12	2, 3, 5, 8, 9, 11, 12	2, 3, 7, 8, 9, 11, 12
AHP EXPERT CHOICE Pro 9.5	2, 11	8, 11	-

Tab. 4: The results of using chosen multi-criteria optimization methods for selecting the best reservoir configuration in western Istra

5 Conclusions

To solve the problem of selecting the best reservoir configuration for irrigating agricultural areas in western Istra PROMETHEE II and ELECTRE TRI methods resulted more appropriate because they use exact input values of criteria and such data were available in this case. They also offer the possibility of using different criterion functions that can describe better the decision-maker preferences. Softwares PROMCALC & GAIA V.3.2. and ELECTRE TRI 2.0a can elaborate a larger number of alternatives and criteria in relation to EXPERT CHOICE Pro 9.5. AHP method can be employed when the input data are exactly defined but it is more suitable when the comparison between alternatives and criteria is given.
From the aspect of preparing input data softwares PROMCALC & GAIA V.3.2. and ELECTRE TRI 2.0a, this is methods PROMETHEE II and ELECTRE TRI, demand the definition of criterion functions and ELECTRE TRI also the limits of categories. All mentioned implicates that some time needs to be spent on learning and becoming familiar with the parameters that have to be definited. On the aspect of using softwares PROMCALC & GAIA V.3.2. is easier to use in comparison to ELECTRE TRI 2.0a and EXPERT CHOICE Pro 9.5.
The application of multi-criteria optimization selected among twelve generated alternatives 2, 3, 7, 8, 9 and 11 as «better» and 1, 4, 5, 6, 10 and 12 as «worse» alternatives. The use of eight criteria (because of the obvious interdependence of economic criteria net-benefit, benefit-cost ratio and internal rate of return the last one was chosen to represent them all) if the interdependence of other criteria is not taken into account pointed out 2 and 11 as the best alternatives. However due to noticed interdependency among some criteria when only independent criteria are used alternatives 11 and 8, and in case of employing ELECTRE TRI alternatives 2, 3, 5, 9 and 12 (with 3 as the most often chosen alternative) are selected as best solutions. If the optimization had been conducted on the base of economic criteria only (taking into account economic criteria are mostly interdependent among them) the chosen best alternatives would be 2 or 11, while other such as 3 and 8, that are slightly worse from the economic aspect, but have certain advantages from environmental and social aspect in relation to mentioned alternatives 2 and 11, would be eliminated.
Employing PROMETHEE II and ELECTRE TRI with different criteria functions (usual/true criterion and level criterion/pseudo-criterion) has shown that the use of appropriate criterion

functions/pseudo-criteria gives more stable ranks less sensitive to changing of criterion weights. The variety of criterion functions/pseudo-criteria gives the decision-maker the possibility of better defining his preferences.
It is concluded that chosen multi-criteria optimization methods are, using critically evaluated input data, appropriate and useful for decision making (choice of optimal solution) in water management problems. Further research and study on improving the process of solving problems in water management should be done in the direction of better defining criteria and measures, better defining complex generated solutions and improving existing and developing new multi-criteria optimization methods.

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Regional Flood Frequency Analysis of L-moments

Silvia Kohnová, Ján Szolgay

Slovak University of Technology, Faculty of Civil Engineering, Department of Land and Water Resources Management, Radlinského 11, 813 68 Bratislava, Slovak Republic: kohnova@svf.stuba.sk, szolgay@svf.stuba.sk

Abstract

This study deals with an alternative approach to flood flow regionalisation based on L-moments. Flood records from 251 small and mid-sized basins with a catchment area ranging from 10 to 360 km^2 from the whole territory of Slovakia were used. An analysis of the seasonality in different regions has indicated that separate analyses of seasonal maximum discharges resulting from rainfall and snowmelt is necessary in order to decrease heterogeneity in the data due to the genetic origin of the flood events. Pooling groups of catchments were defined using clustering according to L-moment ratios L-Cv, L-Cs and L-Ck. Relationships for the computation of the mean annual snowmelt flood were derived using multiple regression. The appropriate flood frequency distribution was estimated using L-moment statistics. The applicability of such formulae using regional parameters in the computation of N-year discharges was tested in selected pooling group and compared with statistically computed values.

1. Introduction

The recent extreme floods in Slovakia and in Central Europe resulted into concerns about the reliability of flood frequency estimates especially in ungauged basins and increased interest in regional flood frequency analysis. The design discharge calculation in small and medium-sized ungauged catchments had mainly been based on empirical regional flood formulae. Other methods, such as the rational method, the unit hydrograph analysis, the SCS model and other mathematical models were far less employed (e.g. Mitková, et al., 2002, Miklánek et al., 2000, Svoboda et al., 2000). The growing number of stations with longer records makes it possible to test how some of the new concepts of regional homogeneity and regional flood frequency analysis reported in recent literature – (e.g., in Acreman and Sinclair (1989), Zrinji and Burn (1994), Meigh, et al. (1997), Hosking and Wallis (1997) and FEH (FEH, 1999)) perform in the estimation of design discharges in small and medium sized catchments. Some of these were developed under specific conditions, and modifications to them may be necessary under the rather heterogeneous geological and geomorphological conditions of Slovakia. Here, results achieved by the application of the Hosking and Wallis methodology (Hosking and Wallis (1997)), together with the index flood method using annual maximum snowmelt flood data from 251 basins in Slovakia were presented. The same method was applied to the summer floods in Kohnová, Szolgay (2002).

2. Data analysis

Seasonality analysis of flood occurrence in Kohnová (1997) and Čunderlik (1999) suggested, that for small and mid-sized catchments the frequency occurrence of rainfall-induced floods and snowmelt floods is comparable and they preferably should be treated separately in flood frequency analysis. For this study annual maximum snowmelt floods were chosen as input data. Flood data from 251 small and mid-sized catchments with the catchment area in a range from 10 to 360 km^2 from the whole territory of Slovakia were collected from the database of the Slovak Hydrometeorological Institute.

3. Identification of homogenous pooling groups

Numerous techniques have been used to identify homogeneous pooling groups or regional types for regional flood frequency analysis. In previous studies by Čunderlík (1999), Kohnová and Szolgay (2000, 2002), Solín (2002), catchments characteristics were used for the identification of homogeneous groups of catchments. In Čunderlík (1999) also at-site L-moment characteristics of annual maximum floods were used for pooling catchments into homogeneous groups and a method based on mapping of L-moment characteristics over the territory of Slovakia was suggested for assigning ungaguged catchments into the pooling groups. Here this approach was followed using at-site *L-Cv, L-Cs* and *L-Ck* values of annual maximum snowmelt floods as variables in the cluster analysis (K-means clustering with Euclidean metrics after Hartigan (1975)) for pooling catchments into regional types.

Since no estimate about the “correct“ number of clusters can be given a-priori, a balance was sought between using pooling groups that were too small or too large. Following experience from previous studies and the results of Čunderlík (1999), the minimisation of the within the groups and maximisation of the among the groups variance of the regionalisation parameters was also attempted. Finally 7 groups were formed and the resulting locations of catchments belonging to these pooling groups are presented in Fig. 1.

Subsequently, the Hosking and Wallis approach was used to test the homogeneity of the pooled catchments. The measure proposed by Hosking and Wallis (1997) for testing the homogeneity of proposed pooling groups compares the between site variation in sample *L-Cv* (coefficient of variation) values with the expected variation for a homogeneous pooling group. The method fits a four-parameter kappa distribution to the regional average *L-Cv* ratios. The estimated kappa distribution is used to generate 500 homogeneous pooling groups with population parameters equal to the regional average sample *L-Cv* ratios. The properties of the simulated homogeneous pooling group are compared to the sample *L-Cv* ratios as

$$H = \frac{(V - \mu_V)}{\sigma_V} \tag{1}$$

where μ_V is the mean of the simulated V values, and σ_V is the standard deviation of the simulated V values. For the sample and simulated pooling groups, respectively, V is calculated as

$$V = \left\{ \frac{\sum_{i=1}^{N} n_i (t^{(i)} - t^R)^2}{\sum_{i=1}^{N} n_i} \right\}^{1/2} \tag{2}$$

where N is the number of sites, n_i is the record length at the site i, $t^{(i)}$ is the sample *L-Cv* at site i, and t^R is the regional average sample *L-Cv*.

Also in this particular case this test was not based on data, which is independent from that used in the grouping of catchments (as recommended in Hosking and Wallis (1997)), it gives an indication about the variation of the L-moment characteristics in the groups and here it is used more as a measure of the quality of the clustering than a proof of regional homogeneity. On the other hand unacceptable high variation within a group would result into the rejection of the approach adopted here.

Tab. 1 Values of the Hosking-Wallis homogeneity measure (H) for the derived pooling groups and the final numbers of catchments included

Pooling group No.	H	No of catchments
1	-2,53	39
2	-2,99	21
3	-2,44	25
4	-0,50	45
5	-3,79	57
6	-3,54	30
7	-1,77	28

The negative values of H in Table 1 indicate that there is less dispersion among the at-site sample *L-Cv* than would be expected of a homogeneous region with independent at site frequency distributions. The most likely cause according to Hosking and Wallis (1997) is positive correlation between the data values at different sites, which causes the sample L-Cv values to be unusually close together. In our case this can be explained as the result of the pooling using these characteristics. Also the use of H as a heterogeneity measure in this case is compromised, since the selection of regions is based on sample L-moments, formally according to the threshold values of Hosking and Wallis the groups could be considered to be homogeneous.

4. Selection of the regional flood frequency distribution

To select the appropriate regional flood frequency distribution function for a particular pooling group, the Z^{DIST} goodness of fit test (Hosking and Wallis (1997)) and L-moment ratio diagrams were used in all pooling groups. The L-moment ratio diagram is a widely used tool for graphic interpretation and comparison of sample L-moment ratios *L-Cs* (skewness) and *L-Ck* (kurtosis) of various probability distributions. The goodness-of-fit test described by Hosking and Wallis (1997) is based on a comparison between sample *L-Ck* and population *L-Ck* for different distributions. The test statistic is Z^{DIST} and is given as:

$$Z^{DIST} = \frac{(\tau_4^{DIST} - \tau_4^R + B_4)}{\sigma_4} \tag{3}$$

where *DIST* refers to a candidate distribution, τ_4^{DIST} is the population *L-Ck* of the selected distribution, τ_4^R is the regional average sample *L-Ck*, B_4 is the bias of the regional average sample *L-Ck*, and σ_4 is the standard deviation of the regional average sample *L-Ck* (for details see Hosking and Wallis (1997)).

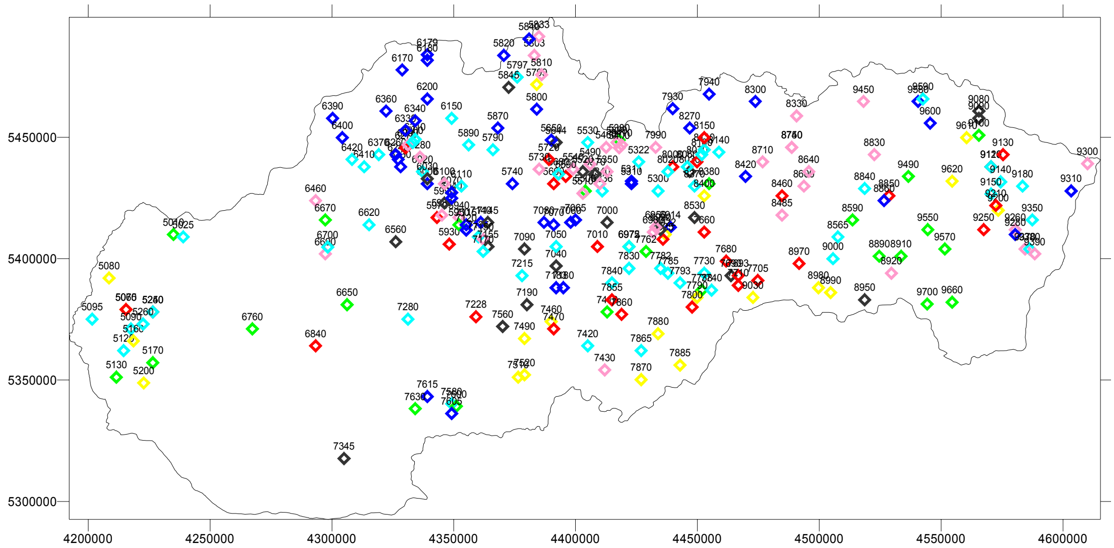

Fig. 1 Location of catchments in the pooling groups

In Fig.2 the L-moment ratio diagram for the seven pooling groups shows values of regional *L-Cs* and *L-Ck* averages. Fig. 3 represents the L-moment ratio diagram for the pooling group No. 4 and shows the dispersion of the L-moment ratios around the regional average value, which is marked as a big point. The GEV distribution, which was closest to the regional average value, was chosen in this group as the regional flood frequency distribution.

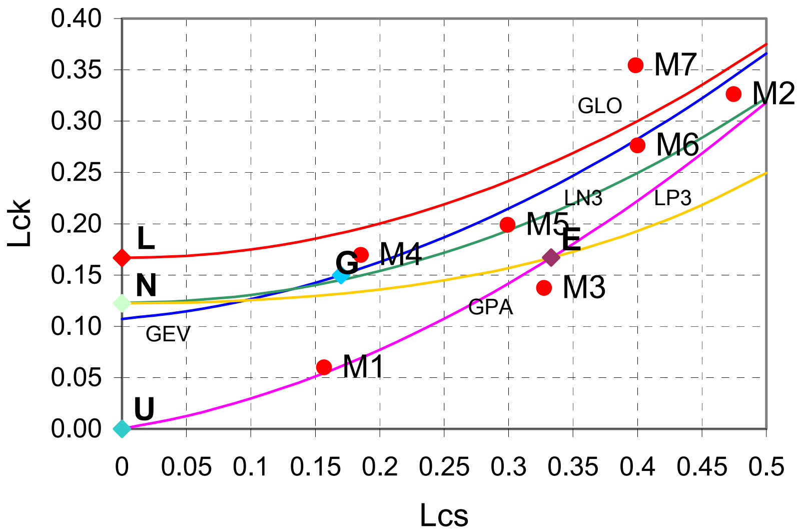

Fig.2 L-moment ratio diagram with regional averages for the seven pooling groups (M1-M7)

(legend of distributions functions: GLO: Generalized logistic, LN3: Lognormal, GEV: Generalized extrem value, GPA: Generalized Pareto, PE3: Pearson III, L: Logistic, N: normal, E- exponencial, G: Gumbel)

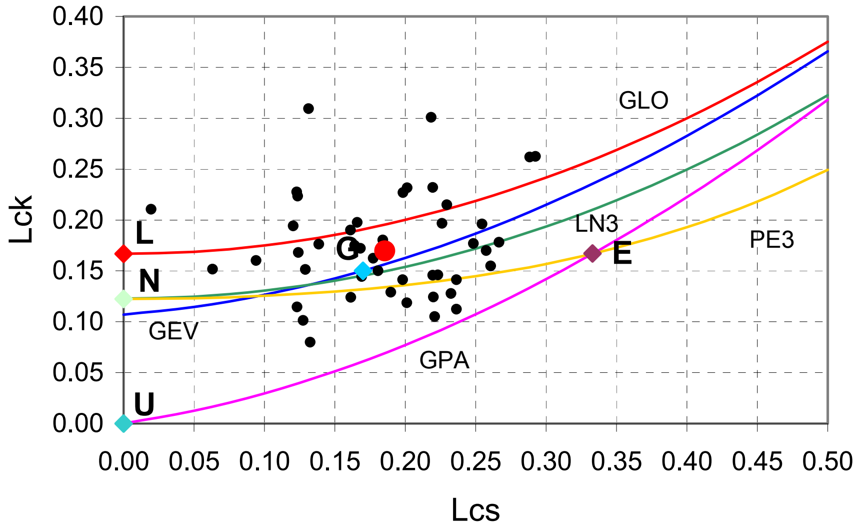

Fig.3 L-moment ratio diagram for the pooling group 4, regional average is marked as big point

(legend of distributions functions: GLO: Generalized logistic, LN3: Lognormal, GEV: Generalized extrem value, GPA: Generalized Pareto, PE3: Pearson III, L: Logistic, N: normal, E: exponencial, G: Gumbel U:Uniform)

5. N-year discharges estimation

For the estimation of N-year discharges the index flood approach was adopted (Dalrymple, 1960). The mean annual maximum snowmelt flood was selected as the index flood. 45 catchments belonging to the pooling group 4, were selected here to test the performance of the regional method. The index flood in this pooling group was estimated by multiple regression. The regional estimates of the N-year flood quantiles were computed in these catchments by multiplying the value of index flood and the N-year values from the dimensionless regional distribution. Statistical at-site design discharges estimates for the return periods of N = 5, 10, 20, 50 and 100 years were also computed using at site parameter values and a distribution function, which was selected as optimal for each of the given sites separately. These at site statistical estimates represent the basis for testing the performance of the regional method in our scenario, since these values would be normally used in engineering design.

In Fig.4 and 5 the comparison of the Q_{10} and Q_{100} values computed using the regional approach and statistical analysis for the selected 45 catchments from pooling group 4 is shown. Both for the 10-year and 100-year discharge the regional approach exhibits acceptable performance.

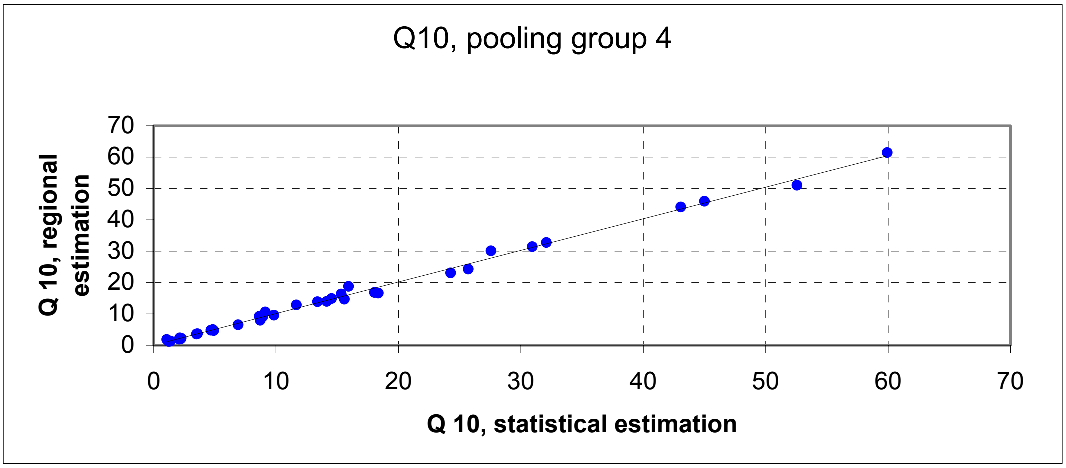

Fig.4 Comparison of the Q_{10} values computed using regional approach and statistical analysis in selected catchments of the pooling group 4.

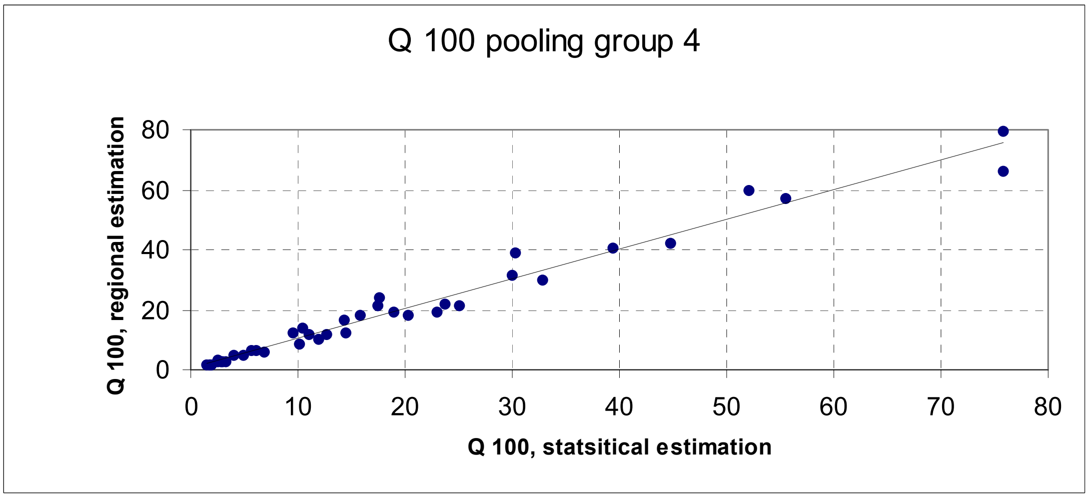

Fig.5 Comparison of the Q_{100} values computed using regional approach and statistical analysis in selected catchments of the pooling group 4.

Finally the relative differences *(X-Y)/Y* between the values of Q_{10} and Q_{100} computed using the regional approaches *(X)* and statistically *(Y)* at-site for each site in the pooling group were computed and compared. In Fig.6 the comparison of mean positive and negative values of relative differences of the *Q10* and *Q100* is shown.

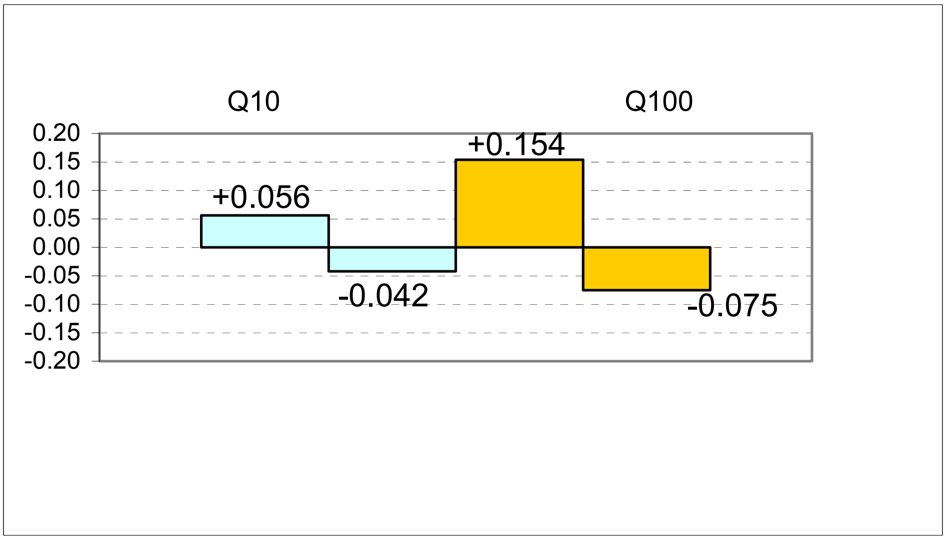

Fig.6 Mean positive and negative relative differences of the Q_{10} and Q_{100} values computed using regional approach and statistical analysis in selected catchments of the pooling group 4.

6. Conclusions

The concept of pooling presented in the study was based on the similarity of L-moment characteristics in the analysed catchments. Cluster analysis was used to define homogeneous pooling groups. The estimation of design discharges in the test-pooling group was based on the regional growth curve and the index-flood method. The under and over-estimation of design discharges within the pooling group, when compared with the statistical reference values, could not be avoided, but was much smaller than in previous studies based on regionalisation principles using physiographic catchment characteristics. The approach will have to be further tested in all pooling groups and the mapping method for analogous to that of Čunderlík (1999) will have to be developed in order to be able to use the method in ungauged catchments.

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Water Resources in Croatia in the 21st Century

Zorko Kos[a], Nevenka Ožanić[b]

[a] University of Rijeka, Faculty of Civil Engineering, V.C. Emina 5, 51000 Rijeka, CROATIA,
[b] University of Rijeka, Faculty of Civil Engineering, V.C. Emina 5, 51000 Rijeka, CROATIA, nozanic@gradri.hr

Abstract

The paper analyses water use in Croatia and their regions, especially in Istria region. The Republic of Croatia is a relatively young state; it was founded in 1991 after the disintegration of the former Socialist Republic of Yugoslavia. Embrace an area of 56.538 sq. km which makes 22,1 % of earlier Yugoslavia with more than 80 % of Adriatic littoral and almost all Adriatic islands. The total population of the country is 4,422 million people (2001), what is about 10 % less than at the previous one (1991).
The country has one of the best location in Europe, it is placed just in the middle of the northern hemisphere-around the 45° latitude N – where the Mediterranean sea deeply penetrate in the heart of Europe (Fig. 1). It is composed of two main wings, the continental, stretching eastward and the littoral along the Adriatic Sea.
The physical characteristics of the country: climate, water availability, natural beauties, preserved and clean environment are very suitable for settlements and specially for tourism and vacationing business all round the year. During the summer time along the sea and during the winter time at the mountains, as the Dynaric chain is stretching closely along the Adriatic coast.

Key words: water resources, Croatia, sustainable development, watery abundance, Istria

1 Introduction

Croatia is situated both along the Adriatic Sea and in the continental (pannonian) part of Europe with some mountainous areas (Dinara Mountain Chain). Taking into consideration the topographic features, the country can be divided into three main regions: the Pannonian plan (51 %), the mountainous area (17 %) and the littoral (32 %). While the Pasnnonian plane is the «granary of the country», the coastal belt is one of the most famous touristic areas of the world.
The straight-line length of the coast (from north to south) is about 580 km, but the well indented coast line with about 1.160 islands and cliffs (65 of which are settled) it makes some 5.750 km. The littoral belt has a very high amount of high quality available water, well foretasted area, clean and extended gravel beaches, very favorable climate and scarcely populated area.
Taking into consideration that the country has the population density of only 78 inhabitants per sq. km, what is the lowest among all EU countries, the existing sources of drinking water could satisfy the country for a long future and have considerable amount for export. The today's total water consumption for all purposes is the smallest among all EU countries – it is only 800 million cubic meters per year (1996), what makes about 2 % of the outflow from

domestic watersheds or about 0,5 % of the total outflow. It is not difficult to prove that in same areas the augmentation of water use will considerably alleviate the flood control and drainage problems.

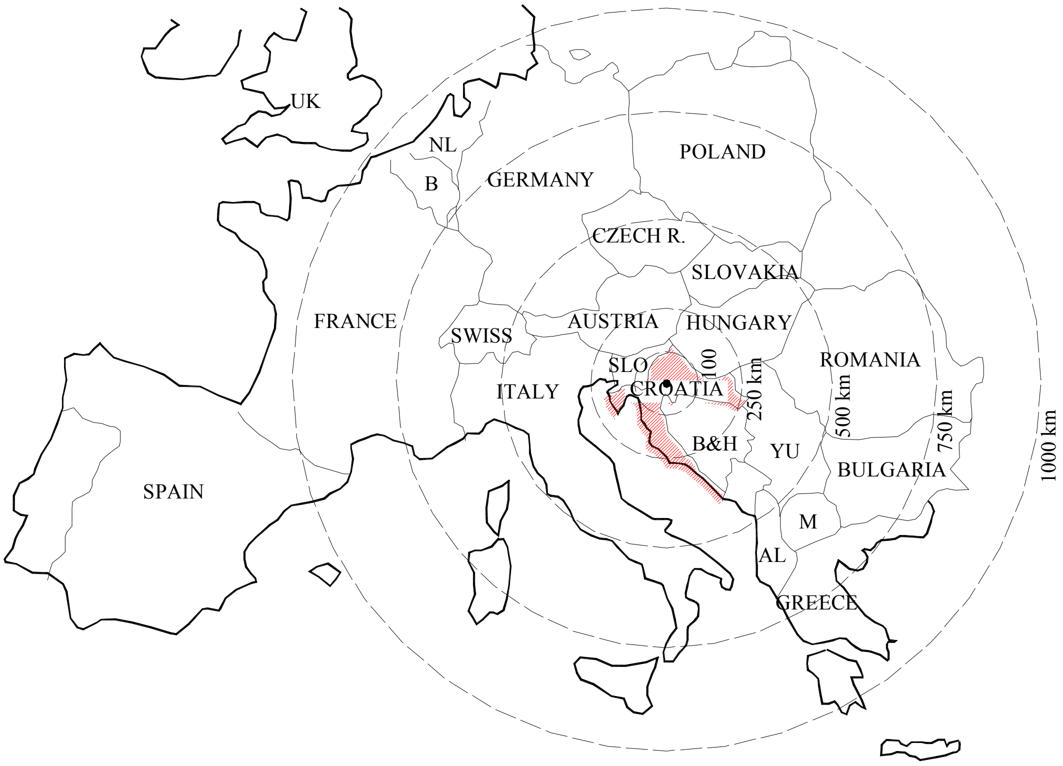

Fig. 1: Croatia in the heart of Europe

2 Analyses

2.1 The physical environment

Croatia is, for purpose of water resources, administratively divided into four regions: Eastern (Drava and Danube), Central (Sava), North Adriatic (Istrian and Littoral watersheds) and South Adriatic (Dalmatian watersheds). The same partition could be made taking into consideration the hydrometeorological criteria.

The main water resources features of the regions are:

- In the eastern region the most pronounced problem is flood control and drainage with available water which by far exceed the needs of the area. The future extension of water use could be realized for irrigation purposes.
- The central region is typically characterized as the transition area between the continental and Mediterranean area. For future development bough flood and erosion control as well as drainage and irrigation have to be implemented. The amount of available water is much higher than the needs.
- The Adriatic region, by all means, the most seriously effected by surplus and shortages of water supply in the country, because its specifically hydrogeological

regime. In general this area is the water richest part of the country as the precipitations varies between 1000 and 3000 mm yearly, but owing to the karstic character of the area the rain water quickly infiltrates in the underground and flows toward the sea appearing in the ground (or see bed) in form of numerous springs or vruljas.

Fig. 2: The four main meteorological regions of Croatia

As stated earlier, the Mediterranean climatic regions have an average yearly temperature along the coastal belt of about 14-15 °C. The rainfalls are dominantly of the orographic type, ranging from a minimum of about 700 mm in the southwest to about 3000 mm on mountainous area of the Gorski Kotar. These variations occur as a result of different orographic and climatic features and particularly from an impact of wet air masses from the see on the rear side of the Alpine chain and succeeding Dinaride masses. Nevertheless, the region frequently suffers of drought during summer time and is affected by floods on fall and spring time.

The region predominantly consists of two principal geological formations: Cretaceous limestones and Tertiary marls. The first one is characterized by typical karstic forms with considerable ground waters circulation and poor to negligent surface hydrography. The latter one is impermeable nearly through its extension and have a highly developed surface hydrography and extremely erodible cover.
As a result of different geological settings, two entirely different types of hydrogeological properties have been developed. In both cases, in the natural state, the available water resources are poor. In the former the available water percolates fast to the impermeable bedrock and flows toward the sea by usually unknown channels at depths which may very with the season. In the latter, atmospheric water runs fast down the steep slopes in torrential streams carrying large amount of suspended and bed load. In both cases, to use the greater part of available water, some works are needed: first some investigations and studies, and then the engineering structures to transport water by channels.

3 Results - Croatian sustainable water resources

The available Croatian renewable water resources in cubic meters per person per year is the highest in Europe (for total outflow) – as stated in the given table.

Table 1: Total outflow of renewable water in Croatia compared with some European countries

Country	Long period mean of outflow water					
	From domestic watershed			Total outflow		
	mil.m^3	m^3/st.	m^3/ha	mil.m^3	m^3/st.	m^3/ha
Croatia	44.590	9.320	7.890	171.870	35.720	30.400
Austria	56.300	7.497	34.710	90.200	12.010	55.610
Hungary	6.000	560	1.125	113.700	10.616	21.240
Italy	185.000	3.242	14.864	187.000	3.277	15.025
Denmark	11.000	2.110	4.150	11.000	2.110	4.150
Sweden	176.000	21.180	58.780	180.000	21.660	60.120
France	168.000	3.130	8.865	207.000	3.850	10.920
Germany	94.000	4.880	17.535	196.000	10.240	36.660
Spain	76.000	2.030	3.700	76.000	2.030	3.700

Source: *Gereš, D.: Manual for Irrigation and Drainage, II/6, pg. 122*

As the average yearly precipitations (in different areas) vary from 700 mm to about 3.500 mm, the overall mean is 1.088 mm. The total discharge of outflow amount to 171,8 km^3 of water what makes 30.400 m^3/ha, or 35.720 m^3/person. As the average yearly consumption – for all purposes – in recent years was of the range of 800 mil cu.m./year, or raufly 180 cubic meters per year per person. In other words the country use only about 0,5 % of their renewable water resources. Even if in the far future the consumption will be considerably increased (e.g. for irrigation) there is a large space to cover all this needs.
To have a more clear and understandable view of the problem concerned we will explain the proposed plan in a small deal of the country which makes about 5 % of the total territory – the Istra peninsula situated in the north Adriatic littoral. Istra is one of 20 Croatian provinces with an area of 2.900 km^2 and 210.000 inhabitants (2001) The average yearly rainfall is 1.100 mm, what makes, with an outflow of yearly coefficient of 0,4 an average yearly outflow of about

40,00 m^3/s. The biggest parts of these are underground waters. To use these waters, a dozen of storage basins have been planed, out of which three have been already constructed.

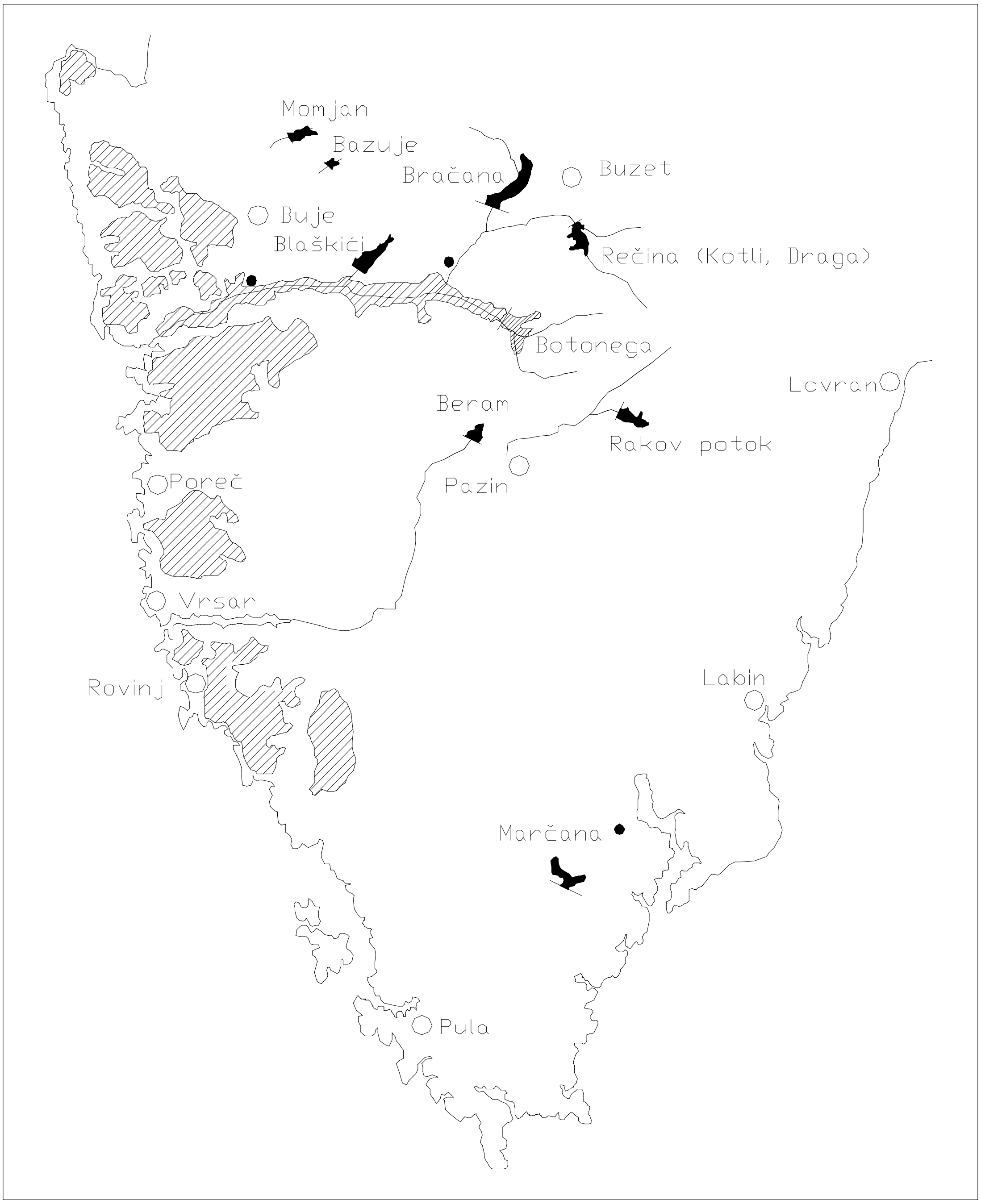

Fig. 3: Possible location for reservoir construction in Istra peninsula and agricultural surfaces

As stated above the total renewable water which can be used (surface and underground) is 21,62 m^3/s or roundly 680 mil. cu. meters. The water need – for all purposes (domestic and irrigation) for a long planning period (2050) results to be of the range of about 140 mil. cu.m. Namely, the only significant user of sweet water today (2003) is domestic and industrial supply with the consumption of about 40 million cubic meters in the 2000. The use of water for agricultural purposes (irrigation) is unknown but anyhow negligible. Plans have been made for seen the need of water – for all purposes – up to the year 2040. The results of these investigations are available as follows:

- Taking into consideration the maximal increasing rate of growth of 2 % yearly; the consumption of water for this purpose is totaling to 88,8 million cubic meters annually (in 2040)
- The agriculture – for irrigation purposes – actually does not use any water. Plans have been made to develop the irrigation systems on all land areas which are suitable for irrigated agriculture. It was find out that in the peninsula exist 21.754 hectares of land which could be irrigated. For implementation of such program it is necessary to provide some 51,3 million cubic meters of water annually.

Summarizing the above planed long – term water needs we came to the conclusion that Istra – in the middle of XXI. Century – could use maximum 140 million cubic meters of water (4,4 m^3/s) yearly what make about 20 % of possible ready available intakes or 7,7 % of total outflow.
Accordingly, only this small area could export annually about 540 million of cubic meter of water.
It is necessary to be stressed out that the greater part of this water infiltrate in the karstic underground, enriches with karstic-born element which considerably better the taste and appear at the surface by uprising springs.
Preparing the above mentioned plans, we took into consideration that the use of water could be made only by safeguarding the basic principle of sustainable development, and precisely:

- fulfill the development needs of today's generation without limitations the same for future ones;
- augmentation of the standard of living for today's generation with safeguarding of the ecosystem;
- ensure the dynamic balance of the earth's ecosystem in permanent duration.

Conclusions

Taking into consideration only the two meteorological regions situated along the Adriatic Sea (see fig. 2.) with a total surface of 25.740 sq. km, the ready usable yearly outflow could be of the range of six billion cubic meters per year. This calculation is made by extrapolation of Istrian data, but the real amount will be higher because the average rainfall is also higher.
The total amount which could be exported make only about 25 % of total yearly outflow of the studied area and is 7,5 times higher than the water used in all country (about 13%). For realizing such a program a lot of water works and other structures have to planned and implement, but in a smaller scale the «business» can start immediately.

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The Use of the Dual Reciprocity Method for Modelling of Venting Remediation

Karel Kovářík
University of Žilina

Abstract
The new numerical method (DRM) has been used for development of the mathematical model of the movement of soil air and vapours of pollutants. This model has been used for appraisal of remediation efficiency in the site of the former military base in Slovakia. The vapours are highly compressible and the modelling of their flow is a non-linear task. Therefore the standard BEM should contain volume integrals and the efficiency of algorithm is decreasing. The dual reciprocity method seems to be a very useful tool to overcome this problem.
The numerical models are very effective tools for remediation purposes. They can be used for evaluation of different possible remediation techniques and they also can help to find an optimal method of pumping etc.

Introduction
Venting is one of the important remediation methods used for decreasing the soil and groundwater pollution. It is based on pump and treats remediation method, but instead of the groundwater the soil air is pumped from the polluted area. The modelling of venting is more complicated than the models of groundwater flow because of the compressibility of flowing media (air). Because the soil air is more mobile than groundwater and liquid pollutants we can neglect the motion of the liquid phase and therefore the governing equation of the airflow could be simplified. The transport of vapour of pollutants in the porous media is described by the usual advection-dispersion equation. The both problems have been solved by the dual reciprocity method. This method was developed during the last ten years as the part of the boundary element method and the results look very interesting for the application in the other various fields.
The more detailed description of the basis of this method is presented in this paper.

Basic equations of flow and transport
Airflow is described by the continuity equation for compressible fluid

$$\frac{\partial}{\partial t}(\rho m) + \frac{\partial}{\partial x_i}(\rho v_i) = 0 . \tag{1}$$

where ρ is density of fluid and m is porosity of the porous medium. The rate of filtration is expressed as the function of the air pressure

$$v_i = \frac{k_p . g}{\nu} \frac{\partial p}{\partial x_i} \tag{2}$$

where k_p is the permeability coefficient [L^2], g is the gravity acceleration [LT^{-2}] and ν is the kinematical viscosity of the air [L^2T^{-1}]. The governing equation of the airflow can be expressed as

$$\frac{\partial}{\partial t}(\rho m) + \frac{k_p . g}{\nu} \frac{\partial}{\partial x_i}\left(\rho \frac{\partial p}{\partial x_j}\right) = 0 \tag{3}$$

The governing equation of the conservative transport of pollutants in the soil air is govern by the usual advection-dispersion equation

$$\frac{\partial(mC)}{\partial t} = \frac{\partial}{\partial x_i}\left(mD_{ij}\frac{\partial C}{\partial x_j}\right) - \frac{\partial}{\partial x_i}(v_i C) \tag{4}$$

where C is the concentration of pollutants in the vapours and D_{ij} is the tensor of the dispersion coefficients.

Dual Reciprocity Method

This method has developed from BEM in order to remove the major disadvantages such as the need of a division of a domain into sub domains in case of a non-linear problem (see Partridge et al. 1992)

The DR method was first used in a solution of the Poisson equation, which has this general form

$$\Delta\Phi = b\ . \tag{5}$$

Its solution can be expressed as the sum of the solution of a homogenous equation and a particular solution $\hat{\Phi}$. As finding this solution is very difficult, the method replaces the solution $\hat{\Phi}$ by a sequence of particular solutions $\hat{\Phi}_i$. The right side of Eq. (5) can be approximated by

$$b = \sum_{i=1}^{N+L} \alpha_i f_i\ , \tag{6}$$

where N is the number of boundary nodes and L is the number of inner points. α_i is a set of unknown coefficients and f_i are approximation functions. The particular solutions from the sequence must fulfil these equations

$$\Delta\hat{\Phi}_i = f_i\ . \tag{7}$$

The approximation functions f_i can be chosen as

$$f_i = 1 + r_i + r_i^2 + r_i^3 + \ldots + r_i^m\ . \tag{8}$$

The sequence of particular solutions $\hat{\Phi}_i$ has the form of a series

$$\hat{\Phi}_i = \sum_{k=0}^{m} \frac{r_i^{k+2}}{(k+2)^2}\ , \tag{9}$$

and the exterior normal derivative is

$$\frac{\partial\hat{\Phi}_i}{\partial n} = \frac{\partial r_i}{\partial n}\sum_{k=0}^{m}\frac{r_i^k}{k+2}\ . \tag{10}$$

If we substitute relations (5.128) and (5.129) to Eq. (11), we obtain the governing equation

$$\Delta\Phi = \sum_{i=1}^{N+L} \alpha_i\left(\Delta\hat{\Phi}_i\right). \tag{12}$$

Then we apply approaches similar to BEM and afterwards we acquire

$$c_k u_k + \int_\Gamma \frac{\partial w}{\partial n} u d\Gamma - \int_\Gamma w \frac{\partial u}{\partial n} d\Gamma = \sum_{i=1}^{N+L} \alpha_i \left(c_k \hat{\Phi}_{ki} + \int_\Gamma \frac{\partial w}{\partial n} \hat{\Phi}_i d\Gamma - \int_\Gamma w \frac{\partial \hat{\Phi}_i}{\partial n} d\Gamma \right). \tag{13}$$

The unknown coefficients in Eq. (5.128) can be determined from the known values b in the $N+L$ points.

$$\alpha_i = \sum_{j=1}^{N+L} F_{ij}^{-1} b_j \, , \tag{14}$$

where we denoted $\mathbf{F}^{-1}$ the inverse matrix to the matrix of approximation functions f_{ij}. A matrix form of Eq. (5.134) is

$$\mathbf{Hu} - \mathbf{Gq} = (\mathbf{H\hat{u}} - \mathbf{G\hat{q}})\mathbf{F}^{-1}\mathbf{b} \, . \tag{15}$$

The left side of Eq. (16) is totally identical with the BEM solution of homogenous equation. On the right side, there are matrices **H** and **G** along with the matrices of particular solutions. The matrices of particular solutions can be easily set up using relations of boundary element method (see Kovarik 2000). The vector **b** consists of given values of the planar inflow in every node of the net and in inner points. The DR method requires a certain number of inner points to be given where the inflow value b is set. The inclusion of boundary conditions is the same as in the BEM because the DR method is a variant of BEM.
After some derivations this method is even fit for a solution of pollutant transport. Before all else, we have to derive the basic equation of transport with the help of this coordinates' transformation

$$\tilde{x} = x \qquad \tilde{y} = y\sqrt{\Lambda} \qquad \Lambda = \frac{D_x}{D_y} \tag{17}$$

to the form of the Poisson equation

$$\frac{\partial^2 C}{\partial \tilde{x}^2} + \frac{\partial^2 C}{\partial \tilde{y}^2} = \frac{v_x}{D_x}\frac{\partial C}{\partial \tilde{x}} + \frac{v_y}{D_x}\frac{\partial C}{\partial \tilde{y}} + \frac{1}{D_x}\frac{\partial C}{\partial t} \, . \tag{18}$$

The right side of Eq. (19) is a sum of three terms b_1,b_2 and b_3 where

$$b_1 = \frac{v_x}{D_x}\frac{\partial C}{\partial \tilde{x}} \qquad b_2 = \frac{v_y}{D_x}\frac{\partial C}{\partial \tilde{y}} \qquad b_3 = \frac{R}{D_x}\frac{\partial C}{\partial t} \, . \tag{20}$$

We can apply the same approach as above to the first two terms b_1,b_2. Let us approximate them by functions f_i

$$b_1 = \sum_{i=1}^{N+L} \alpha_{1i} f_i \qquad b_2 = \sum_{i=1}^{N+L} \alpha_{2i} f_i \, . \tag{21}$$

N is here the number of nodes in the boundary elements and L is the number of inner points. To set the unknown coefficients α_{1i} and α_{2i} in Eq. (22), we need to know the values b_1,b_2 in $N+L$ points. Then the coefficients gain the form

$$\alpha_{1i} = \sum_{j=1}^{N+L} F_{ij}^{-1} b_{1j} \qquad \alpha_{2i} = \sum_{j=1}^{N+L} F_{ij}^{-1} b_{2j} \, . \tag{23}$$

After substituting for b_1,b_2 from Eq. (24), we obtain

$$\alpha_{1i} = \frac{v_x}{D_x}\sum_{j=1}^{N+L} F_{ij}^{-1}\frac{\partial C_j}{\partial \tilde{x}} \qquad \alpha_{2i} = \frac{v_y}{D_x}\sum_{j=1}^{N+L} F_{ij}^{-1}\frac{\partial C_j}{\partial \tilde{y}}. \tag{25}$$

Just to be illustrative, let us focus on an equation that has only b_1 on the right side

$$\frac{\partial^2 C}{\partial \tilde{x}^2} + \frac{\partial^2 C}{\partial \tilde{y}^2} = b_1. \tag{26}$$

If we use the notation of the boundary element method, we have

$$\mathbf{HC} - \mathbf{GQ} = \frac{v_x}{D_x}(\mathbf{H}\hat{\mathbf{C}} - \mathbf{G}\hat{\mathbf{Q}})\mathbf{F}^{-1}\frac{\partial \mathbf{C}}{\partial \tilde{x}}. \tag{27}$$

Now we need to express the concentration's derivative in N+L points. To do this we use this formula (see Patridge et al. 1992)

$$\frac{\partial \mathbf{C}}{\partial \tilde{x}} = \frac{\partial \mathbf{F}}{\partial \tilde{x}}\mathbf{F}^{-1}\mathbf{C}. \tag{28}$$

After substituting Eq. (29) into Eq. (30), we obtain

$$\mathbf{HC} - \mathbf{GQ} = \frac{v_x}{D_x}(\mathbf{H}\hat{\mathbf{C}} - \mathbf{G}\hat{\mathbf{Q}})\mathbf{F}^{-1}\frac{\partial \mathbf{F}}{\partial \tilde{x}}\mathbf{F}^{-1}\mathbf{C}. \tag{31}$$

Let us denote

$$\mathbf{A} = \frac{v_x}{D_x}(\mathbf{H}\hat{\mathbf{C}} - \mathbf{G}\hat{\mathbf{Q}})\mathbf{F}^{-1}\frac{\partial \mathbf{F}}{\partial \tilde{x}}\mathbf{F}^{-1} \tag{32}$$

and we have

$$(\mathbf{H} - \mathbf{A})\mathbf{C} - \mathbf{GQ} = 0. \tag{33}$$

If we do the same for the b_2 term, the matrix has a new form

$$\mathbf{A} = (\mathbf{H}\hat{\mathbf{C}} - \mathbf{G}\hat{\mathbf{Q}})\mathbf{F}^{-1}\left(\frac{v_x}{D_x}\frac{\partial \mathbf{F}}{\partial \tilde{x}} + \frac{v_y}{D_x}\frac{\partial \mathbf{F}}{\partial \tilde{y}}\right)\mathbf{F}^{-1}. \tag{34}$$

Equation (35) stays formally unchanged. The term b_3 includes a derivative of the concentration with respect to time and we can use the same approach as above. We can define

$$\alpha_{3i} = \frac{1}{D_x}\sum_{j=1}^{N+L} F_{ij}^{-1}\frac{\partial C_j}{\partial t}. \tag{36}$$

Equation (37) changes to

$$(\mathbf{H} - \mathbf{A})\mathbf{C} - \mathbf{GQ} = \mathbf{B}\frac{\partial \mathbf{C}}{\partial t}. \tag{38}$$

The matrix **B** has this form

$$\mathbf{B} = \frac{1}{D_x}(\mathbf{H}\hat{\mathbf{C}} - \mathbf{G}\hat{\mathbf{Q}})\mathbf{F}^{-1}. \tag{39}$$

Equation (40) can be solved by a standard approach (integration over time). Let us introduce new symbols

$$\mathbf{C} = (1-\xi_c)\mathbf{C}_1 + \xi_c\mathbf{C}_2 \qquad \mathbf{Q} = (1-\xi_Q)\mathbf{Q}_1 + \xi_Q\mathbf{C}_2, \tag{41}$$

where ξ_c, ξ_q are parameters that determine the course of values **C**, **Q** between times 1 and 2. Let us substitute this into Eq. (42) and subsequently we obtain a recurrent formula

$$\begin{aligned}&\left(\frac{1}{\Delta t}\mathbf{B} + \xi_c(\mathbf{H}-\mathbf{A})\right)\mathbf{C}_2 - \xi_Q\mathbf{G}\mathbf{Q}_2 = \\ &= \left(\frac{1}{\Delta t}\mathbf{B} - (1-\xi_c)(\mathbf{H}-\mathbf{A})\right)\mathbf{C}_1 + (1-\xi_Q)\,\mathbf{G}\mathbf{Q}_1\ ,\end{aligned} \tag{43}$$

which can be solved in every time step. We approximate also the course of variable Q. This stands for an exterior normal derivative of concentration. The use of the dual reciprocity method in groundwater hydraulics is still in its beginnings. Results that have been acquired by this method so far show an immense stability in time, which allows us to use larger time steps as compared to the finite element method.

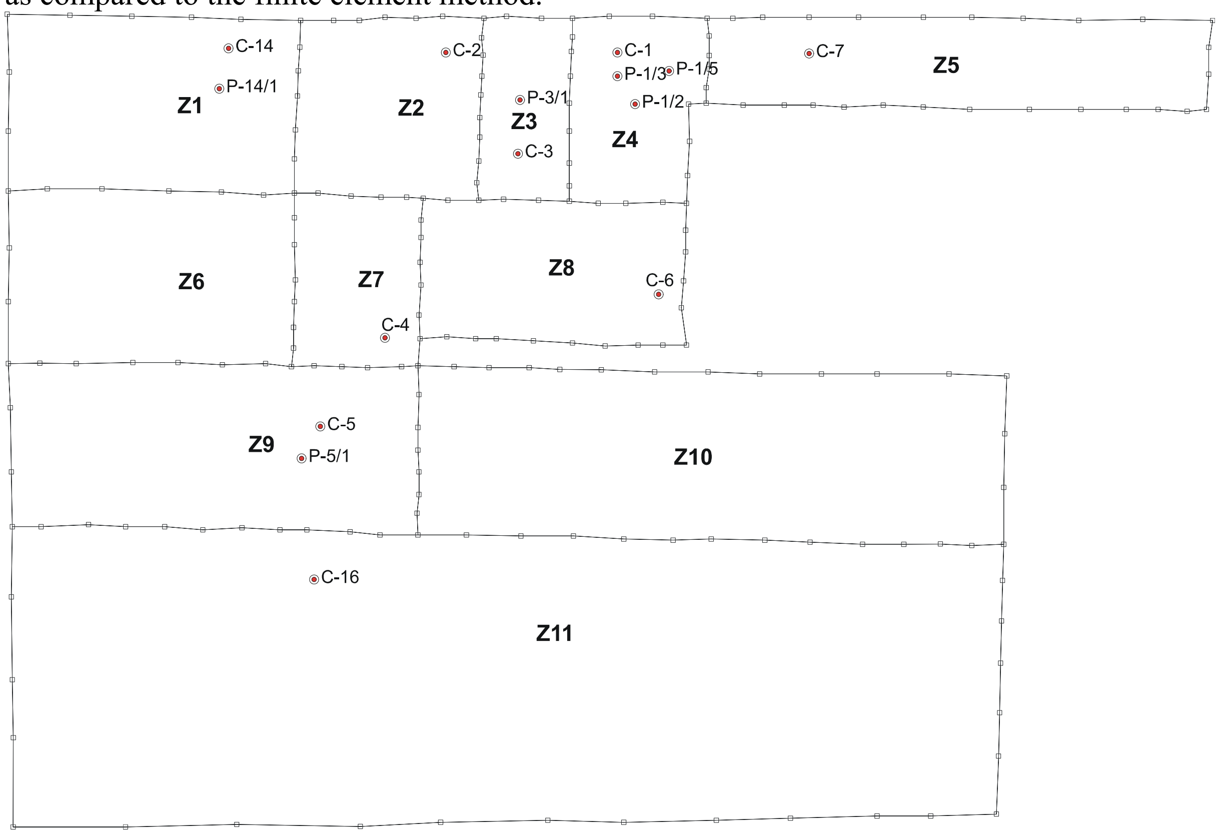

Fig. 1

Numerical model of venting

We can present a practical example of this method. There has been developed the computer system AIRBEFLOW using the aforementioned DR method. This system has been used for appraisal of remediation efficiency in the site of the former military base in Slovakia. The area of interest is the former parking place for military vehicles. It is covered by concrete pavement and the subsoil of this pavement is polluted mostly by oil hydrocarbons. The polluted areas have been documented by hydrogeological survey and the venting boreholes have been situated in the centres of these areas. These boreholes served to the common pumping test. The test was used to verify the model of the airflow. The modelling area has irregular shape (see Fig.1) because there is a system of buildings, which creates the impervious part of boundary. The rest of boundary is an open area, where the pressure is equal to atmospherical pressure. This part represents the boundary condition of the first order, where the difference pressure is zero. The area has been divided into 11 zones and permeability coefficients of these zones are fixed during the process of the verification of the model. The values of these coefficients see Tab.1

Borehole	Yield [$l.s^{-1}$]	Permeability K_p [m^2]
C-1	12.78	1.00E-11
C-2	13.89	2.75E-11
C-3	12.78	1.35E-10
C-4	12.78	8.01E-12
C-5	12.78	1.45E-11
C-6	9.44	1.75E-11
C-7	9.44	9.78E-12
C-14	11.11	1.56E-11
C-16	12.78	2.29E-11

Tab.1

Values of pumped yield from every borehole are also presented in Tab.1.
Contours of values of negative pressure during this test are presented on Fig.2 as the result of the model.

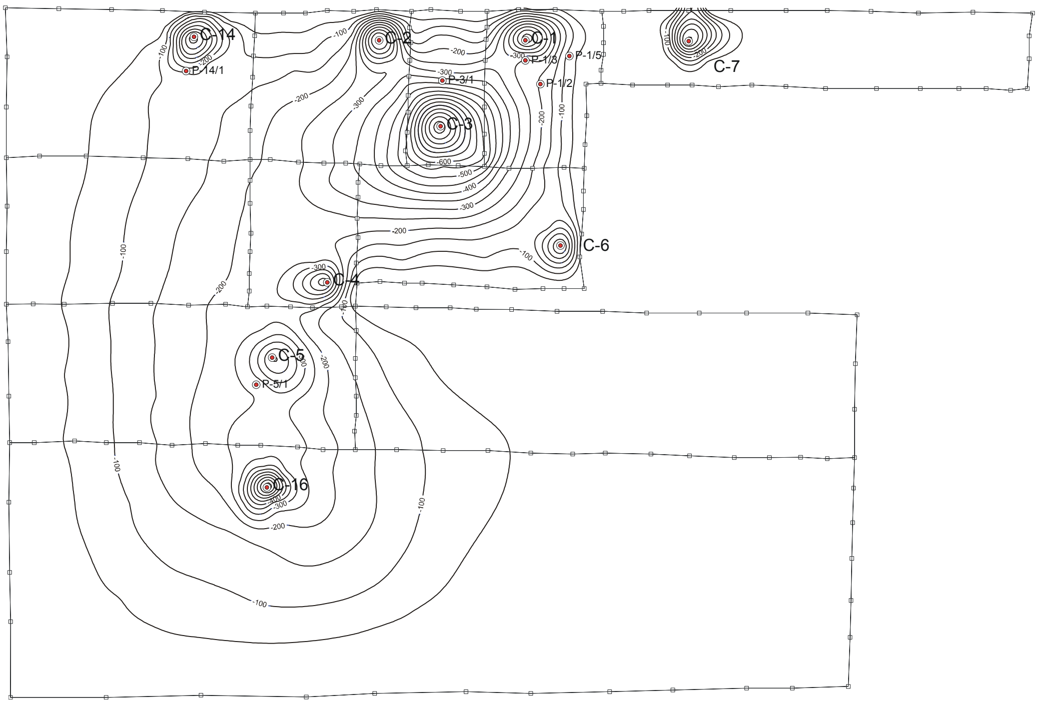

Fig. 2

The remediation pumping shall be conducted from the first seven boreholes C-1 till C-7 (see Tab.2). The total pumped yield is 14 $l.s^{-1}$.

Borehole	C-1	C-2	C-3	C-4	C-5	C-6	C-7
Yield [$l.s^{-1}$]	0.571	1.570	7.704	0.610	1.655	1.332	0.558

Tab.2

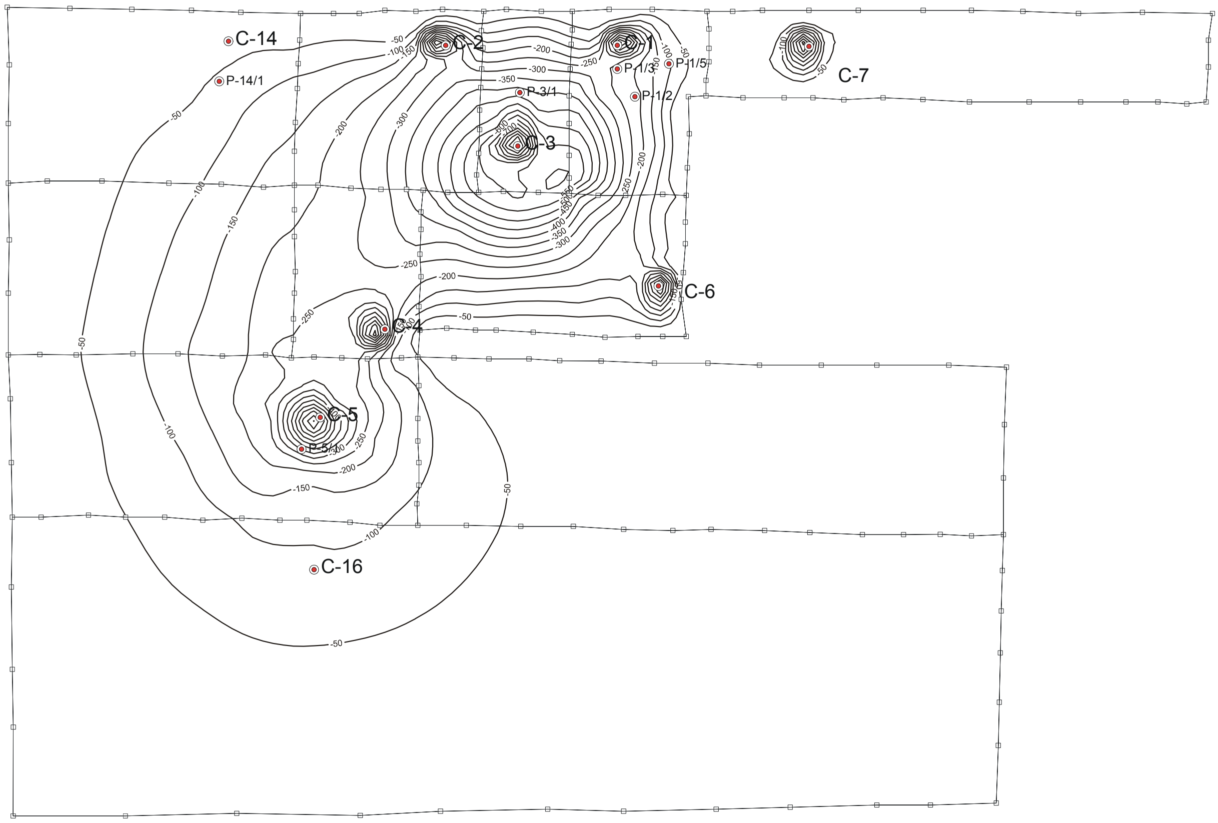

Fig. 3

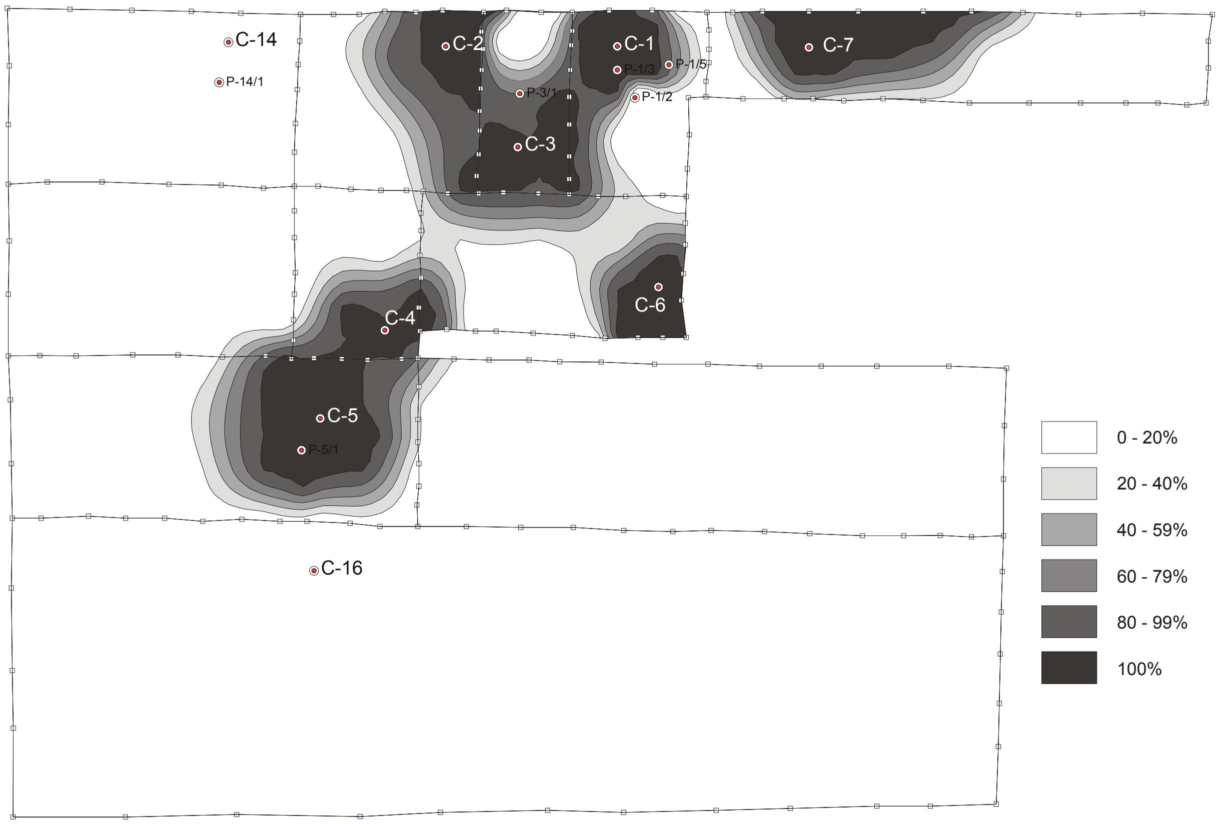

Fig. 4

A result of the model of airflow is presented in the form of contours of negative pressure (Fig.3).

Initial distribution of values of relative concentration at time $t_0=0$ is based on the result of a monitoring of the vapour concentration (see Fig.4). When the remediation pumping starts the concentration will be decreasing very rapidly during the first 10 hours (Fig.5)

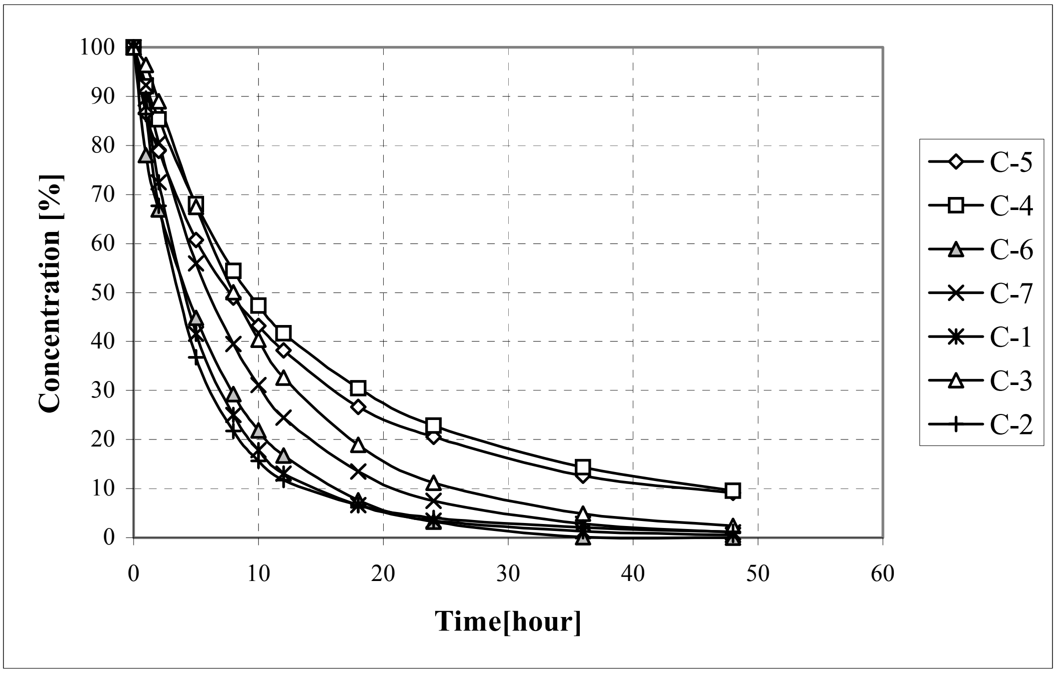

Fig.5

The areal distribution of air pollution after 24 hours of remediation pumping can be seen on Fig.6

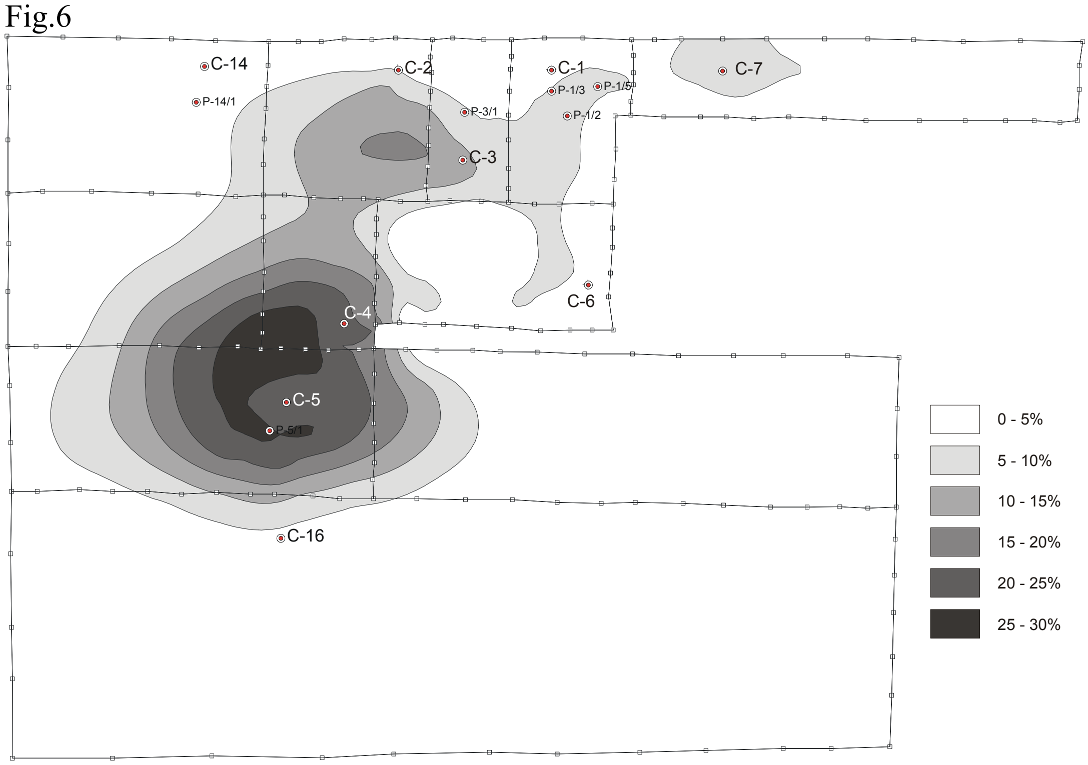

Fig. 6

Contours of concentration on Fig.6 shows that the concentration of pollution in all pumped boreholes decreases but it still remains in the area around borehole C-5. In this area is creating a region with relatively higher concentration of pollutants. Therefore the second version of remediation pumping has been prepared and the total pumped yield is increased to 21.7 $l.s^{-1}$ (see Tab.3).

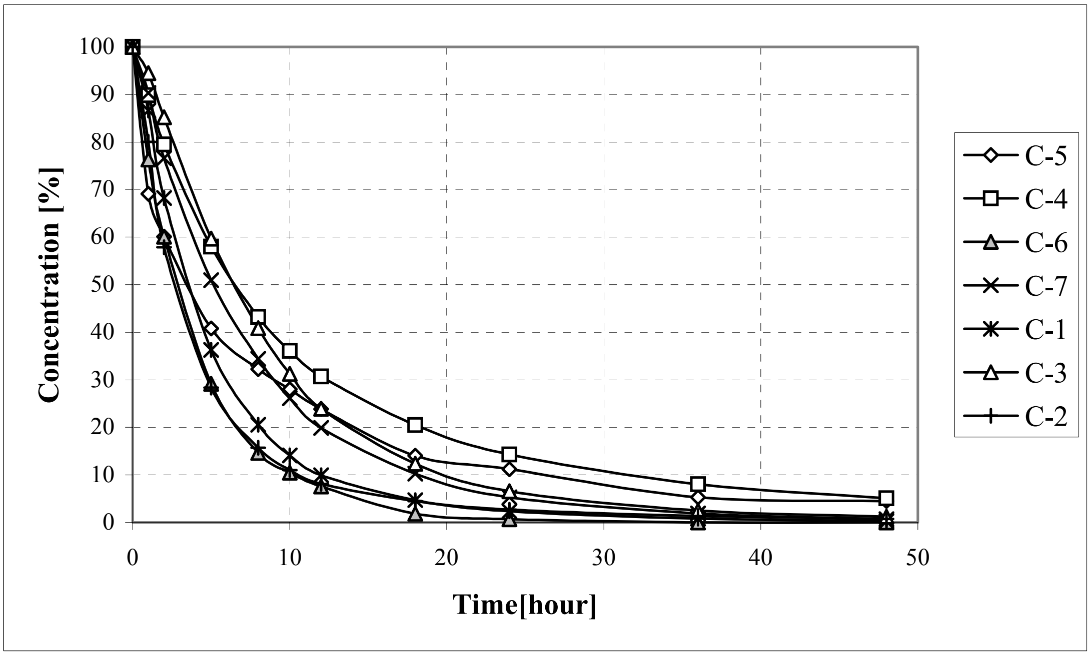

Fig.7

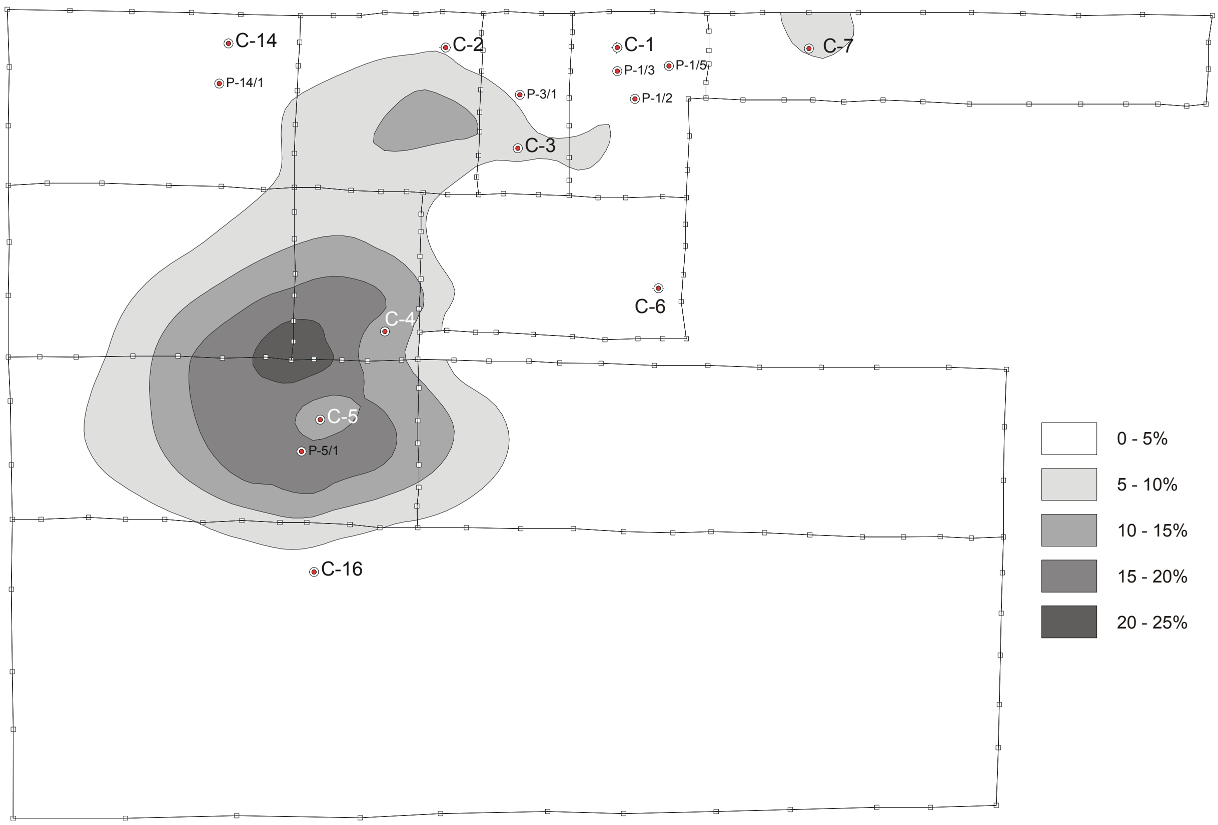

Fig.8

Borehole	C-1	C-2	C-3	C-4	C-5	C-6	C-7
Yield [$l.s^{-1}$]	1.269	3.491	7.704	1.356	3.680	2.962	1.241

Tab.3

Of course the concentration of pollution is now decreasing more rapidly (see Fig.8).
Contours of relative concentration are presented in Fig. 8. We can see that there are still higher concentrations of pollutants in the region between boreholes C-5 and C-4. There is created watershed dividing and the velocity of airflow is very slow and therefore the concentration of pollutant remains higher than in the vicinity of pumped boreholes.

Conclusions

The dual reciprocity method seems to be a good tool for modelling a non-linear task because it excludes use of internal elements. The next step shall be focused on including a term of desorption into advection-dispersion equation because the desorption of oil hydrocarbons is the main source of vapour in the soil media.

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Disinfection of water facilities

Jozef Kriš, Ivana Mahríková, Oskar Čermák
Slovak University of Technology, Department of Sanitary Engineering, Radlinskeho 11, 813 68 Bratislava, Slovakia, mailto: kris@svf.stuba.sk ,:mahrikov@svf.stuba.sk
Tel: 02/ 59274615, Fax: 02/ 52921184

Abstract:

The topic of paper is disinfection of water facilities in water treatment plants and other facilities. Our department prepare new Slovak Standard, which describe several possibilities of the water disinfection using chemical and physical processes. Chemicals used for treatment of water intended for human consumption: chlorine and its compounds, ozone and ultraviolet radiation. Requirements and testing.

Introduction

The organisms create in water a necessary, permanent and specific biocenosis, which biotope is water. There can exist cyanophytes, algae, bacteria and many other organisms in it. Except of this, there are various pathogenic bacteria and viruses that can cause illnesses of the population. There are above all causal agents of frequently arising enteric infections, but also other unpleasant and dangerous illnesses. Owing to their adverse influence on human organism, the pathogenic germs of viruses and bacteria cannot be present in water used for drinking purposes. To prevent the risk of water-borne infection, it is necessary to provide the health control and securing of water (water disinfection). To disinfection of drinking water are generally used the chemical methods of disinfection. The mostly used disinfection agents are chlorine, sodium hypochlorite, chlorine dioxide, less used are hydrogen peroxide and ozone. Nowadays there is a frequent use of physical method of water disinfection by means of UV radiation.

This norm holds for designing, construction, pressure examinations and operation of devices and objects of health securing of water (further just *water disinfection management*). It defines the agents used for water disinfection as for example chlorine (Cl_2) and its compounds: chlorine dioxide (ClO_2), sodium hypochlorite (NaClO), further ozone (O_3), hydrogen peroxide (H_2O_2) and in water management operations of waterworks organisations, industrial enterprises and municipal water management devices (pools) and similar the UV radiation. The management of health water securing in water management services is represented by complex of objects or parts of objects and devices for storage, preparation and dosing of disinfection agent and measuring of its concentration in water. This contains generally these premises:

a) Main storage;

b) Operation storage;

c) Chlorine chamber with antechamber, eventually a chamber for dosing of disinfection agent;

d) Room for ventilation and heating facility;

e) Deposit of safety tools and other facilities.

DESIGNING AND CONSTRUCTION OF FACILITIES OF HEALTH WATER SECURING MANAGEMENT

The disposition and technical design of *main deposit* of disinfection agents, chlorine, sodium hypochlorite), sodium chlorite, hydrochloric acid or hydrogen peroxide is normally proposed for three-months consumption of chlorine. By determination of the number of places for pressure tanks 100% of full and 50% of empty pressure tanks are used. It is proposed as a segregate facility, which has two inputs (the gate and the door), usually situated on opposed sides of the tank. The facility for sub-pressure venting and the control room are placed in separate room, if the main storage is placed out of the pourpresture with operation buildings. It is recommended to place a covered manipulation ground in front of the gate. In reasonable cases, when for water disinfection sodium hypochlorite, sodium chlorite, hydrochloric acid or hydrogen peroxide are used, the main storage need not to be a segregate facility with sub-pressure venting, instead the conventional ventilation can be used. To the main storage can be additionally built out the operational chlorine storage with antechamber, as well as the room for the sub-pressure venting device and heating. The *operation storage* need not to be a segregate facility with sub-pressure venting, instead the conventional ventilation can be used. The operation storage of pressure tanks is designed as a segregated room in the ground floor of the operation facility with segregate entrance from outside. The operation chlorine storage may contain the operation pressure tanks in number, necessary for the highest present demand of chlorine and the same number of emergency pressure tanks. If the highest present demand of chlorine is not determined, the operation storage can contain maximum four 500 l barrels, therefrom maximum two of them may be operational, or ten 40 l bottles, therefrom maximum three operational. The operation storage may contain such a number of free barrel holders that is corresponding with number of operation barrels. The operation storage of other disinfection agents contains generally non-pressure tanks in number needed for maximum demand of disinfection agent and the same number of emergency tanks.

By short-term storing of *chlorine dioxide* dilutions the manipulation must comply with STN EN 12671. When using chlorine dioxide as a disinfection agent, there must be established segregate storage for gaseous chlorine, for sodium chlorite in compliance with STN EN 938 and also for hydrochloric acid (STN EN 939).

Sodium chlorite is stored in double-cup containers or in a tub without direct connection to the drainage system. The measuring of levels in tanks must be secured in form of electric output or a liquid-level gauge. Sodium chlorite cannot be discharged directly into the sewage system (STN EN 938). The main storage of high-capacity tanks must be equipped with neutralization tank, and if there are used the double-cup containers that do not allow the transfer of sodium chlorite into other container, as well as in case of placement of the container in a retention tub, which cannot be at the same time used as a neutralization tank. The main storage must also dispose of the source of power water

Sodium hypochlorite is stored in closed containers according to STN EN 901.

Hydrochloric acid is stored in closed containers or in double-cup containers, which are placed in a tub without direct connection to the drainage system, according to STN EN 939. At the same time the measuring of levels in tanks must be secured in form of electric output or a liquid-level gauge. The main storage of high-capacity tanks must be equipped with neutralization tank, and if there are used the double-cup containers that do not allow the transfer of the received volume of hydrochloric acid into other container, as well as in case of placement of the container in a retention tub, which cannot be used as a neutralization tank. The main storage must also dispose with the source of power water.

By utilization of *hydrogen peroxide* as a disinfection agent is required a segregate room and the hydrogen peroxide is stored in closed containers, which are accommodated to this purpose according to STN EN 902.

Ozone is produced in an ozone generator directly in the city. STN EN 1278 defines the details of its location.

UV radiation is generated directly on the place and requires the manipulation space necessary for connection of UV generator into the system. Generally is considered the space for storage of reserve and worn mercury lamps.

By designing of the main operational storage and small **chlorine chamber** must be respected the distance from other facilities according to the figure 1. By designing of main storage of pressure tasks for chlorine is necessary to consider the surrounding terrain. There cannot be undulations around the storage.

Figure 1 – The minimum respectable distance [m] of chlorine storage from other facilities in depending on the capacity of pressure tank storage.

Facility sort	Amount of stored chlorine [l]	Number of 40 l chlorine bottles	Minimum distance of the storage from the facility [m]
Operation storage*⁾	to 20 000	to 500	12
	to 120 000	to 3000	15
	over 120 000	over 3000	20
Flat-buildings			
	to 2 000	to 50	12
	over 2 000	over 50	25
Non-flat buildings and the mass civil defence shelters	without respect to the storage capacity		30
Communications			
	to 2 000	to 50	5**⁾
	over 2 000	over 50	10**⁾
Undulations, shafts, windows, cellar entrances and similarly	to 2 000	to 50	5
	to 20 000	to 500	12
	to 120 000	to 3 000	15
	over 120 000	over 3 000	20

*⁾ Relates to main storage or to the whole storage capacity in the main and operation storage, if both storages are situated in one common object.
**⁾ If other regulations do not rule otherwise.

If the disinfection management of water represents one segregated object, its disposition solution according to figure 1 in dependence on stored amount of containers for disinfection agent. If the main storage is situated outside of the fencing of waterwork areal, it must be separately fenced.

The main chlorine storage is a segregate fire section, where at most 20 000 l of chlorine in bottles or barrels. Bearing structures and dividing structures must be fireproof. From every fire section must be a separated exit to the free space. The rooms must be equipped with a protection against lightning and at the same time they must not be drained directly to the public sewerage system, but must be equipped with neutralization tank

Ventilation and heating

The mail chlorine storage does not need to be heated. The sub-pressure ventilation is designed for changing of the air at least five times in an hour. The ventilation must be controlled by a push-button, placed by the storage entrance both outside and inside of it and must be equipped with optical signalling. Moreover, the storage must be equipped with an air offtake directly from the floor of the room and with an intake of adequate amount of fresh air.

The main storage of sodium chlorite barrels does not need to be heated. When storing these chemicals in high-capacity tanks, the storage should be heated and the recommended temperature is 10 °C.

The construction of the main and operation storage and of small chlorinating chamber must avoid the light penetration into the storage space, and prevent so the temperature to exceed 35 ºC. The recommended storage temperature of *sodium hypochlorite* is under 15 °C and the minimum temperature need not decrease under 0 °C. *Hydrochloric acid* is stored under 15 °C, the recommended temperature for storage of components for *chlorine dioxide* making *should* not exceed 25 °C and its minimum temperature need not decrease under 10 °C.

Most appropriate for the rooms are steam-heating, warm air heating or electric heating. Optimal temperature in operation storage, in the room for dosing of disinfection agents and in small chlorinating chamber is recommended between 20 and 25 ºC, if the supplier of whole facility (not only chlorinator). The temperature must be between 10 and 35 ºC.

Operation storage, the room for dosing of disinfection agents and the small chlorinating chamber must be equipped with sub-pressure ventilation, which must be dimensed for changing of the air at least five times in an hour. The ventilation tubing is conducted to the height of 1 m over the roof of the highest building, within a ten-metres radius from operation storage.

WATER DISINFECTION MANAGEMENT FACILITIES

Chlorine is used as a disinfection agent, in main chlorine storage and in the room for dosing of disinfection agent are generally placed following devices: storage bottles with holders, storage barrels with seating, eventually devices for pressure tank manipulation, chemicals as for example sodium hypochlorite, sodium chlorite, hydrochloric acid and hydrogen peroxide, signalling equipment.

Following devices are possible to be placed in the operation storage: operation barrels, available barrels, restricted number of reserve barrels or bottles, force plant, compression chlorine distributor, vacuum chlorinator, vacuum chlorine distributor, devices for manipulation with pressure barrels, scales for pressure tanks, alarm equipment.

Chlorinators that are placed in the room for dosing of disinfection agent may be pressure or vacuum. The configurations of chlorinators respectively of the devices for production and dosing of chlorine, sodium hypochlorite, chlorine dioxide, ozone, hydrogen peroxide and UV radiation are generally assessed by producers of these devices. Individual segments of gaseous chlorine must be closing, whereby every of them must capable of safe outgasing. The distributor of gaseous chlorine must dispose of reliable securing device (backward closure), which is situated prior to its flowing into water and prevents the intrusion of water into the compression tank. If the conduit with chlorine in sub-pressure state or with chlorine water runs through other rooms, it must be suceptible to control and the temperature in these rooms need not to be lower than 5 °C. The underground sub-pressure distributor of chlorine or the distributor of chlorine water must be piped in a direct chase with unified sloping. The pipeline in the chase must not made be of demountable connections. The chase cannot be conducted under objects.

The temperature in chase shall not decrease under 1 °C. For short segments is possible to use a fender max. 3 m long. Longer fenders of the conduit of chlorine water and sub-pressure chlorine distributors are not recommended. There can be placed both the sub-pressure conduit with chlorine and the distributor of pressure water or the conduit with chlorine water and the distributor of pressure water in one chase.

During installation and before puttig in operation of chlorine management there must be accomplished the technical inspection and pressure exams of firmness and tightness of the distributor of pressure and sub-pressure gaseous chlorine and chlorine water by required overpressure.

Technical requirements of chlorinators, dosing pumps and injectors

The construction and production of chlorinators, dosing pumps and injectors must enable the simple installation, adaptation and operation of the device. The chlorinators, dosing pumps and injectors must be made of the materials that are resistant to the corrosive chlorine impact, or their components must be made of the materials, which are equipped with protective coat against chlorine impacts. An amount of disinfection mean that is supplied to the consumption place must comply with values, stated in the Art. 272/1994 Coll. on human health protection, as amended in later regulations. Chlorinators, dosing pumps and injectors must be operable by room temperature of 10 °C to 35 °C. The chlorine dosing must be stopped automatically by closing of feed water inflow or by decrease of its pressure. The dosing of chlorine through chlorinator must stop automatically after closing of feed water inflow or by decrease of its pressure. The dosing of chlorine in vacuum chlorinators must stop automatically by vacuum decrease in dosing conduit with gaseous chlorine. The content of undissolved substances in transport water mast be lower than 25 mg/l and moreover must not contain fibrous substances. By the operation of chlorinator it is necessary to keep security standards according to STN 83 2003. Every chlorinator, dosing pump and injector must have the attest of the producer that confirms, that the chlorinator has been examined and screened and it has met the requirements.

Construction design

The main chlorine storage is designed as a ground object with fireproof construction system (according to STN 73 0804), where the enclosure walls must by provided with construction of type D1. Storage without windows is recommended. If the windows are designed, there must be, the intrusion of sunlight on pressure tanks must be prevented with help of another measure. The gate and door must be constructions of type D1 as well. The floor must be flat, fireproof, with non-skid surface. Ceramic paving with socel by the door is recommended. The height of the room should be at least 3 m.

The chlorine chamber and small chlorine chamber must be equipped with an antechamber and ventilation room as well as with the door of D1 construction. Its recommended for the floor to be of ceramic paving as well as the wall tiling, which shall be 1,8 m high. The storages: (the main one and the operation one) with chlorine tanks must be equipped with wind direction indicator, that in case of bigger (accident) gas-escape, the enterprise parts located in the wind direction, could be timely alarmed to the danger. The construction of chlorine chambers and storages is proposed in compliance with requirements of fire safety of STN 73 0804. The environment of facilities of health water securing is considered according to STN 33 0300. The approach of the lighting of the facilities is made in accordance with STN 36 0451. The work emergency lighting is designed on places, where the device must be operated durably also by voltage loss. In the rooms that are without permanent service is recommended to design except of the standard lighting also a complementary lighting on the level of emergency orientation lighting.

OPERATION AND MAINTENANCE

For operation of the management of health securing of water there is a local operation order, processed in accordance with STN 38 6405. It is a part of the operation order of water management operation and contains all available information on operation and construction of the disinfection device. Technical reviews and exams of reserved gas devices are realized according to the relevant, generally binding regulation at least once a year, in compliance with instructions of the producer or distributor of the device. Technical reviews of the operating equipment (chlorinator, injector) are provided at least once a quarter-year term. The technical reviews of ozonizers, UV radiators and chlorine dioxide generators are provided at least once a half-year term. Technical reviews of dosing pumps are provided at least once a year. Functionality of safety valves is controlled at least once a week. The vent must join into sucking of necessary sub-pressure ventilation. The dosing of the disinfection agent must be proportional to the flow and controllable. The concentration of chlorine dioxide in the back part of the facility cannot increase over 25 g/l of ClO_2 (owing to explosion danger). By operation accident by non-pressure connection after the reactor, there must be assigned dilution to the concentration < 8 g/l ClO_2 . In the back part of the device there must not be created the air bubble and the chlorine dioxide cannot vent. The dosing places must be designed that way that the batch dilution is apportioned equally in the conduit cross section. For securing of operation is recommended to install a close and a pressure valve. It is recommended to do so that way, that by the installation would be not necessary to drain water out of the device. By putting the disinfection device into operation is necessary to determine the worker and his substitute that will be responsible for service and operation of the device. For putting the device into operation and for schooling of the operation and service personal should be responsible the supplier of the device or the person appointed by authorized distributor. Control, purification and service activities shall be provided by the producer of UV reactor and shall be described in the operation book (e.g. lamp exchange, senzor calibration). The operation book should be placed near the device.

Manipulation and discharge of pressure tanks

The tanks must be protected against the impact. Their manipulation must be careful. The loaded bottles, barrels and containers must be appropriately protected against the effects of sun radiation. Before the use of the pressure container it is necessary to control the state of containers, accessories, marking etc. If there is a damage detected, they will be returned to the operator of the filling station with description of the damage. To the time of the return of faulty pressure containers it is necessary to of mark them visibly to avoid confusion. If its not possible to discharge the chlorine from operation storage into the distributor owing to unreliability of valves or other damage, and if the security will be not threatened by transportation, the containers must be returned to the operator of filling station, with assignation of the damage. If the security reasons do not allow transporting of containers back to the filling station, he will ask the chlorine distributor for sending the expert that will take the necessary measures.

Maintenance and storage of pressure containers

The maintenance of chlorine containers must be provided only in filling stations or test-rooms according to STN 07 8305. On the door of main and operation chlorine storage, respectively on components for production of chlorine dioxide, ozone or hydrogen peroxide, the chlorine chamber and small chlorine chamber must be, according to STN 01 8014, placed an information sheet with identification of the used chemical and identification of maximum number of stored pressure containers, respectively there must be placed the negative sheets. Further are here, according to STN 01 8012, placed the sheets: “No Admittance to Unauthorized Persons” and “Danger of Poison Gas“. The requirements of use of identification, symbols and signals for security providing and health protection on workplace are designated by Regulation of SR Government No. 444/2001 Coll.

SECURITY AND HEALTH PROTECTION ON WORKING PLACE

The general security rules for chlorine are alleged in STN EN 937, for chlorine dioxide in STN EN 12 671, for ozone in STN EN 1278 and for UV radiation in articles of the norm prepared. The staff, which will take part in the pressure test of gaseous chlorine distribution must be demonstrably acknowledged with operation regulations, technological testing approach, related to provisions and alarm standards, further with the principles of toxic impacts of chlorine and the first aid rules.

The working places, where the pressure tests of the gaseous chlorine distribution are made must be equipped with basic protection means for work with chlorine. The staff that operates the chlorine management must be demonstrably expertly prepared to these activities. The persons that take part on manipulation and transportation of pressure containers for chlorine must be informed in within the range of local operation order and gas alarming system.

The main, operation storage of chlorine and chlorine chamber must be for case of chlorine leakage equipped with alarm signalling device – with honker and watch light. The small chlorine chambers must be equipped with signalling device only in case of increased movement of persons. If increased chlorine content is signalised in room, where barrels are, the service staff can enter the room only with use of breathing apparatus. By damage discovery the regulations of local operation order and emergency alarm order are followed.

If there occurs only a smell of chlorine caused by untightness of junction, the untight place must be immediately detected with help of ammonia liquor. Only persons that are trained to this purpose can detect these, they must use the anti-gas protective masks with filter against chlorine, or resuscitators, up to the range of the damage.

By operation and main chlorine storage and chlorine chamber, near to the entrance into the antechamber must be placed a box with number of well-tried protective masks with filters against chlorine and reserve filters and with personal protective tools, according to the local operation order. The protective tools for small chlorine chambers must be placed near by the entrance. If there is an isolated facility without permanent service considered, it is possible to allow the placement of protective tools in the installation vehicle. In case of the operation and maintenance of more chlorine management facilities without permanent centralized control it is always necessary to keep breathing apparatus and protective masks together with reserve filter and prescribed protective equipment in installation vehicle in accordance with local operation order. In case of work injuries, accidents and technical equipment breakdown it is inevitable to act according to security provisions. Ventilation of room contaminated by chlorine shall be regulated with respect to site and its surrounding in compliance with alarm order and current atmospheric conditions in the way that do not exceed the highest concentrations of chlorine in atmosphere in surroundings of ventilated room. Chemicals for accident remediation are shown in table no.2.

Table 2 – Theoretical amount of chemicals needed for removal of one kilogram of Cl_2 released by the accident

Amount of chemicals (in kg)	Chemical	Application
1,13	NaOH (caustic soda)	33 % solution
1,50	Na_2CO_3 (Sodium carbonate)	12 % solution
1,04	$Ca(OH)_2$ (calcium hydroxide)	10 lime milk
0,88	$Na_2S_2O_3 . 5\,H_2O$ (anti-chlorine)	10 % solution

It is inevitable to wear protective glasses during the visual control of switched on UV lamp (ordinary glass eventually plastic provides sufficient eye protection). Long-term effect of UV radiation causes erubescence and burned skin. UV emitters can be hot. It requires careful manipulation. Before built in of new lamp it is necessary to clean protective tube by alcohol. Workplace intended for manipulation with chemicals and UV reactors has to be equipped with sufficient number of appropriate fire extinguishers, first aid kit and relevant instruments.

Conclusion

Selection of methods and agents for water disinfection depends on various factors as water amount, treatment process, local conditions as well as economic conditions. In accordance to valid legislation it is necessary to ensure safety of water intended for human consumption in water management plants, organizations, industrial plants, agricultural and communal water management facilities.

This article considers the different approaches and requirements for operation objects, storage facilities, preparation and dosing of disinfection agents for particular methods of health securing of water. The involved requirements are a summary of data presented in new STN 75 5050 „Disinfection in water facilities", which was after the acceptation brought out by SUTN in Juny 2003.

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[18] (75 8406).
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Morphological Changes in Large Croatian Rivers

Prof. Neven Kuspilić, Ph.D., B.Sc., C.E.
Assis. Damir Bekić, C.E.

Address:
University of Zagreb, Civil Engineering Faculty, Croatia
Kačićeva 26, 10000 Zagreb, Croatia

Email:
kuspa@grad.hr
damirb@grad.hr

ABSTRACT:
Permanent changes of channel geometry is one of the major river characteristics. They occur in more or less stabilized boundaries with direction to dynamic balance. Human activities have a large contribution in modification of this dynamic balance. In large Croatian rivers modification of river bed geometry are clearly caused by human activities. Due to stabilized river banks, geometric changes are indicated in river bed degradation. This paper gives analysis of channel depth changes in transfer zone of river Sava and channel depth changes in depositional zone of river Drava. Both rivers are navigable on these sequences, and channel geometry changes have a large impact on navigability purpose. On river Sava additional issue is water level degradation which causes troubles in use of rivers. It is possible to make strong relation between river bed degradation and water-level degradation. Additional river bed degradation is caused by gravel exploitation and dredging for increasing waterway depth. This paper gives critical review with recommended actions to lower negative impact of river bed degradation on water regime.

1. Introduction

Sava and Drava are two large Croatian rivers. By regulation level, in water management mean, they belong in group of "heavily modified water bodies". Beside being major water recipients of large Croatian areas, water management's moment of these two rivers is great. For that reason they where in the past (and today but less) subject of regulation actions on water body and water regime changes. Now days these river embankments are stabilized by themselves or with the regulation structures. Current condition is mainly kept with the scheduled regulation actions. More obvious changes on both watercourses are caused by the river depth changes. Consequently there are changes in water regime.

2. Morphological changes on the river Sava

In the middle 70's significant changes were registered which was seen as a progression of minimal and mean annual water levels on a serial number of water ganges on the river Sava. During a period of 20 years (1970-1990) the mean annual change is showing permanent decrease of minimal, mean and also maximum water level.

Table 2::1 shows the overview of a mean annual water level decrease on water meters on the river Sava.

Table 2::1. Mean annual change of water level and water area for the period 1970-1990.

Water meter	Minimal annual water level [cm]	Mean annual water level [cm]	Maximum annual water level [cm]	Estimated increase of wetted area [m^2]
Jasenovac	-8,4	-7,0	-3,5	250
Mačkovac	-3,5	-5,2	-3,0	920
Sl. Kobaš	-3,0	-5,2	-4,2	290
Sl. Brod	-2,7	-5,8	-4,1	300
Županja	-4,8	-7,1	-4,5	96

During this period of 20 years the change in cross-section area had been recorded on the water ganges. This 20-years-change was estimated according to cross-sections soundings [I] and is shown in Table 2::1.

This behaviour trend is explained by many hydrotechnical actions on the river Sava, mostly done in the 70's and 80's of the last century, which main purposes were flood protection and partially the use of water for thermo and nuclear plant. The exploitation of construction materials (sand and gravel) from the river banks made also a significant influence.

To estimate the influence of each previously mentioned parameter on the water level decrease is very difficult. On one hand water levels are decreased due to the changes of water regime, and on the other due to the river bed degradation, which is consequence of the hydrological change or of the commercial sand and gravel dredging.

For the cca. 30-km-section on the river Sava (from Šamac to Županja) a hydraulic analysis of the cross-section changes was performed [II]. On this section river bed degradation is obvious, just as on the whole river Sava. River bed cross-section, sounded in the period of 20 years (1981-2000), have been used in analysis. For different period of discharge (30% - 95% duration) water levels were calculated and compared. Figure 2::1 shows relative change of water levels. On the figure it can be seen that the river-bed degradation has influenced the water level decrease up to 20 %, which is absolutely 37 cm for the 95% water level duration.

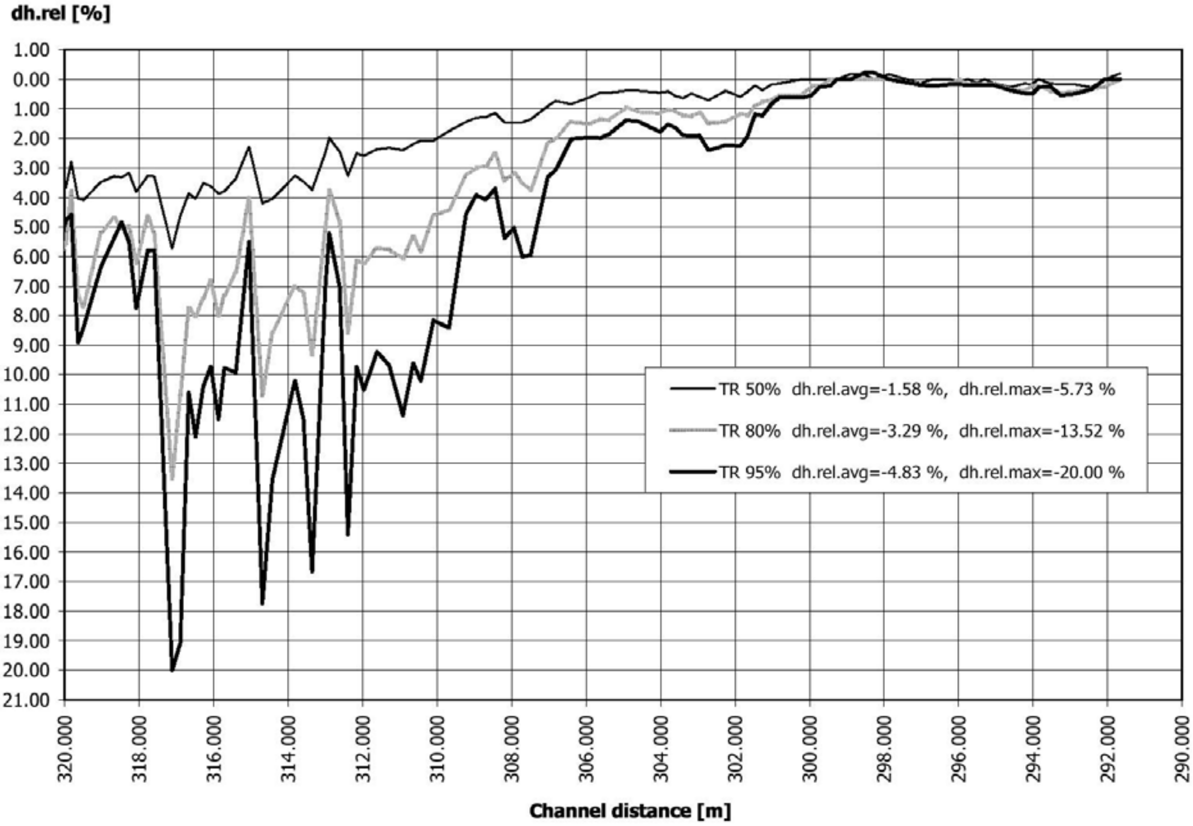


Figure 2::1. Relative water level change of the river Sava section from Šamac to Županja.

3. Morphological changes on the river Drava

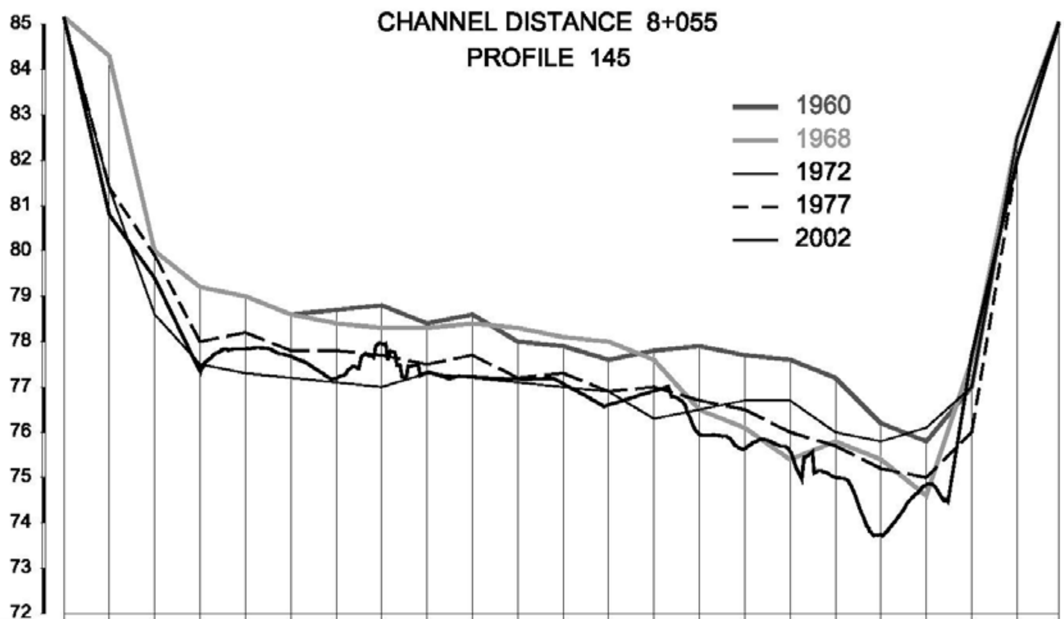
In the past the river Drava was a subject of significant regulation actions, mainly meander cutting and bank protectioning. They have resulted significant decrease of river length (cca. 40% for the whole watercourse). The progression of this process in the past is shown in Table 3::1.

Table 3::1. Watercourse length change of the river Drava from mouth to Osijek

Year	1784	1830	1842/46	1860	1886	1904	1966/68	1987/88
Length [km]	32	31,2	29,1	20,4	23,6	22,2	21,2	20,9

Now days the bank of the river Drava are relatively stable and their morphological change shows river bed degradation process. In this paper is presented the example of the depositional zone of the river Drava, near the mouth into Danube, in the length of 22 km. The water level in this section is strongly influenced by back water of river the Danube. In hydraulical sense, this Drava section as a depositional zone is different from the previously analyzed Sava section as a transfer zone.

According to more or less permanent soundings of river cross-sections, it is evident that depth change process is increasing. On the Figure 3::1 the river bed changes are shown on one characteristical profile.



DISTANCE	0.00	10.00	20.00	30.00	40.00	50.00	60.00	70.00	80.00	90.00	100.00	110.00	120.00	130.00	140.00	150.00	160.00	170.00	180.00	190.00	200.00	210.00	219.19
ELEVATION	85.16	84.30	80.00	79.20	79.00	78.60	78.70	78.80	78.40	78.60	78.00	77.90	77.60	77.80	77.90	77.70	77.60	77.20	76.20	75.80	77.00	82.50	85.05

Figure 3::1. Characteristic profile on the river Drava.

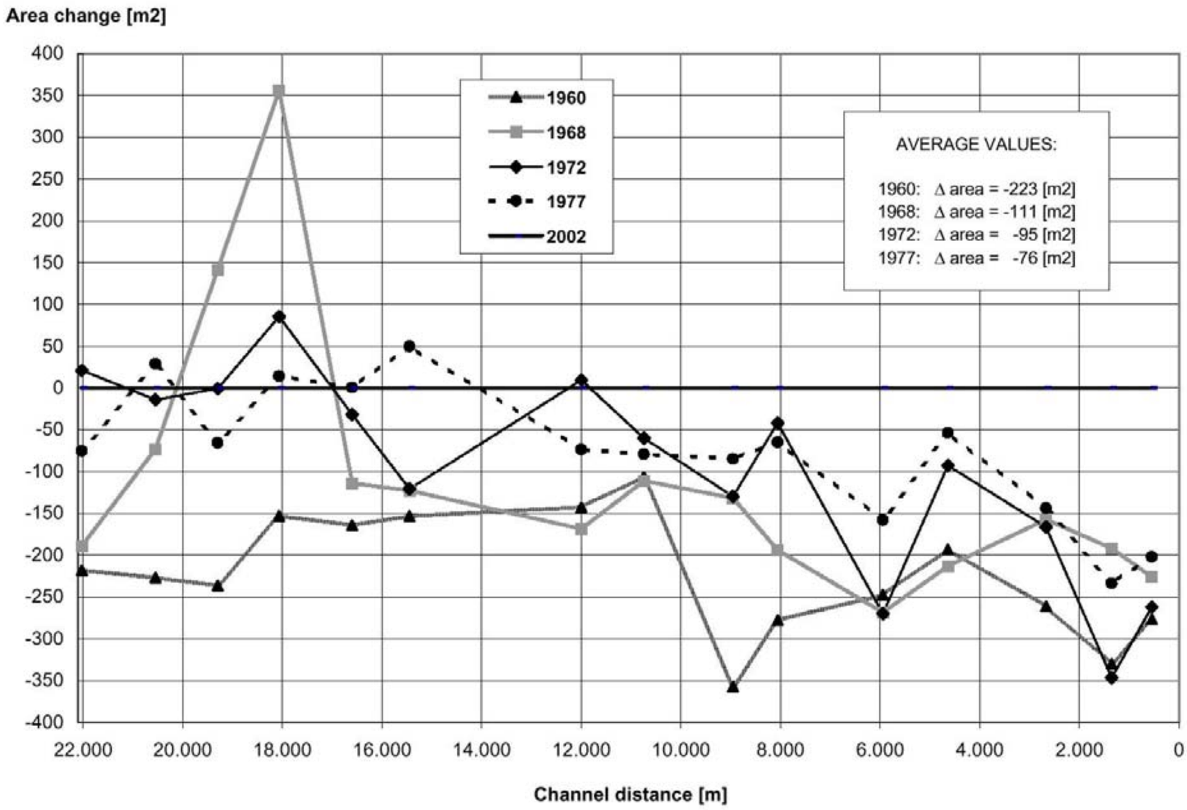


Figure 3::2. River bed changes of the river Drava in a period 1960-2002.

A state of the river bed in a year 2002 has taken as a reference for the analysis (Figure 3::2). Comparing that state with a state in the last 42-years, average increase of the cross-section profile for the whole distance is cca. 225 m^2 (average annual is 5,3 m^2). In a period from 1960 till 1968 it was recorded the maximum average annual river bed decrease 31,7 m^2. In a next period from 1968 till 1972 the decrease was 4,0 m^2. Further, in a period 1970-1977 the decrease was 3,8 m^2, and finally in a period 1977-2002 it was 3,0 m^2. That gives a conclusion that, although there exist permanent annual river depth increase on the observed section, river bed degradation trend is decreasing.

Possible causes for that changes are not presented in this paper, because it is difficult to estimate the influence of each cause separately:

- construction of several hydroelectric power plants on upstream sections, which made changes in water regime and sediment transport regime,
- construction several river regulation structures,
- regulation of the catchment area,
- commercial gravel and sand dredging.

The most important issue is that measured data shows gradually falling of river bed degradation process, which is positive cognition.

4. Conclusion

Measured diminution of minimal and average water levels of the river Sava has indicated the river bed degradation. Profile soundings have confirmed significant increase of the cross-section profiles beside constant river with. For 30 km long section of the river Sava, hydraulic analysis has shown influence of the river bed degradation during 20 years period on the water level decrease. That decrease for low water is 20 %. Trend in cross-section geometry changes is not evaluated, which should be made due to the possible technical actions for its subsidence. On the other hand, in the depositional zone of the river Drava bed degradation process is also evident. According to the carried analysis average cross-section area increase in the last 40 years is diminuting. This trend indicates subsidence of the bed degradation process and in the future dynamic stabilization of the river bed. Final realisation of these processes is possible only with maintenance of the current water level and sediment regime and also applying right technical actions on the river bed and catchment area.

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Mathematical Model of Water Work Drahovce - Madunice

Radomil Kveton, Peter Dusicka

Slovak University of Technology, Department of Hydraulic Engineering, Radlinskeho 11, 813 68 Bratislava, Slovakia, kveton@etirs.sk, dusicka@svf.stuba.sk

Abstract
The water work Drahovce - Madunice is the simplest part of Vah hydroelectric system from the point of view of mathematical modelling. 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.

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 (Kamensky a kol.,1998) 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 (Dusicka, Kveton, 2001). 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.

The presented paper describes the hydro dynamical model (HDM) of HPP Madunice as a channel power plant model with an upper boundary condition water reservoir and an down boundary condition water jump over a small stone dam at confluence of outlet channel from HPP and the river Vah.

Solution of the mathematical model HPP Madunice is based on results from hydro dynamical modelling of water level regime in channels HPPs controlled by company Vodne elektrarne Trencin.. The water level regime in the intake and the outlet channels (as non permanent appearance by start and termination of working channel HPPs) belongs to important control characteristics of the channel HPPs. It is not only important from the energetic point of view, but mainly from the point of view of safety.

The unsteady flow is characterized by the change over time of the flow and the change in the water level in the separate profiles of the open channel. In the gradual change of these quantities (shock-less waves), the physical substance can be expressed by means of a primary system of modified Saint-Venant partial differential equations as follows:

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} - q_{\ell} = 0$$

$$\frac{\partial(\beta QV)}{\partial x}+\frac{\partial Q}{\partial t}+gA\frac{\partial h}{\partial x}=gA(i_0-i_e)+q_\ell v_\ell$$

where

Q	water discharge [m^{3}s^{-1}]
A	flow area [m^3]
q_l	density of lateral side inflow or outflow [m^{3}s^{-1}]
x	length coordinate [m]
t	time [s]
V	average section velocity [ms^{-1}]
h	water level [m]
g	gravitation acceleration
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 [ms^{-1}]

The computer solution of the model of the water work (WW) Madunice – Drahovce was involved in the system of partial models solved by computer (Dusicka, Kveton, 2002). Together with the involving of described model in computer program, the system was extended with the following possibilities:

- boundary condition of the water reservoir, based on knowledge of the volume curve as function of the water level in the reservoir
- possibility to change the discharge through HPP by the manipulation on the installed hydraulic objects by setting time change of characteristic parameter

2 Hydrodynamic model of WW Drahovce – Madunice

The WW Drahovce Madunice was divided (from the point of view of modelling) to 2 sub-models :

- sub-model of the intake channel from the upper reservoir of HPP Madunice to Drahovce dam
- sub-model of the outlet channel from the confluence channel to the down reservoirs HPP Madunice

The cross sections (input topological data into model) were obtained from the project documentation of the WW. In comparison with documentation, the only difference found out was the upper level of the small stone dam at the confluence of the outlet channel and the river Vah. The measured level was 139,10 m a.s. in comparison with projected level 139,40 m a.s.. This fact was confirmed by the measure of the water level under HPP by the permanent non-working stage. The other changes are expected mainly in the intake channel due to the sediment accumulation from the reservoir Slnava. The volume curve of the reservoir Slnava (together with the outlet channel from HPP Horna Streda) and discharge curves through the installed hydraulic objects were taken over manipulation order of HPP. The measures on the WW Madunice – Drahovce were realised on 7-th September 2002. The discharge through HPP was controlled in order to the modified plan (see Fig.2). The different value of the discharge in experiment No.3 (the third peek in the discharge through HPP form 13:00 hour) between the measured discharge through HPP Madunice (only the discharge through turbines) in comparison with the discharge applied in model (total discharge through HPP) is due to the addition of the discharge through the navigation chamber.

2.1 HDM of outlet channel HPP Madunice

Sub-model outlet channel is running with the boundary conditions:

- down boundary condition of type Q(h) – the discharge over the small stone dam in the depending of the water level
- upper boundary condition of type Q(t) – the time manipulation of the discharge through HPP

The down boundary condition Q(h) was not measured previously by user of HPP, therefore it was prepared on base of the water level measures in experiments No.1 and No.2 and it was verified in experiment 3.

2.2 HDM intake channel of HPP Madunice

Sub-model of the intake channel is running with the following boundary conditions :

- upper boundary condition of type h(t) – time function of the water level on the dam Drahovce , calculated from the volume curve of the reservoir Slnava as a function of the previous water level, inflows into reservoir and outflows from reservoir
- down boundary condition of type Q(t) – time manipulation of the discharge through HPP

In the part of the dynamic computing of the upper boundary condition the inflow from HPP Horna Streda has decisive position. The inflow to the reservoir Slnava in the cross section of the dam Drahovce was added to computing with a time delay, which was set up due to experimental fulfil of requirement to equal measured water level in the dam Drahovce and the computed water level in the same cross section.

3 Comparison outputs from HDM with measures on WW

The comparisons of the outputs from the model and the measured values on the outlet channel are on Fig. 3. There are compared the two most significant water levels (the water levels on the start and at the end of modelled section). The modelled channel is quiet short (4,6 km) and the agreement was achieved very well. Due to the relative short discharge peaks (max. 1,5 hour) there were not observed significant changes in the outlet channel depending on the rising water level in the natural river-bed of the river Vah under the small stone dam. More complicated situation is by modelling of the intake channel where the stabilization of the water level needs longer time. The model achieved significantly better results for higher discharges (experiments No.1 and No.3). The biggest error was at the experiment No.2 where the experiment was started into an unsteady state with a relative small discharge (rolling reserve), which has minimal effect for the stabilisation of the water level.
On Fig.4 there are compared two results from modelling of the intake channel :

- time developing of the water level on the dam Drahovce – fulfilment of the upper boundary condition
- time developing of the water level on the HPP Madunice

The important using of the water wave regime is shown on Fig.5. The water wave (end of the experiment No.2) was used to clean trails of screens on the dam Drahovce .

4. Conclusions

The presented paper shows the possibilities of the water level regime models in channel HPP by energetic working. The integration of the natural river-bed data of the river Vah into the model would improve the quality of the model. After integration the model will be ready to use to model situations with raised discharges and flood discharges too.

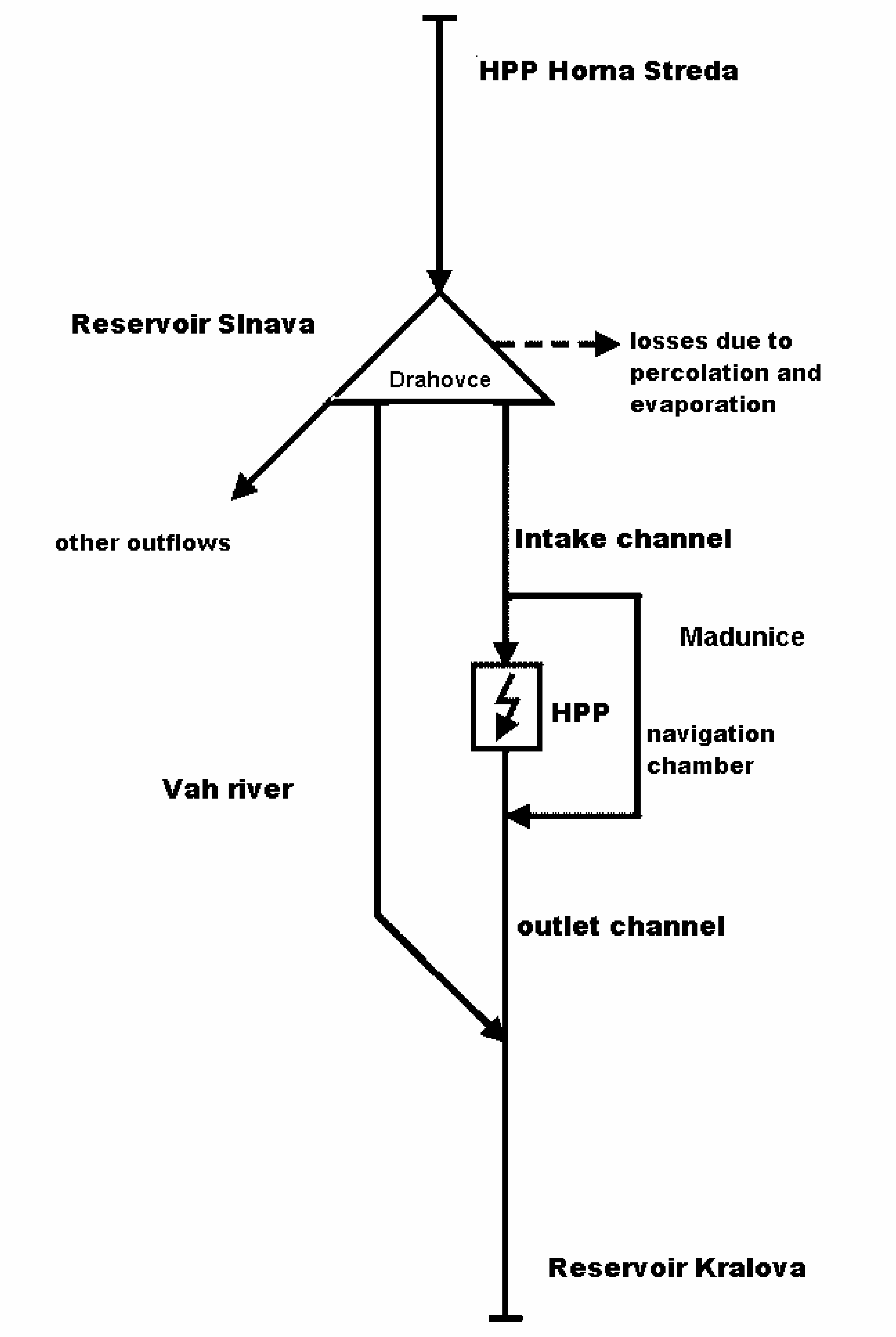

Fig.1 Scheme of water work Drahovce - Madunice

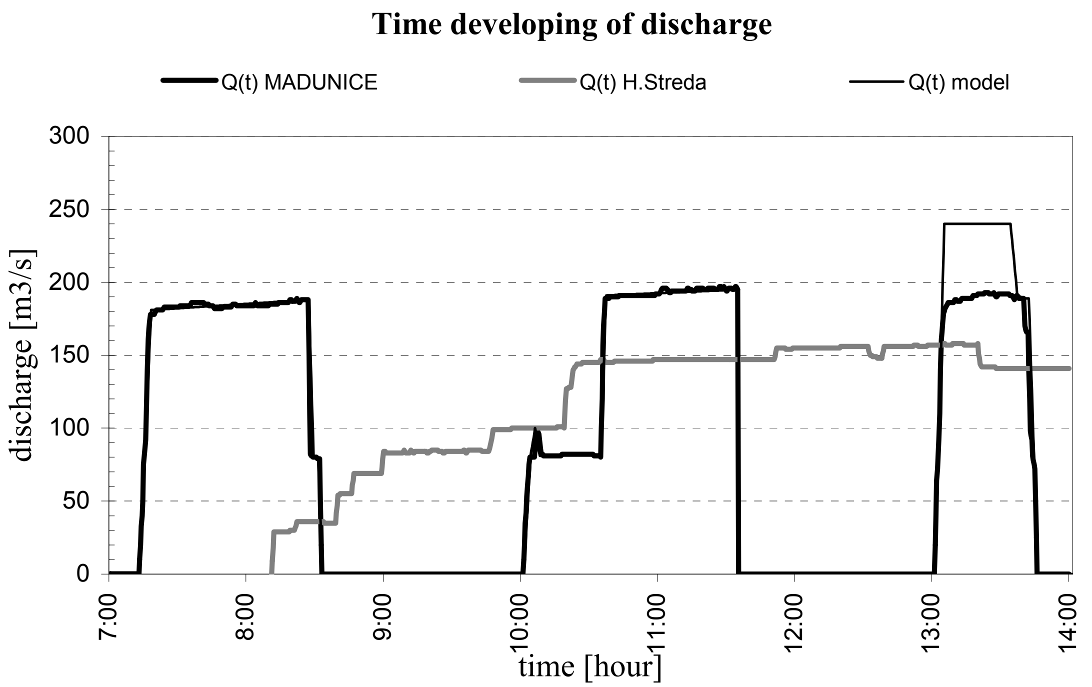

Fig. 2 Time manipulation of discharge HPP Madunice – experiments No.1 – No.3

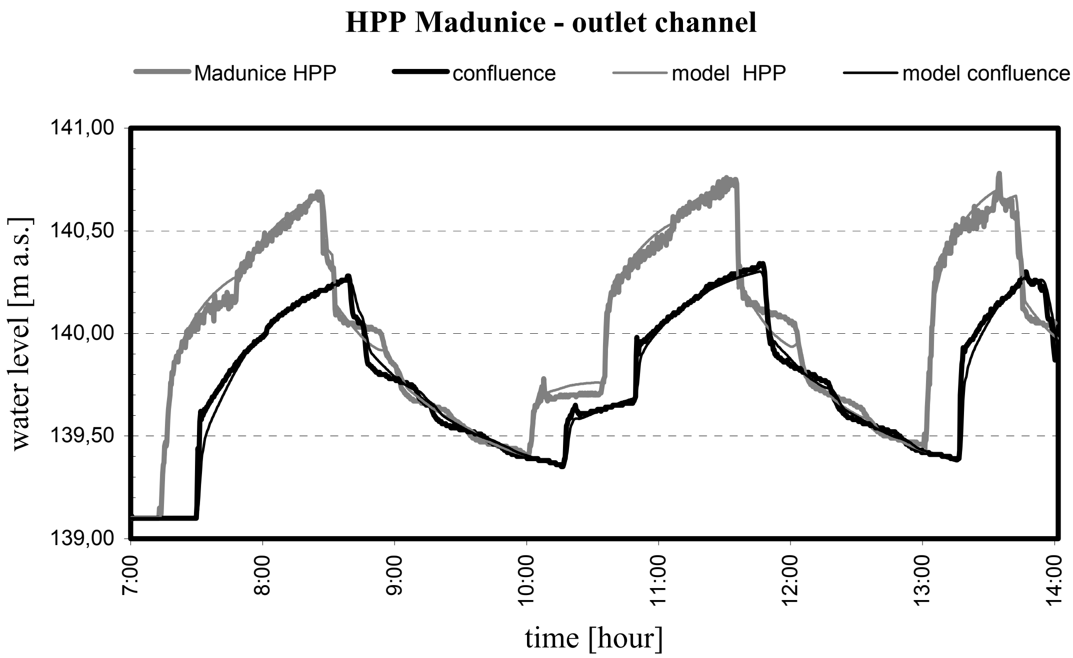

Fig. 3 Comparing water level measurements to results from model

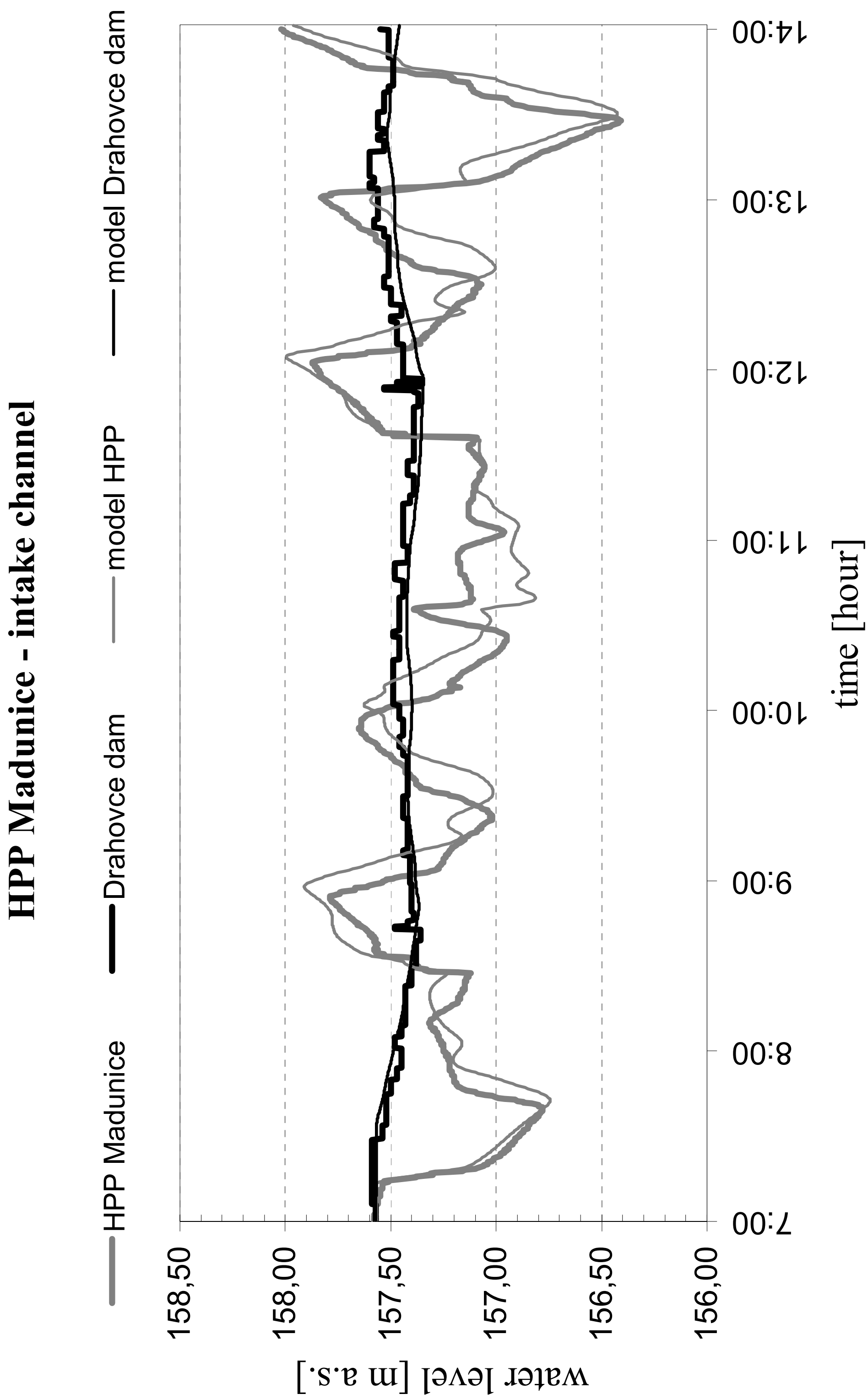

Fig. 4 Comparing water level measurements to results from model

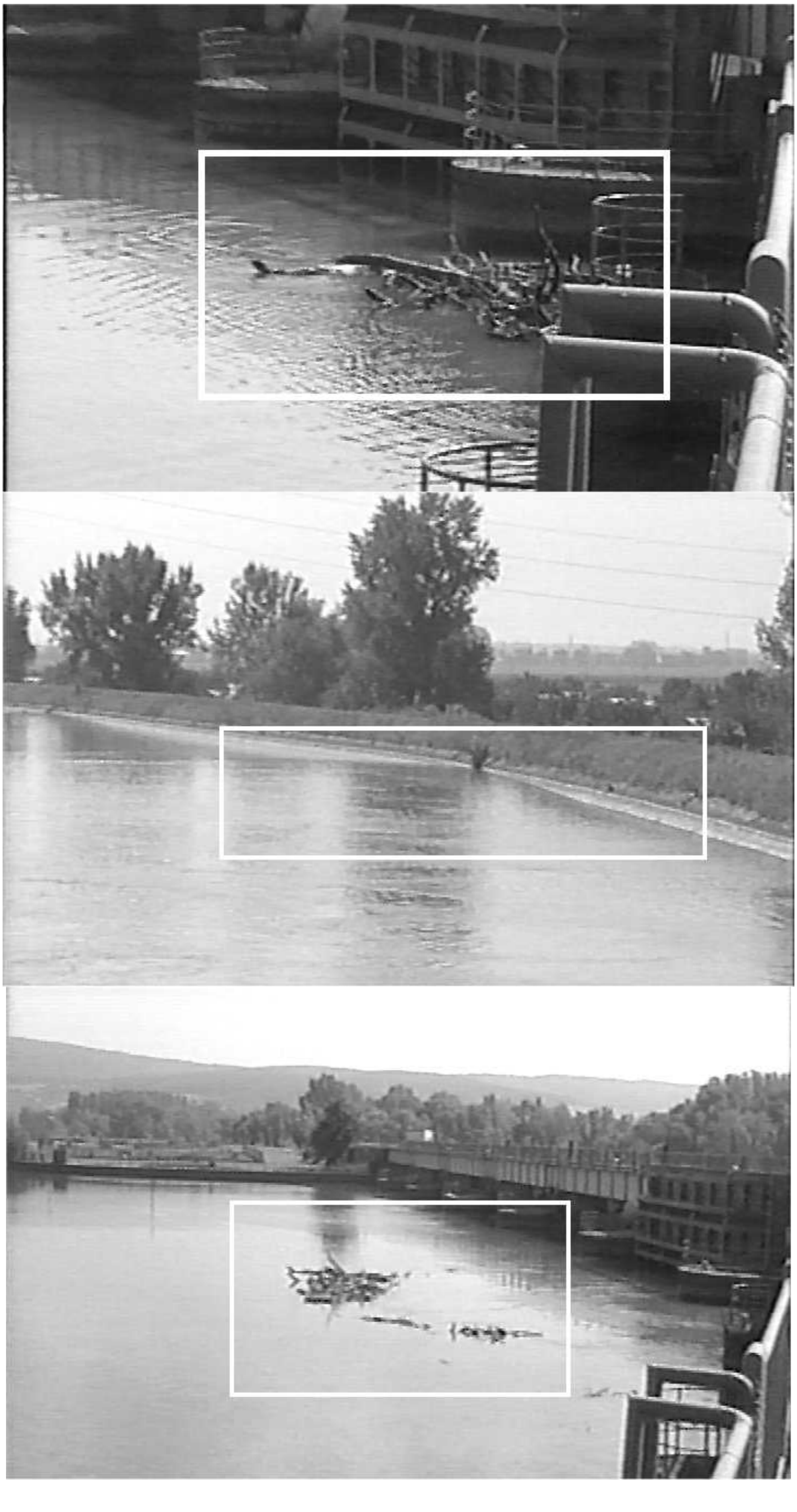

Fig.5 Cleaning trails of screens using water wave

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FACTORS INFLUENCING CHANGE OF DESIGNED PARAMETERS OF HYDRAULIC STRUCTURES IN SLOVAKIA

Prof. Michal Lukac, PhD., Slovak University of Technology, Slovakia
Eng. Miroslav Lukac, Water Research Institute, Slovakia

1. INTRODUCTION

Designed parameters are frequently considered unchanged in the real operation, despite relatively long service life of reservoirs, dams and other hydraulic structures. The specific natural conditions of individual sites, like geological, climatic or hydrological conditions are not adequately taken into account. These conditions influence change of designed parameters in time. This phenomenon, which is connected with the ageing of hydraulic structures, has to be monitored and accepted, based at intensity of process in the given conditions. Designed parameters and purposes of hydraulic structures have to be substituted with disposable ones. For instance, change of available storage volume of reservoir influences decrease of probability of water draw-offs.

The ageing of hydraulic structures has to be monitored adequately in the real operation, taking into account their long service life (sometimes several hundreds of years). The operational rules of individual reservoir have to be updated, based at the changes in the available parameters of reservoir or in the dam behavior, which result from the monitoring. Mutual interaction of hydraulic structure and environment plays an important role. The paper deals with analysis of some typical factors of hydraulic structure's ageing in Slovakia. The emphasis was put into individual components of environment, purpose, age and real-time behavior of hydraulic structures.

2. TYPOLOGY OF RESERVOIRS AND DAMS IN SLOVAKIA

Dams and reservoirs can be classified according to various criterii. From the viewpoint of age and significance for the society, Slovak reservoirs can be divided into three groups (see figure 1):

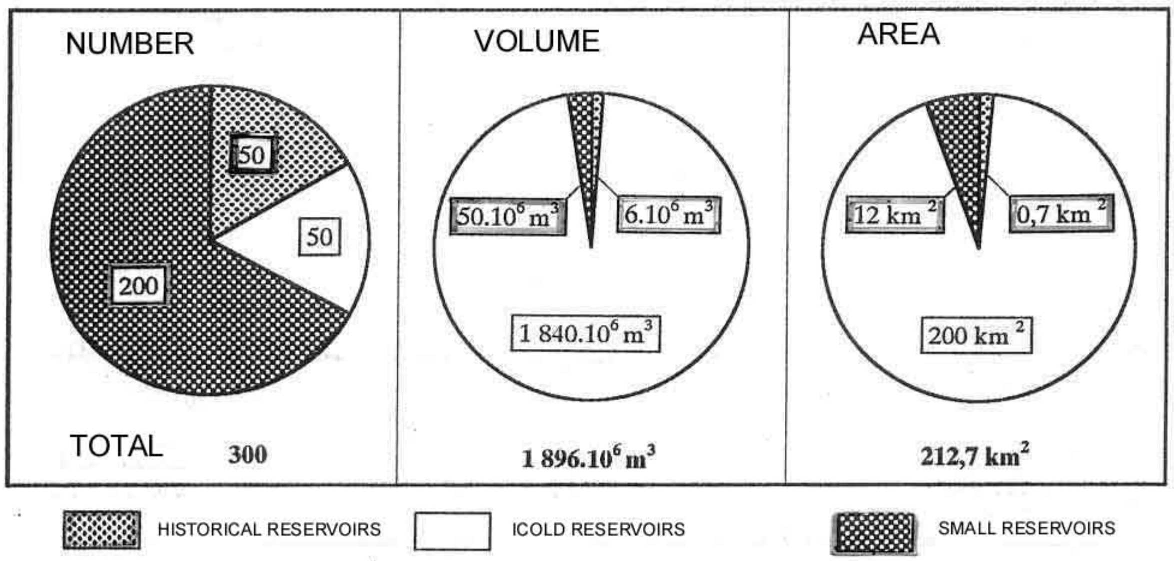

Fig. 1: Basic parameters of Slovak reservoirs

a) historical reservoirs from 16^{th} to 19^{th} centuries, the original number of which was 50,

b) small reservoirs of local importance, mainly constructed in the second half of the 20th century – total number more than 200,
c) reservoirs registered by the ICOLD (International Commission on Large Dams), the current number of which is 50.

Czechoslovak Republic was one of the ICOLD's founding members. Typology of reservoirs can be also based at their purpose. The basic water management functions of reservoirs are:

- retention function – decrease of flood wave peak and retention of water during flood,
- storage function – water accumulation and its balanced outflow, in order to cover demands of various consumers (navigation, power production, irrigation, water supply, etc.).

Slovak storage reservoirs serve 10 direct and 4 indirect purposes. From the viewpoint of flow control and regulation cycle, reservoirs can be classified as:

- reservoirs with over-year or yearly regulation cycle, which is typical for multipurpose ICOLD registered reservoirs and drinking water supply reservoirs,
- reservoirs with seasonal or short-term regulation, typical for power production reservoirs and small reservoirs of local importance.

Slovak reservoirs are sometimes grouped into cascades or systems of reservoirs. Extensive system of reservoirs started to be constructed in the beginning of 16th century. At the end of 19th century it consisted from around 50 reservoirs. Main purpose of this system was water supply of mining region in the central Slovakia. Individual reservoirs were interconnected with numerous canals, shafts and tunnels, which collected water also from the distant parts of the catchment, or even from the other catchments. This system of reservoirs represents unique and effective system of water utilization. It is situated in the surrounding of Banska Stiavnica, famous historical mining town (mining of gold, silver and copper). This region, together with its technical monuments was included in the list of World Cultural Herritage of UNESCO in 1993. Another historical water management technical monument – Turcek water main, has been constructed in another mining region (gold ores), close to the town of Kremnica. It represents first water transfer between two river basins in Slovakia.

Overall flow control conditions at the territory of Slovakia can be expressed with the coefficient of accumulation, which reads around 14 % (total volume of reservoirs – $1,896.10^9$ m^3, divided with the volume of long-term average run-off – $12,5.10^9$ m^3). The greatest volume ($1,84.10^9$ m^3) is, of course, accumulated in the large ICOLD registered reservoirs. With respect to the morphology of Slovak territory, reservoirs are either created with dams in the river valleys, or with weirs in the lowland regions. The largest Slovak reservoirs situated in the river valleys are – Liptovska Mara (total volume V_T=$360,5.10^6$ m^3), Orava (V_T=345.10^6 m^3), Vihorlat (V_T= 330.10^6 m^3), Velka Domasa (V_T=185.10^6 m^3), Starina (V_T=$59,8.10^6$ m^3), Nova Bystrica (V_T=35.10^6 m^3), Ruzin I (V_T=59.10^6 m^3). The largest reservoirs in the lowland regions of Slovakia are – Cunovo, Kralova and Drahovce, the total volume of which is around V_T=220.10^6 m^3. They are significant mainly from the viewpoints of power production and navigation (Danube and Vah rivers). The total retention volume of Slovak reservoirs is relatively small – around 240.10^6 m^3 (including Besa polder in the eastern Slovakia). This volume is even smaller than total "dead" volume of Slovak reservoirs (V_D= 310.10^6 m^3), which can not be utilized for water management purposes.

The total volume of 8 drinking water supply reservoirs is 167.10^6 m^3, which enables guaranted balanced outflow of 4,5 $m^3.s^{-1}$ and supply of around 800 000 citizens. Originally isolated reservoirs Hrinova, Malinec and Klenovec in the southern part of central Slovakia have been connected into extensive drinking water supply system. The largest drinking water supply reservoir is Starina with total volume V_T=59.10^6 m^3. Slovak large dams can be classified in 5 typological groups, from the viewpoint of their construction material:

- earthfill dams (TE), either homogenous or heterogenous – total number 26,
- rockfill dams (ER), either with natural (loamy) or artificial sealing elements – 8,
- concrete gravity dams (PG) – 5,
- composite dams (CM), combined from earthfill and concrete gravity dams – 10,
- weirs (BM) – 1.

Very interesting from the constructional viewpoint are 2 historical earthfill dams – Vodarenska (from 1510) and Rozgrund (1744, height 30 m, slim construction, see figure 2), the first Slovak rockfill dam Ruzin I (1968, height 64 m, highest Slovak dam, see figure 3) and two concrete gravity dams Orava (see figure 4b) and Nosice (see figure 4a). The Dobsina dam represents the first use of PVC foil (in combination with precast units) as a dam's sealing element, all over the world (year 1960) – see figure 5.

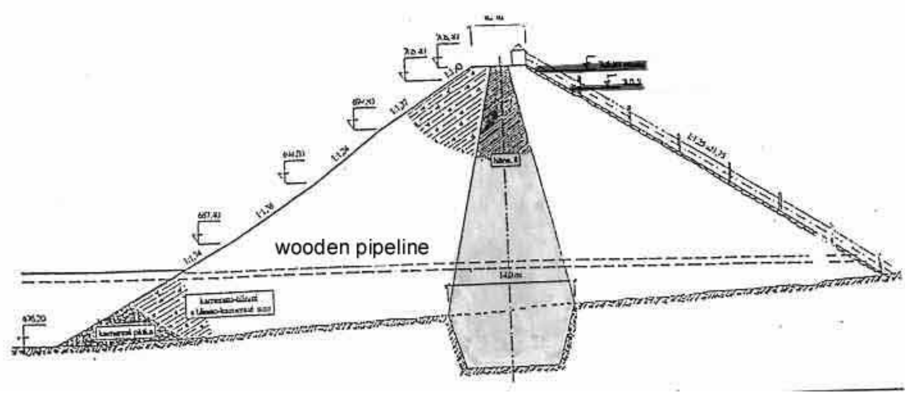

Fig. 2: Cross-section of the Rozgrund historical dam

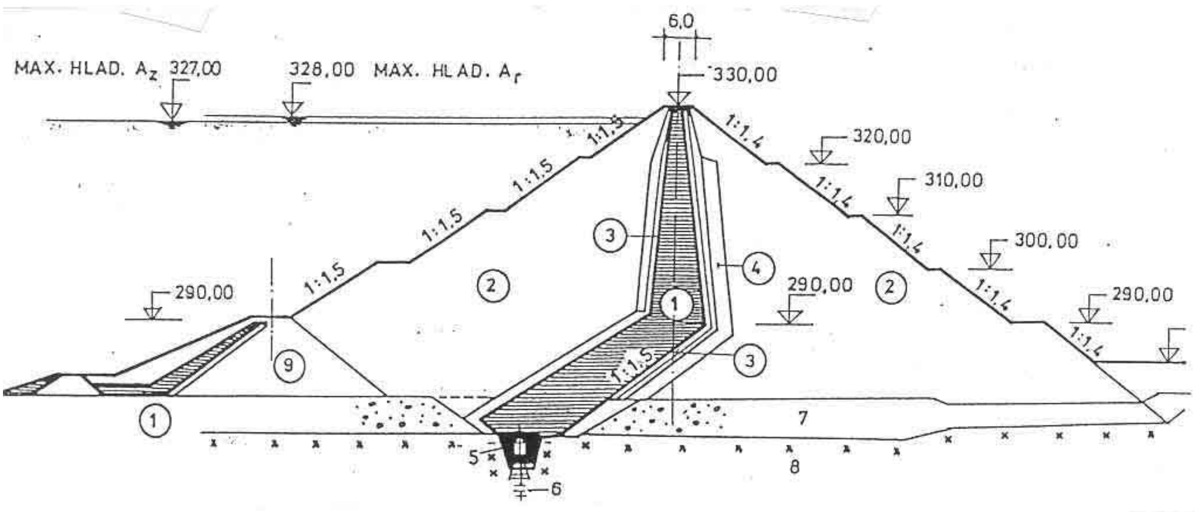

Fig. 3: Cross-section of the Ruzin I rockfill dam

Alternative materials (loam, asphaltic concrete, PVC foils with precast units) and technologies (grouting, cut-off walls, vibroflotation, etc.) for sealing of dykes, reservoir and navigation locks have been applied at the Cunovo – Gabcikovo complex of hydraulic structures, largest and the most important one at the Slovak reach of the Danube river. This waterwork, situated in the lowland area, with lateral power and navigation canal, is comparable to similar structures at the large rivers of Europe and the USA. The purpose of structure changed substantially during its construction. Because of controversary dispute with Hungary, the original scheme changed. The construction of Nagymaros balancing reservoir and connected objects was stopped and canceled. It also resulted in the decrease of power production and other effects (flood control, recreation, water sports, fishery, etc.) of the waterworks.

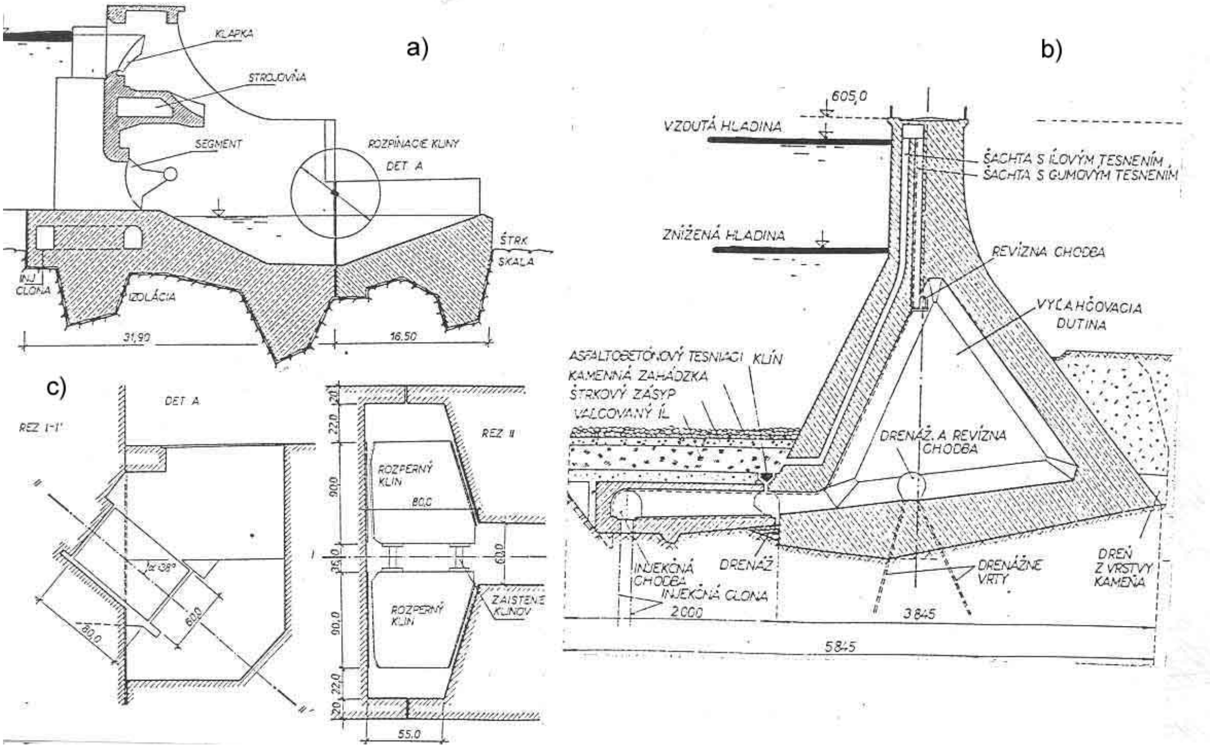

Fig. 4: a) cross-section of the Nosice dam, b) cross-section of the Orava dam, c) constructional detail of the Orava dam – bracing wedges

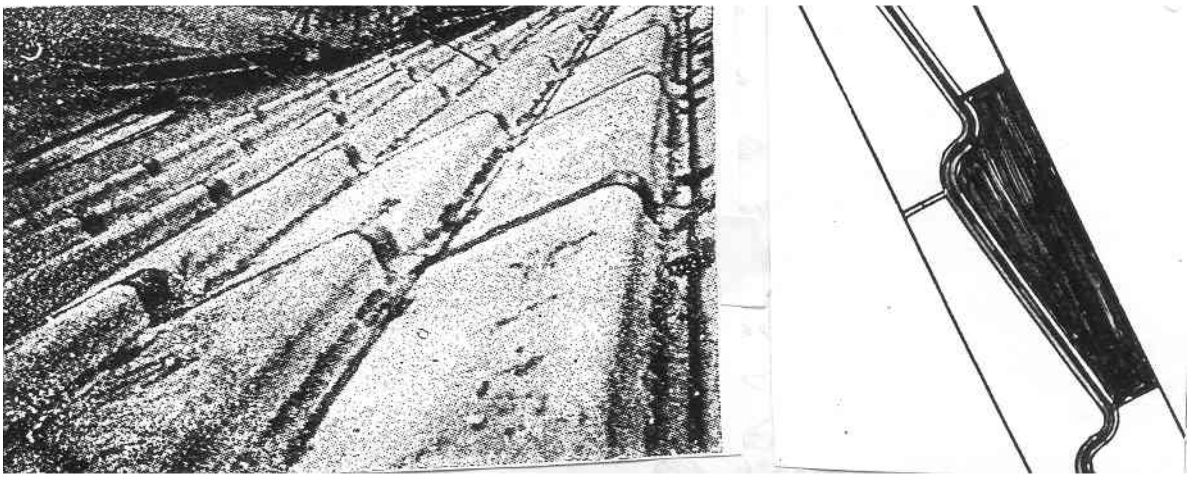

Fig. 5: Dobsina dam – view of PVC foil sealing during construction and scheme of the sealing – precast units+PVC foil

Individual purposes of multipurpose reservoirs can be antithetic from various viewpoints. It is frequently supposed that some indicators of dams behavior, like seepage, uplift pressure, deformations, piezometric levels, filtration velocities, will correspond to the model used in the project. Careful monitoring based at "in situ" measurements can indicate differences between model and real operation. Structural and hydrological safety of the waterwork influence its overall safety, which is very important in the real operation. There is natural mutual interaction between the dam and reservoir. It is evident, that a dam with anomalies in its behavior can influence designed purposes of the waterwork. The hydrological safety of a waterwork can decrease due to low reliability of design hydrological data (shape and peak discharge of design flood), undersizing of the reservoir retention volume, application of deterministic methods. Design flood can change in the service life of the waterwork, because of the changes in the design methods, in the level of flood protection (environmental and economic criteria). There is an effort to harmonize standards, guidelines and design methods for the design of spillways, bottom outlets and other objects of the dams in the ICOLD's member countries. The examples given in the next section illustrate influence of uncertain hydrological data, extraordinary cases and of anomalies in the dam behavior at the reliability of reservoirs operation.

3. ANALYSIS OF SOME FACTORS, WHICH INFLUENCE CHANGE OF DISPOSABLE PARAMETERS OF RESERVOIRS

The principal factors, which influence change of disposable volume of reservoirs are:

- processes of erosion and sedimentation, wave abrasion of reservoirs banks,
- extreme natural conditions (precipitation, run-off),
- uncertainity in the design hydrological data,
- anomalies and failures in the dam behavior.

Above mentioned factors we want to illustrate with selected examples from Slovakia.

3.1 Change of disposable volume of reservoir due to erosion-sedimentation processes

Despite relatively small area of the Slovak territory, geological conditions are manifold and complicated. The processes of erosion and sedimentation are intensive mainly in the regions of so called Carpathian flysh (multiple layers of sandstones and slates). Sedimentation of major Slovak reservoirs is monitored , based mainly at the field measurements performed by the Water Research Institute. The specific transport of sediments T at the selected reservoirs situated in the flysh regions reads approximately:

- Hricov reservoir: $T = 100\ m^3.km^{-2}.year^{-1}$,
- Nosice reservoir: $T = 300\ m^3.km^{-2}.year^{-1}$,
- Velka Domasa reservoir: $T = 450\ m^3.km^{-2}.year^{-1}$.

The Velka Domasa reservoir (catchment area 870 km^2) is situated at the Ondava river in the eastern Slovakia, in the flysh region. Its operation started in 1967. Sedimentation was the most intensive at the beginning ($T = 900\ m^3.km^{-2}.year^{-1}$ for the first 10 years of operation). The loss of reservoir volume supposed in the project was 3-times lower, comparing with the latest measurements (1996). The banks of this reservoir are also exposed to intensive wind waves abrasion, which contributes to the overall sedimentation, too. Large bars of the sediments, situated in the backwater area influence negatively flood control of the adjacent territory.

Another important flysh region is in the Vah river basin. A cascade of relatively small reservoirs (Krpelany – Hricov – Nosice) is situated here. The main purpose of this cascade is power production. The loss of reservoirs total volume represents 58 % for the Krpelany reservoir, 25 % for the Hricov reservoir and 22 % for the Nosice reservoir. Sediments in these shallow reservoirs have negative environmental and economic (decrease of power production) effects. Removal of sediments is complicated, expensive and not efficient.

3.2 Change of reservoir disposable volume caused with unreliable design data

Design hydrological data (peak discharge, volume and shape of design hydrograph) of lower reliability represent risk factor in the flow control during a floods. The examples of 2 Slovak large reservoirs – Orava and Vihorlat can be given in this respect. In the project of the Orava dam a design flood with the peak discharge of 1070 $m^3.s^{-1}$ was considered. Major floods occurred here already in the initial period of the reservoir operation – flood in 1958 with peak discharge of 2300 $m^3.s^{-1}$ and flood in 1970 with peak discharge of 1500 $m^3.s^{-1}$. It was decided to increase retention volume of the reservoir by 100 %, from $V_R = 20.10^6$ m^3 to $V_{R'} = 40.10^6$ m^3 in the period of summer months, which is typical for the occurrence of large floods here. The retention volume increased by the change of operational rules – lowering of the maximum water level of the storage volume during mentioned period. This measure helped significantly in the next period of operation.

Similar situation occurred at the Vihorlat reservoir. Its retention volume (100.10^6 m^3) is the largest in Slovakia. The peak discharge of design flood wave (see figure 6) was reached in the first 10 years of operation (1965-1974) and later exceeded several times by about 30 %. There were also another risk factors:

- repeated landslides of the intake canal slopes,
- frequent failures of the dykes protection,
- insufficient freeboard of reservoir dykes with respect to extreme wind waves, risk of overflow under certain circumstances,
- insufficient capacity of the canal downstream from the spillway,
- insufficient capacity of the control structure in Petrovce.

Extensive reconstruction of key elements of this waterwork is necessary, having in mind its extraordinary importance for the flood control in the whole lowland region of the eastern Slovakia.

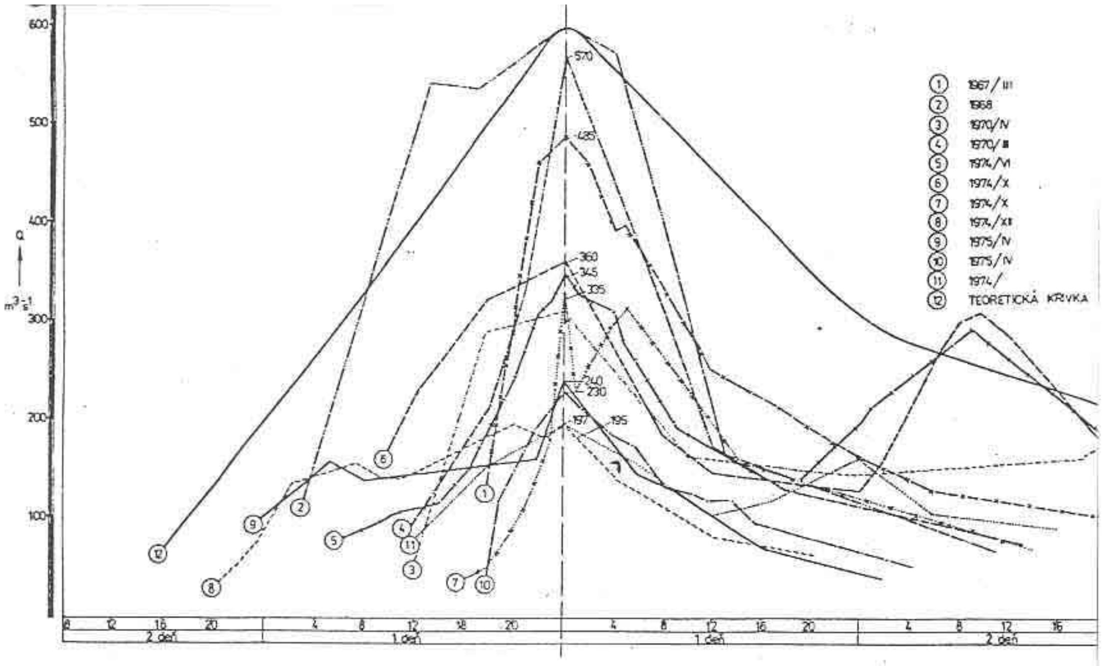

Fig. 6: Vihorlat reservoir – comparison of design flood hydrograph with observed ones in the period 1965-1974

Updating of the design hydrological parameters for the cascade (power production) of Ruzin I and Ruzin II dams, situated at the Hornad river, resulted in the construction of new additional spillways, which should help to pass safely larger floods. Updating was provoked by the change of design standards and guidelines. Similar example can be given from the Czech Republic. A study was prepared, how to increase capacity of the Sance dam (63 m high rockfill dam in the Odra river basin) spillway and bottom outlets.

3.3 Failures and anomalies in the dam behavior and their influence at the waterwork safety

The anomalies in the earthfill and rockfill dams behavior are the most frequently caused with excessive values of seepage, deformation, tension or pore pressure. Combined earthfill-rockfill dam Hrinova can be given as an example. This 51 m high dam was put into operation in 1965. In the period 1966-1971 anomalies occurred in the seepage regime (max. seepage around 100 $l.s^{-1}$) together with excessive local deformations of the dam crest and dam sealing core. Failures of drainage system and filters of loamy core required long-term decrease of reservoir water level by about 3-13 m (see figure 7). It also resulted in substantial drop of disposable storage volume from $7{,}6.10^6$ m^3 down to $2{,}5.10^6$ m^3. This fact significantly influenced drinking water supply in the water distribution system Hrinova – Lucenec – Filakovo – Velky Krtis in the southern part of central Slovakia. Careful long-term analysis of the reasons of failures resulted in the realization of remedial measures in 1995. Younger drinking water supply reservoirs in the region – Klenovec (1974) and Malinec (1994) helped to cover deficits in the water supply. Later, all 3 reservoirs were interconnected in a single water supply system.

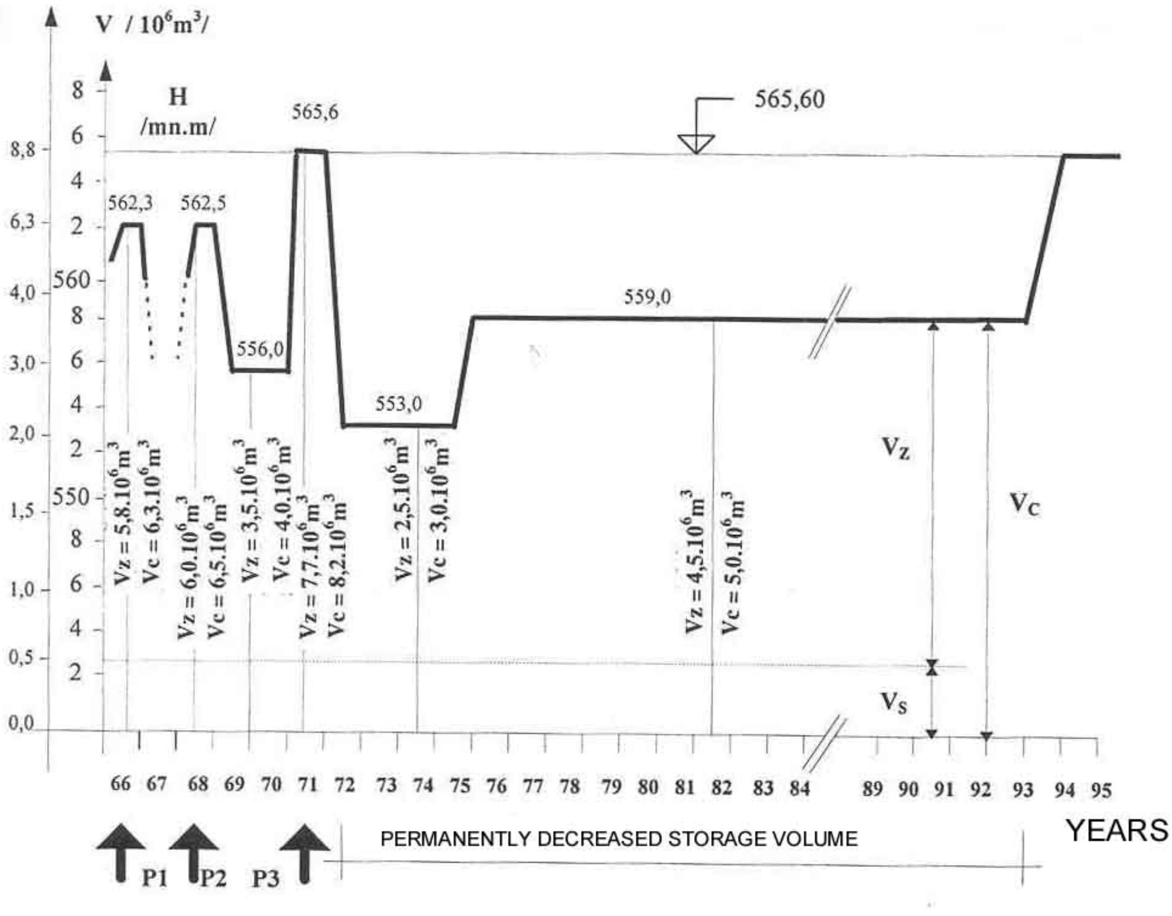

Fig. 7: Hrinova reservoir – schematized time history of the reservoir water level during the period of failures

3.4 Possibilities of reservoir outflow control using emptying of storage volume prior to floods

The management of floods is very sensitive to disposable parameters of reservoirs in the given time and place. The real-time control of reservoir operation during floods has to be based at as actual data as possible. Then, it is possible to apply alternative strategies of flow control. The utilization of storage volume in favor of flood control represents such a strategy. This is the case of already mentioned cascade of small reservoirs at the Vah river – Krpelany, Hricov and Nosice. In a normal situation, priority is put at the utilization of storage volume for the power production. These reservoirs have no retention volume, but their storage volume can be utilized if it is empty (or at least with enough decreased water level) in time. Such strategy requires good forecast and monitoring of flood situation, as well as cooperation of power engineers with water management engineers. The results of study performed at the Department of Geotechnics of the Slovak University of Technology, Faculty of Civil Engineering for the Nosice reservoir are briefly summarized in the table 1.

Table 1: Possible utilization of the Nosice reservoir during floods

flood	Q_{max} ($m^3.s^{-1}$)	Q_E ($m^3.s^{-1}$)	Q_R ($m^3.s^{-1}$)	T_E (h)	T_0 (h)	T_F (h)
design flood	1900	1049	1270	25	4	28
50-year flood	2480	1242	1765	22	8	25
100-year flood	2870	1367	2103	21	9	24

Explanations to table:
Q_{max} – peak discharge of given flood wave
Q_E – reservoir emptying outflow
Q_R – routed discharge
T_E – time of reservoir emptying
T_0 – time of empty reservoir
T_F – time of reservoir filling.

It resulted from the given study, that utilization of storage volume, emptied prior to the flood occurrence is the most effective for 50-years flood in the case of Nosice reservoir. Empty reservoir can contribute to flood wave routing rather significantly.

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Wastewater Collection, Treatment, and Disposal in Small Communities in Croatia

Davor Malus[a]
Gorana Ćosić - Flajsig[b]

[a]University of Zagreb, Faculty of Civil Engineering, Kačićeva 26, 10000 Zagreb, Croatia, malus@grad.hr
[b]Croatian Waters, Water Management Institute, Av. Grada Vukovara 220, 10000 Zagreb, Croatia, gcosic@voda.hr

Abstract
In Croatia there are 6579 settlements of which 6532 are with less than 2000 inhabitants and they make more than 40% of total population. Those small settlements, mostly without adequate wastewater collection, treatment, and disposal have adverse effects on the environmental situation in the country.
The strategic plan on state level doesn't exist and most of district and town plans propose classical sewerage and biological secondary treatment in settlements, or connection to the nearest big urban center. This kind of approach isn't economical or sustainable.
An integrated assessment approach is proposed which involves many key parameters in decision-making process, leading to highly sustainable solutions. Besides possibility to choose among different technological solutions of wastewater collection and treatment, other issues like nutrient reuse, energy production, and irrigation are analyzed. Strong sociological analysis is proposed with bottom-up approach, giving end users a chance to participate in decision-making process.
In order to achieve planned goals, a list of measures is proposed, involving technical, legal, economic, and social aspects. With proposed approach in decision making process, certain mistakes made by developed countries can be avoided, awareness about wastewater problems will improve at the user level, and on institutional level decision makers will get an aiding system, which will enable them to identify sustainable technologies in close collaboration with stakeholders.

1. Introduction

In Croatia there are 6579 settlements. Out of that number 189 settlements have between 2000 and 10000 inhabitants, and 6532 settlements have less than 2000 inhabitants, making more than 40% of total population. Only 148 biggest settlements have collecting systems built to some extent, including 104 settlements in range of 2000 – 10000 inhabitants.
Small, mostly rural settlements don't have proper sewerage, thereby endangering the health of their inhabitants and quality of underground and surface waters.
In order to achieve the "good quality" status as defined in the European Water Framework Directive, there is a considerable need for investment into wastewater collection, treatment, and discharge (WWCTD).
At present, the awareness of non-existence of sound wastewater disposal and its adverse effects on the environmental situation of the inhabitants of small settlements in most rural areas is low.
It is therefore necessary to raise this awareness, and to convey the message that a change is needed in order to improve the existing situation.

Problem of WWCTD of small settlements is not strategically solved on the state level. It is left over to design and construction firms under dominant influence of traditional centralized sewer systems. In the district and town WWCTD plans small settlements are usually connected to the urban gravity wastewater collection systems with long rising mains or have their own secondary WWTP.
However, when some plans try to introduce innovative solutions, many problems stand in the way of their acceptance. Reasons for that are different but can be summarized as:

- lack of knowledge about new technologies for wastewater collection, treatment, reuse and disposal,
- resistance of communal firms to everything that is out of their existing practice,
- lack of technical standards for implementation,
- lack of legislative framework for "new" solutions,
- lack of positive examples in the surrounding,
- skeptical attitude towards cultural tradition, mentality, and behavior of population in the field of communal affairs.

2. WWCTD strategy for small settlements

The main WWCTD objective is to achieve high level of sustainability. Sustainability is referred to as a mutual relation between economy and ecology. Economical sustainability is regarded as a basic condition for ecological sustainability, but the opposite also applies: a good ecology is a prerequisite for sustainable and healthy economic conditions.
In developed countries like the EU or the US, a centralized wastewater disposal system is often considered to be the state of the art (by authorities and engineers), and even best available technology (BAT). Such practice is possible only if legally defined ecological parameters (thresholds), and cost of technology are assumed in decision-making process.
Integrated assessment also includes aspects like reuse of nutrients in agriculture, reuse of wastewater, in particular for irrigation and reuse of energy, whereas potential new threats to the environment, e.g. caused by micro pollutants in the wastewater, are also considered. Such approach guarantees higher level of sustainability.
Taking into account such an integrated assessment, it must be emphasized that the wastewater disposal practices applied in the EU and the US should not generally serve as an example for other countries.
Therefore, the decision on possible WWCTD system is not the only issue, but rather the application of an integrated multi-criteria approach. Evaluation of only technical and technological elements is not enough, but also transfer of solutions from big towns to small settlements.
WWCTD systems must be optimized from the point of costs of construction, operation and maintenance, but other issues that would give them quality of sustainability are dominant, too. Such issues are:

- water reuse
- nutrient reuse,
- use of energy,
- on-site treatment
- reliable operation
- simple operation and maintenance
- sociological acceptability

Another major issue for sustainability attainment is bottom-up approach. It is necessary to involve all stakeholder groups in the decision making process. Such approach is unknown in practice, because the end users don't participate in decision-making process. Final decisions are imposed from higher levels of decision-making. Bottom-up approach demands highly informed and environmentally educated citizens, and guarantees efficient application of chosen technology of WWCTD.

2.1 Actions proposed for WWCTD plan implementation for small settlements

In this chapter a list of actions is proposed in turn to implement an efficient WWCTD system for small settlements:
Settlements have to be classified according to the:

- number of inhabitants i.e. equivalent population
- economic and social status
- topographic properties (hilly, flat)
- distance to the nearest already built WWCTD system
- kind of the final recipient
- geological and hydrogeological properties of the terrain
- climatologic properties
- identification of "agglomerations" [1,2]
- identification of settlements in the areas sensitive on nutrients[1,2]
- extent to which the existing WWCTD system has been built
- existence of surface, underground and waste water monitoring program.

A list of available collection, treatment, and disposal technologies must be prepared. That list will contain:

- conventional and alternative systems for water collection with basic technological, constructive, and economic properties
- conventional and alternative systems for waste water treatment and reuse with basic technological, constructive and economic properties
- wastewater disposal systems without close recipient (river, lake, sea), with basic technological, constructive and economic properties

After that, the most appropriate WWCTD technologies will be coupled with previously defined types of settlements through multi-criteria analysis.
Besides choosing appropriate wastewater treatment technology, it is also important to choose water collection system. Choosing of suitable wastewater collection system can be accomplished through the analysis of certain data like:

- population density (number of inhabitants/ha, number of inhabitants/km of streets)
- number of possible house connections
- number of demanded pump stations/ha
- installed power (kW/number of inhabitants) etc.

Optimally chosen collecting system and wastewater treatment system need legal and organizational prerequisites for full implementation. Application possibility of the existing legislative must be analyzed, and missing parts must be recommended.
The final solution must ensure:

- legal status of the WWCTD systems (public status is preferred)
- ownership (private, public, mixed)
- technical standards in planning, design and construction

- legal procedure in issuing discharge permits for point and nonpoint discharging
- legal procedure in issuing construction permit, supervision, technical and financial support, repressive and stimulating measures.

Those solutions must be in concordance with EU and US practice.
All proposed measures should pass thorough scientific, professional, and public debate, after which various activities follow:

- legislative changes
- introduction of new technical standards
- field investigation instructions
- instructions for construction, operation and maintenance.

All technical aspects can be gathered in a software package, which will provide help with choosing proper WWCTD system according to the location-specific data set.
Whole plan should end with at lest three pilot projects for three typical settlements. That pilot project should be used as a test for the whole project and a feedback for possible changes.
Mentioned pilot projects should also serve for introduction and testing of bottom-up approach. Local citizens should be informed, educated, and interviewed for final decision on proposed WWCTD system.

2.1.1 Available technologies for WWCTD:

This chapter presents short review of available technologies for WWCTD in small settlements. An international source book recently published by the IWA on environmentally sound technologies for sustainable wastewater and storm water treatment[3] can serve as a good guide.
Technical (conventional) systems include aerobic systems, e.g. conventional activated sludge systems, sequencing batch reactor, trickling filter, biofilm reactors, membrane bioreactor, and anaerobic systems, e.g. the UASB and EGSB reactors. Natural (alternative) technologies usually encompass constructed wetlands and waste stabilization ponds. Constructed wetland technology is a promising solution to the problem of wastewater treatment, especially in rural areas. In general, the use of constructed wetlands has proven to be a cost-effective and technically feasible approach to wastewater treatment. In addition to this classification, ecological sanitation systems have been established as their own concept: Based on the principles of separating the wastewater into its constituent parts (yellow water, brown water, gray water) and reuse of water, nutrients and energy, it encompasses both technical and natural treatment technologies. Furthermore, it includes composting technologies for the brown water fraction, and needs special toilets. Also, various technologies are available for the collection of the wastewater, ranging from cesspools and settled sewerage systems to vacuum technology.
Since in some parts of the Croatia (Adriatic coast with islands) water is a limited resource, wastewater reuse for irrigation must also be taken into account. Moreover, since often in rural areas no water supply systems exist yet, implementation of water saving technologies for wastewater disposal in turn results in lower design flows for the water supply system and thus contributes to the principle of saving natural resources. General constraints in treated wastewater irrigation focus on the fate of excess nutrients in the environment (e.g. in the non-growing season), the fate of pathogens, problems related to soil salinization and the fate of micro-pollutants. In this respect it must be noted that in some countries the common practices already overruled the constraints, and even raw sewage is used by farmers and individuals to meet their water and nutrient needs. Therefore, it is recognized that there is a strong need to

develop cost-effective and safe treatment technologies for the reclamation of domestic wastewater. The treatment systems to be developed can be complemented with appropriate agricultural engineering in order to address the constraints mentioned above. Use of treated sewage in agriculture needs an interdisciplinary approach, which has not yet been developed to a satisfactory extent and includes sanitary, civil, and environmental engineering, irrigation and water management, agricultural sciences, as well as social sciences and economics.

2.1.2 Sociological issues

An important aspect of the elaborated approach is an integration of social analysis throughout the whole decision-making process. Social analysis and assessment is often viewed as a formality, and usually funded separately from environmental and scientific research, and by donors such as the World Bank. Such externally commissioned and funded research is often not taken into account in scientific research interpretation and application. Integrated, systemic approach advocated here will strengthen the research culture and directly introduce social analysis.

Similarly, scientific and environmental research in our country has tended to be conducted in a top-down way, without consultation about rural people's needs and perceptions. Taking a participatory approach, and emphasizing the methodological component of the project, it will contribute to a more demand-led research culture with more potential for relevant outputs and uptake of results.

Alternative WWCTD systems sometimes demand very active and entirely different engagement of end-user and communal company, compared with classical ones. Technology of wastewater separation is a good example for this. Without close cooperation in operation and maintenance those WWCTD systems have no chance of successful implementation. Those are the reasons why social component in the whole plan is so important.

Average Croat thinks that WWTP is sophisticated high-tech machinery that is able to treat any kind of waste without their participation. End-users need only to pay a fee, and all other activities are up to the communal company.

Traditional communal companies have a rigid image about their function in WWCTD. Special procedures in operation and maintenance and cooperation with end-users are unknown. The towns own all communal companies, which is the reason of resistance to any changes in the scope of their work.

2.1.3 Legal issues

Some settlements are far from water bodies as potential recipients, and some wastewater treatment technologies use ground for treatment and final disposal. In both cases wastewater is discharged to underground with potential underground water impairment. Croatian legislative doesn't cover those cases of final disposal of treated or untreated wastewaters. Underground discharge is forbidden or allowed only for first class water equivalent[4]. That way, the possibility of certain wastewater treatment and disposal technologies is restricted. Obviously, existing legislative needs changes and improvements.

Some WWCTD systems are completely autonomous (one household – one system), and some systems comprise few households or entire settlements. Each WWCTD system should be public and sanitary safe with possible communal supervision. It means that access for officials to all parts of WWCTD system should be ensured. Therefore, to accomplish mentioned demands some issues should be legally covered:

- property of land on which elements of WWCTD systems are built (infiltration fields, cesspools, compact treatment plants etc.)

- operation and maintenance costs jurisdiction
- legal responsibilities of WWCTD system users

3. Conclusion

Proposed plan for WWCTD of small settlements in the Republic of Croatia advocates integrated assessment approach with accent on bottom-up component. Such approach will ensure high level of sustainability that is the key goal of the plan.
The contribution of the elaborated approach to sustainable development will be:

- At the user level, raising awareness about wastewater problems; understanding of local perceptions and development of relevant adaptable solutions, incorporating the values, beliefs, etc. of the users in the definition of sustainability.
- At the institutional level, it will provide consultants, water authorities, etc. with a decision aiding system, which will enable them to identify sustainable technologies in close collaboration with stakeholders. It will also provide national decision makers (National water authority, relevant Ministries, etc.) with recommendations and guidelines to adapt the decision making framework in order to allow more sustainable technologies to be chosen; and it will provide decision support to adapt national policies and to incorporate environmental principles of EU policies.

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Water Levels as the Basic Indicator of the Kopački Rit Hydrology

Siniša Maričić[1], Josip Petraš[2], Silvio Brezak[3]

[1] Faculty of Civil Engineering, University of Osijek, Drinska 16a, 31 000 Osijek, Croatia, smaricic@most.gfos.hr
[2] Faculty of Civil Engineering, University of Zagreb, Kačićeva 26, 10 000 Zagreb, Croatia, jpetras@master.grad.hr
[3] Croatian Waters, Zagreb - Water Management Department for Drava and Danube River Basin, Osijek, Splavarska 2a, 31 000 Osijek, Croatia, sbrezak@voda.hr

Abstract

The Kopački rit nature park is a wetland under the government protection. It is located at the Drava mouth into the Danube, bordered by the waterbeds of both rivers and flood embankment in the Northwest. The preservation of the park nominated by the UNESCO for the world natural heritage is of world significance, and the Ramsar convention listed it into world - protected areas. It is also included into IBA - Important Bird Area List.
The wetland wilderness and dangers from mines and explosives left from the recent war in Croatia have limited the freedom of movement in the area.
The wetland is rich in biological diversities, and thus different scientists have investigated it over a time. The studies have shown that some acute changes in biocenosis have recently taken place, and there are some alterations in the terrain morphology as well. Today's research focuses on the origins of such changes. The major cause of the change in the biocenosis might be climatic conditions that affect the hydrologic regime. Recently undertaken investigations should define such regime and its variations.
The paper deals with the outlines of the area hydrology. It gives the water level measurements on the locations of the Drava, Danube and the central area, which is Kopačko jezero (lake). The paper shows the results of measurements and preliminary hydrologic analysis. The basic relations of water levels and water flow have been stated as well. The Danube waters are the main water supplier of Kopački rit. The management of Kopački rit water regime can mainly rely upon the management of inlet and outlet of water quantities from Danube. The managing of the in- and out delivery of the Danube waters could direct the managing of the water regime of Kopački rit. The investigation of water movement in this area is planned to be undertaken by mathematical modelling.

1 Introduction

In September 2000 European Parliament has passed the base document defining framework of European Union's water policy - Water Framework Directive (WFD). It comprises some 30 directives directly or indirectly related to water management and protection. That document is mandatory for all EU members, and it was adopted by Croatia, as well. Recent change of man's attitude toward the nature is manifest in the fact that in the WFD directives natural watercourses, seas, lakes, wetlands and subterranean waters are regarded as water environment, i.e. ecosystem in which his mainland part is included, too, in direct dependence on water. So, as opposed to recent practice, WFD includes significantly emphasized environment protection in its management goals.

National Strategy And Action Plan of Biological And Landscape Diversity Protection was adopted in the Republic of Croatia in 1999. This document points out that protection and improvement of existing biodiversity is a state goal and that necessary measures will be legislatively incorporated into all economical activities using biological resources.

Diversity protection is based on the Nature Protection Law with an emphasis on special protection of particular areas and species. Protected areas are the axis of general protection and make crucial ecological network nodes, i.e. represent natural reserves of biodiversity. More than one tenth of mainland territory is protected in Croatia. The law envisages eight categories of spatial protection, out of which two most important are: *national park* category and *natural reserves* category. Croatia has eight national parks and ten natural reserves. Those are areas of national or international value whose protection falls within state authority. [1]

One of the natural reserves is Kopački rit. It is a wetland, i.e. inundation area with wetland characteristic. It is in the north-east of Croatia, close to the Drava rivermouth into the Danube. Its borders are riverbeds of those rivers and a flood protection embankment from the northwest. The area is abundant with water of subterranean and surface inlet, makes it suitable for life of numerous plant and animal species.

Water appearance in inundation flood areas of rivers is undoubtedly the main drive behind natural processes and establishment of ecosystems peculiar to such areas. Therefore the defining of biodiversity protection procedures primarily depends on the knowledge of their hydrological regimes, i.e. those procedures shall be defined on the strength of research and study of their hydrological elements. Hydrological regime manifests the dynamics and stochastics of water condition changes as well as hydrological balance of wetland ecosystems. In such areas, all other ecological, water management and wider economical investigations can be based only on such and similar indicators, and analyses carried out which are indispensable for organization and conducting of water management on the basis of sustainable water management. [2]

Regarding those facts, the Faculty of Civil Engineering in Osijek in 2001 initiated and began basic hydrological measurements in Kopački rit. As of now, only one water gage has been placed there with continuous digital water level recording in largest surface reservoir of this area - Kopačko jezero (lake). Setting of that instrument coincided with the restoration of modern water level monitoring for the Drava and Danube rivers, which was interrupted during the war in the early nineties of the last century in Croatia. Gathered data right now enable merely a preliminary review of water regime in the Kopački rit area. This is what this paper is focusing on.

2 Kopački rit – the description of area and its problems

Today's Kopački Rit area was created during a long geological morphogenesis period of the Danube and Drava rivers, in a wide flood zone (approx. 50.000 ha) between those flows.

By their natural values (forest ecosystems of flood flatlands, wetlands and lakes), the right

Danube bank area from 1383 to 1410 river km and left Drava area particularly stand out, comprised within the Kopački rit - natural reserve (approx. 23.000 ha). An area of approx. 7000 ha has been set aside within that region in a capacity of special zoological reserve with special protection regime. Kopački rit represents one of the best preserved naturally flooded areas in Europe and within its boundaries more than 2000 plant and animal species have been registered so far. Its preservation is of a global significance. UNESCO has nominated this area as a World Heritage Site, and according to Ramsar convention it was listed among globally protected areas. It also entered IBA (Important Bird Area) list. [3]
Natural flood conditions of this flood area were artificially altered during previous centuries by the construction of flood protection embankments and melioration systems for agricultural lands drainage, but by other antropogenic (fishponds, riverbed regulations) impacts, as well. That directly or indirectly caused changes in previous natural life conditions in that area. [4]
Dominant structures of this area are the flood protection embankment (Kopačevo – Zmajevac), which in the northwest separates Kopački rit from adjacent Baranja agricultural areas, and pump stations built for surplus water drainage from agricultural areas.

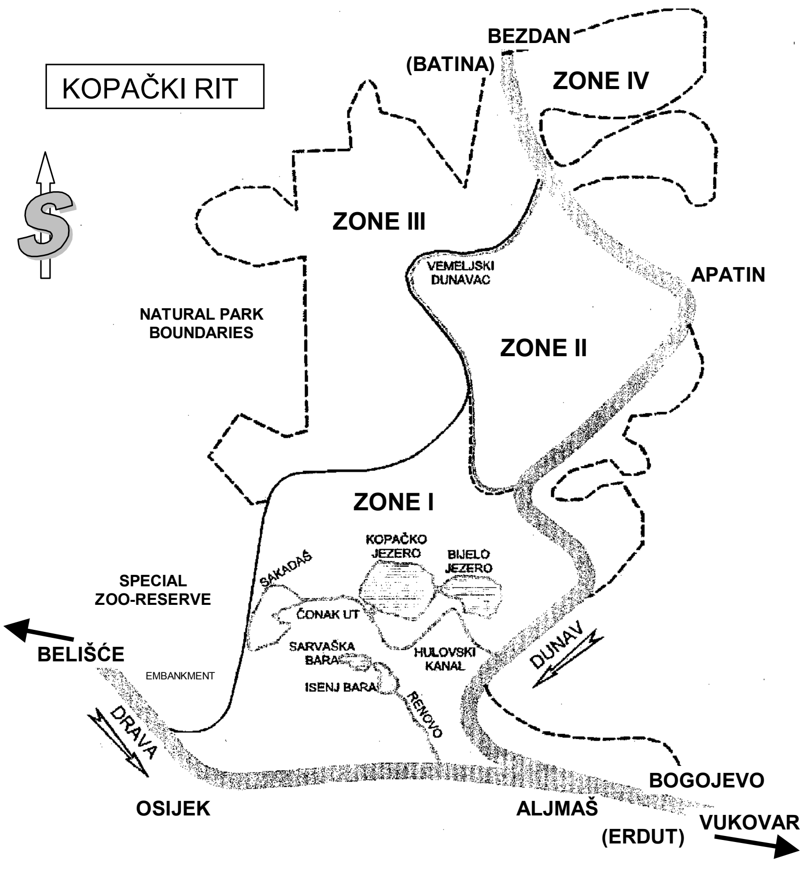


Figure 1: Sketch of theKopački rit area with an indicator of main guidlines

During the last war some structures were destroyed in the Kopački rit, while wildlife was hunted and forests cut in an uncontrolled way. After the war, a pollution problem by mines and other explosive means was left in its wake. Demining is hampered by yearly flooding and water receding, fast vegetation growth and morphological terrain changes, caused by soil erosion and alluvium deposition. [5]
Investigations in this area are hampered by natural conditions of wetland wilderness, danger

from mines and other explosive means, as well as current unfavourable economical conditions.
Just as elsewhere, even here the ecological conditions of the area depend on natural variability of meteorological-hydrological parameters in time. General climatic changes primarily affect hydrological regime changes here, and by extension main characteristics of the system, as well.

3 Morphological-topographic terms of reference for the hidrology of the area

The Danube river is the most influential element of this area. Its riverbed here is wide, depth median and gradient mild. Numerous remnants of meander forms are present, adapting themselves to natural courses.
According to specific qualities of hydrographic elements and water regime in Kopački Rit Natural Reserve, four zones can be discerned. They are marked on *Figure 1*, where main water surfaces are shown, too. The distinctions between first two zones were emphasized by Tadić and his collaborators – [6]. Analyses of the Kopački Rit water conditions conducted so far yielded following data – [7], [8]:

- a weir overflow was constructed on the Hulovski canal with the top level at 79.5 m above sea level (m asl), and under that level there is no surface flow;
- overflow from canals, ponds and lakes usually occurs above the 81.5 m asl, with main inlet through the Hulovski canal
- above the 82.0 m asl, the filling begins (usually of the second zone) from Vemeljski Dunavac, with the development of inner watercourse network;
- at the 82.5 m asl, natural terrain weirs are overflowed and the flow from the northern (II) section into the southwern (I) one begins, primarily through the Nađhat canal;
- at the water level of 83.0 m asl, the major part of Kopački rit is under flood, and the outlet beginning with Renovski canal spreads over entire contact area with the Drava.

	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

Table I: Basic data about hydrological stations of wider Kopački rit area

4 Comparison of water levels in the wider Kopački rit area

Water levels are a primary hydrological indicator the flows are detremined from by the Q-H

connection. As the Q-H connection has not been established for any spot in the Kopački rit, the hydrological analyses of that area can so far relate only to water level analyses.
Since the Kopački rit is in a flatland region, within the flow area of two large rivers, monitoring of river flows was established in its surroundings a long time ago. Seven measurement stations are of interest for Kopački rit on the Danube, and three on the Drava. Some of them are equipped with a limnigraph, and some only with a water level lath. Some are on the territory of other countries (Hungary, Serbia), and some of them have been in operation only for a shorter period and/or with interruptions. Basic data about meaurement stations are shown on *Table I*, and *Table II* shows values for charcteristic water levels and flows obtained at those measurement stations.
According to what has been said in this introduction, the first water gage with a continuous digital recording of water level was set in the largest surface water reservoir of this area – Kopačko jezero (lake) – as recently as March 2001. A pressure-piezzometric type instrument was set up during low water levels, by drilling a round hole in the lake bed soil and placing in it a perforated metal pipe, protected by a fine sieve, into which a pressure indicator (sensor) of water level was installed. The sensor was set at a depth below estimated minimum possible water levels in the lake. Automatic datalogger was placed in a locked box, which is attached to a nearby tree, high enough to avoid flooding during high water occurrences in the lake.

	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	-

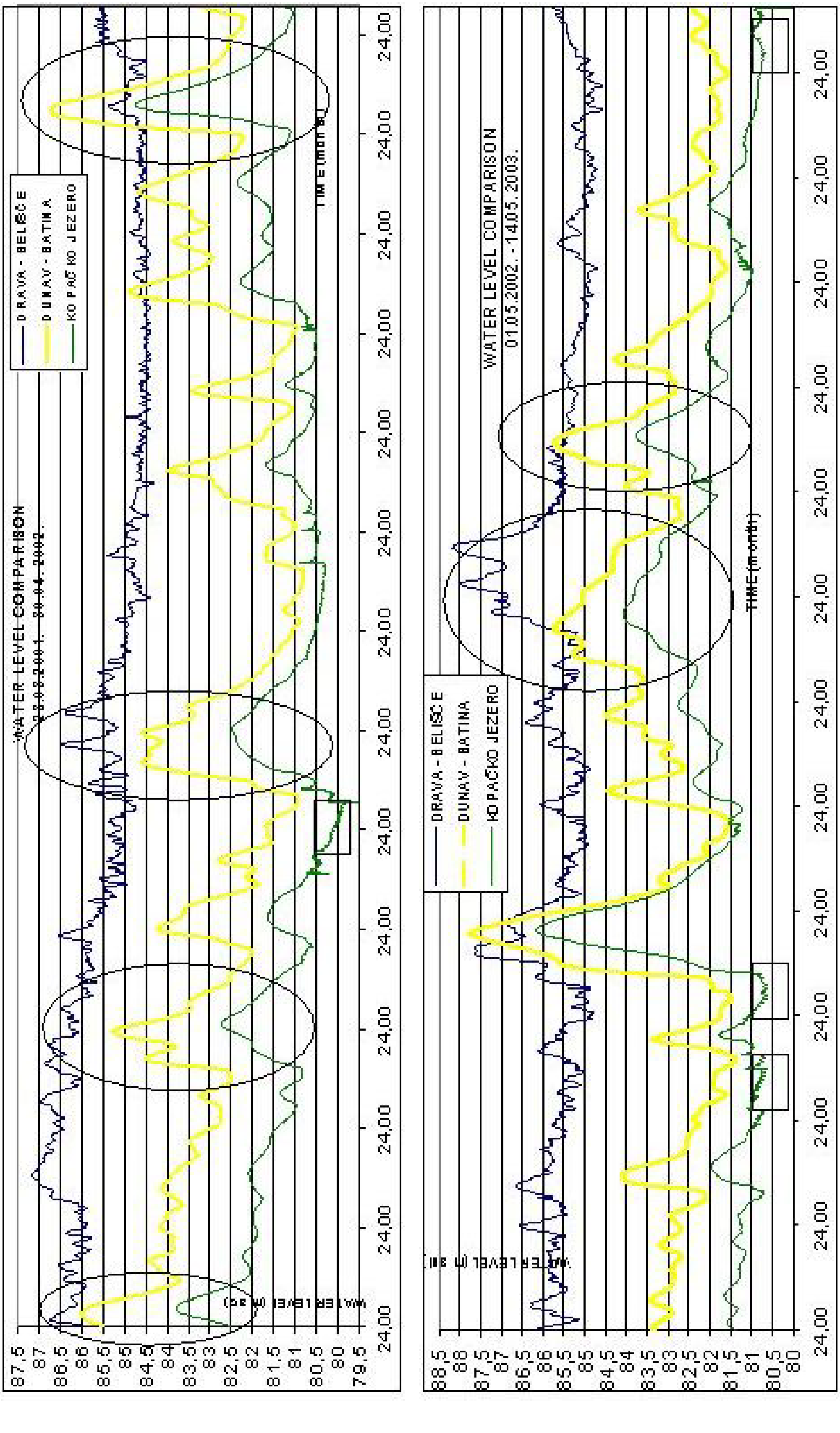
** - last 50 years. and two-year monitoring* *** - thirty-year course (1961.-1990.)*

Table II: Characteristic water levels of observed hydrological measurements

Instrument logs water level in the lake during the submersion of perforated pipe top, while in cases of low water levels it logs subterranean water level at the bank immediately by the lake. Data are saved into the memory at an hourly rate.
Available data for this area of the Danube catchment mainly reach back to the mid-twentieth century, which means to the point when climatic and overall condition changes due to intensified human activities were first observed. It is therefore safe to assume that gathered data reflect complete caused processes. [7]
On the basis of totally available data fund on the Danube and Drava water levels and flows, hydrological analyses were conducted and they showed there was a trend of water level changes, i.e. a trend of fall for all water level categories (maximum, minimum and mean annual water levels) was observed. By and large, the biggest fall was observed by mean water levels (for the Danube it amounts to approximately 1 cm annually, and for the Drava even more than that), and the smallest ones by minimum annual water levels. [9]

Furthermore, a preliminary hydrological analysis of the water levels, taken at the same time in the 2001-2003 period on Kopačko jezero (lake), the Danube (near Batina and Vukovar) and the Drava (near Belišće and Osijek) was conducted for the purpose of this paper. All the data relate to water level registered at the hourly rate during a two-year period – from April 2001 to May 2003. The comparison of those water levels is graphically shown in *Figures 2, 3* and *4*.

Dunav-Batina and Drava-Belišće water levels are indicators of water flow into the monitored area, whereas the water level in Kopačko jezero and to an extention of the Drava-Osijek, too, give indications about Kopački rit water condition. Water outlet indicator for Kopački rit is the interrelationship between Kopačko jezero and the Dunav-Vukovar water levels. The Dunav-Batina water levels are usually lower than the Drava-Belišće water levels, but there are exceptions, as well. Maximum and minimum water level values indicate the oscillation range. For example, for this two-year period it is 6.96 m for the Danube at Batina, and 6.35 m for Kopačko jezero -*Tbl. II*.



Figures 3: Water levels of the observed period with an indication of high and low water phases of interes

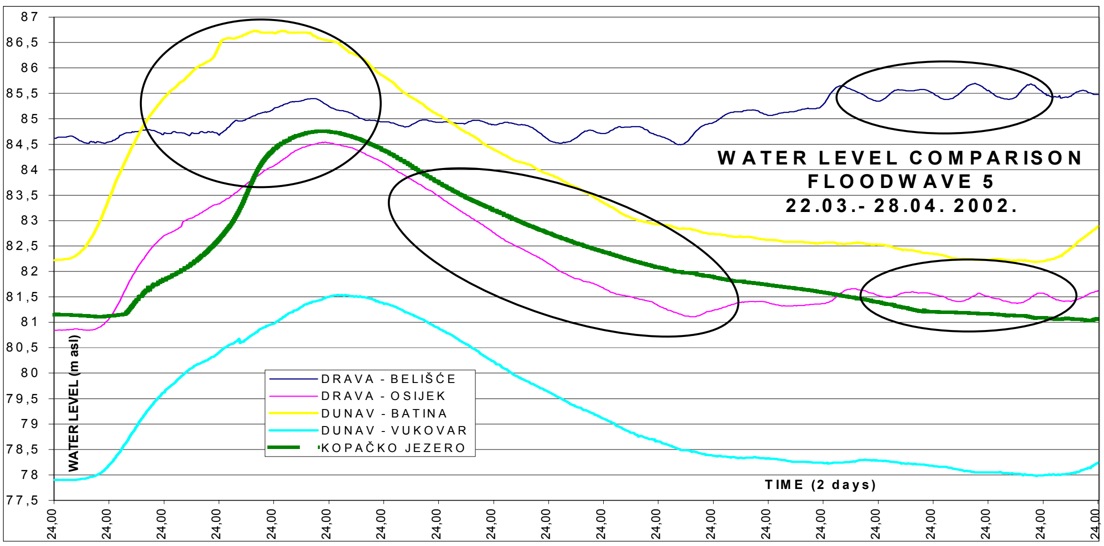
In the obsereved period, the water overflowed seven times to a larger extent in the Rit area, which is indicated by Kopačko jezero (lake) water levels in excess of 82.5 m asl (*Figures 2*). Three times entire area was flooded with water levels in excess of 84.0 m asl (*Figures 3*). Four times a low water period was recorded, with levels under the 81.0 m asl (*Figures 4*).
Water level comparison indicates an excessively dominant impact of the Danube waters on water conditions in Kopačko jezero and Drava at Osijek. As Kopačko jezero (lake) is connected through canals with other big lakes of the southern Kopački rit zone, it is safe to assume that the lake represents a general condition in the entire southern zone.
During high Danube waters, Drava-Osijek is the first to react and then Kopačko jezero. It can be seen from isolated representations of bigger floodwaves in *Figures 3*. Drava-Osijek follows the Danube water level changes at Batina in a more regular manner than Kopačko jezero does.

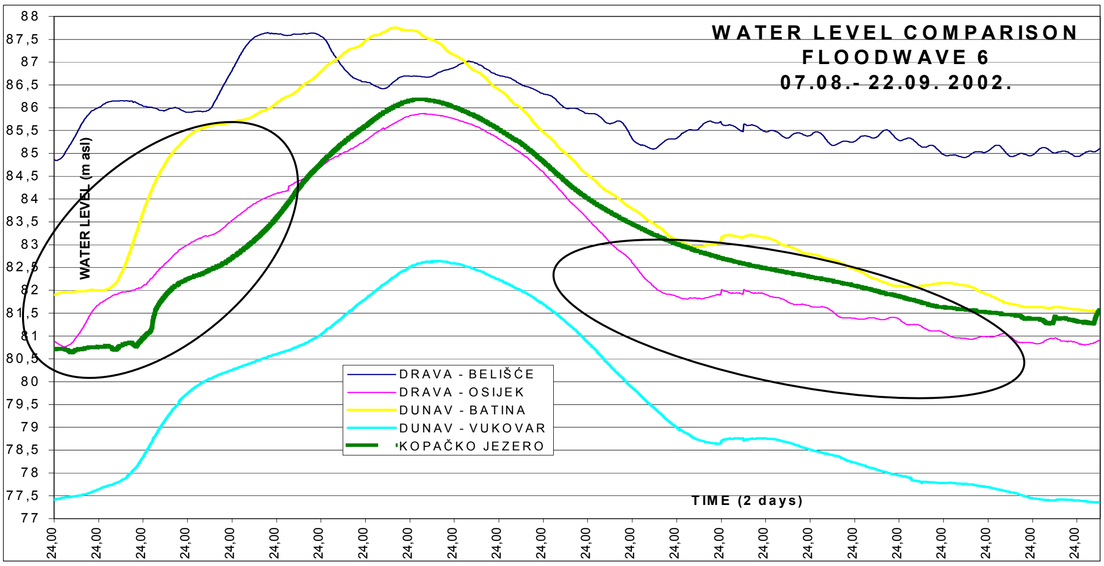
At extremely high Danube water levels, its impact in the form of a back-water rising may be ascertained even at Drava-Belišće. Downstream, at the Danube-Vukovar water levels, retention impacts of the Kopački rit may be observed. Floodwaves here are of a somewhat longer duration, and their level decreases compared to the the same Dunav-Batina waves (a fall of a magnitude order of as much as 1 m is recorded on isolated waves).
Investigations showed that the high water coincidence probability for both the Drava and Danube is not high, but experiences of several floods from the seventies of the last century point out at the possibility of such a coincidence. [10]
A parallel review (*Fig. 2, 3* and *4*) shows a daily water level oscillation rhythm for the Drava. It is lost at Osijek where the impact of back-water rising by the Danube is felt. It is a consequence of the daily operation regime of upstream hydropower plants at the Drava, the nearest one (HE Dubrava) being just over 250 km away.
Low water phases in the observed two-year period of water level measurement are represented in different seasons. It confirms the existence of years with a lower water inlet, i.e. pronounced drought occurrences. Otherwise, in normal hydrological circumstances, at the onset of summer with the beginning of highest insolation and, by extension, evapotranspiration, when the life in and around ponds and lakes abounds, creating the lack of oxygen in the water, that is the time when the high Danube waters arrive. It is precisely the absence of those waters that mostly puts at risk the survival of numerous species in the Kopački rit. No researches have been conducted of subterranean currents in those circumstances as of yet.
The Drava and Danube water level fall causes the flow of water from the Kopački rit. Surface flow occurrs through Hulovski canal into the Danube for as long as the Rit water level is higher than the weir overflow level in the canal (79.50 m asl). After that it is assumed that the dominant role is taken over by the vertical water balance components – precipitation, infiltration, evaporation and evapotranspiration. Backwater in lakes, ponds and canals during warm summer days loses 1-2 cm/day of its water level. Thereby the water surfaces are significantly reduced. [11]
Hydrometeorological calculations indicate the highest potential evapotranspiration of approximately 170 mm (the Šermer equation, grassy surface - [7]), and highest monthly evaporation values for free water surfaces are of that range, too. Even significantly higher values may be expected for wetland areas like the Kopački rit. This is an order of magnitude of water level decrease that falls around those 1-2 cm/day according to the experience. Of particular interest are the data on the water level fluctuation in Kopačko jezero below the 81.0 m asl. *Figures 4* show unusually regular daily rhythm of water level change. Daily ranges of

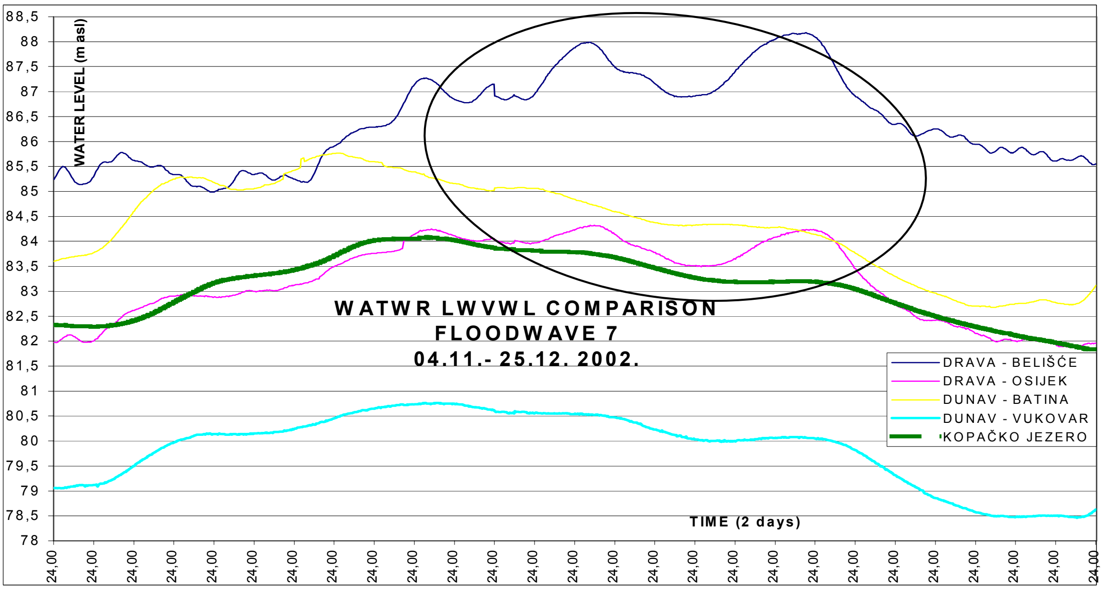
those changes, regardless of whether water levels generally fall or rise, recorded in hot months are of as much as up to ten cm. It opens many questions demanding additional field investigations.

5 Conclusion

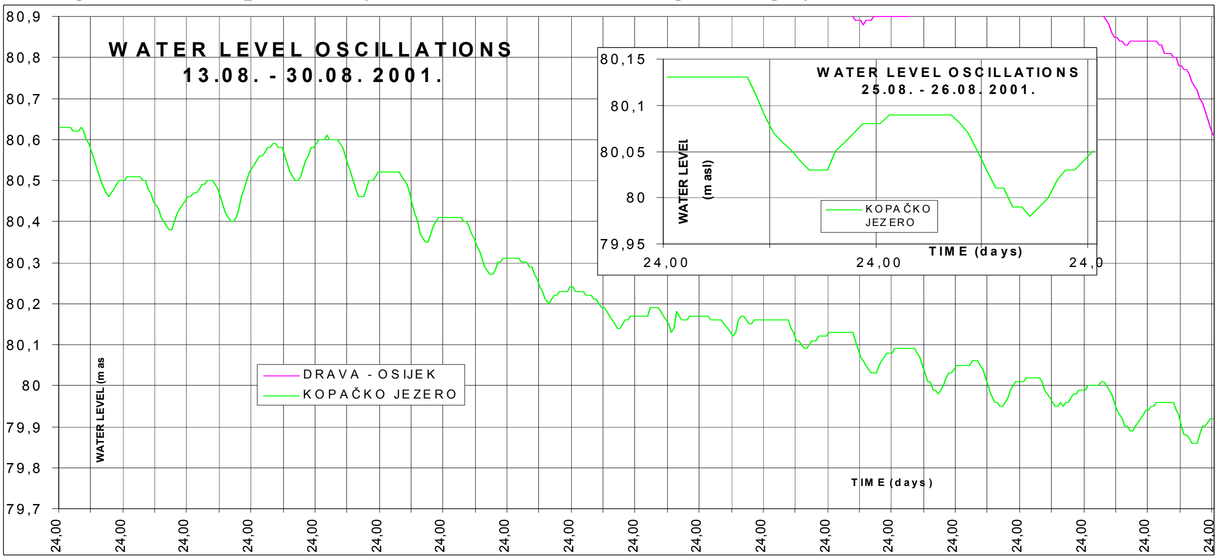
Two-year long monitoring of Kopačko jezero water level is the first continuous hydrological monitoring within the protected area of the Kopački rit natural reserve. Results of that monitoring are a significant information source about water conditions in entire Kopački rit area. Fortunately, this monitoring coincides with a similar (hourly) water level on restored upstream (inlet) hydrological profiles on the Danube at Batina and on the Drava at Belišće, as well as on downstream (outlet) hydrological profile on the Danube at Vukovar. It enables the conducting of the analysis of the Kopački rit area retention effect on the downstream flow of the Danube and the Drava. Preliminary comparison of observed water levels indicates the dominant impact of the Danube waters on Kopački rit water condition, as well as a considerably lesser impact of the Drava. Interesting water level oscillations of the Kopački rit, of the daily and seasonal character, recorded during the low water periods demand additional investigations. Gathered data fund will contribute to mathematical-modelling investigation of water fluctuation in this area, earmarked for the future. Management of the Kopački rit water regime can mainly be based on the management of inlet-outlet water quantities from the Danube. Inlet and outlet of the Danube water primarily depend on the natural morphological changes of canal network in the Kopački rit, but there is also a possibility to construct regulation structures (weirs, dams, pump stations and the like) to that purpose. The establishment of the inlet and outlet control of the Danube water will enable the management of the Kopački rit water regime, directed towards ecological and wider economic goals, and in accordance with the sustainable development concept.

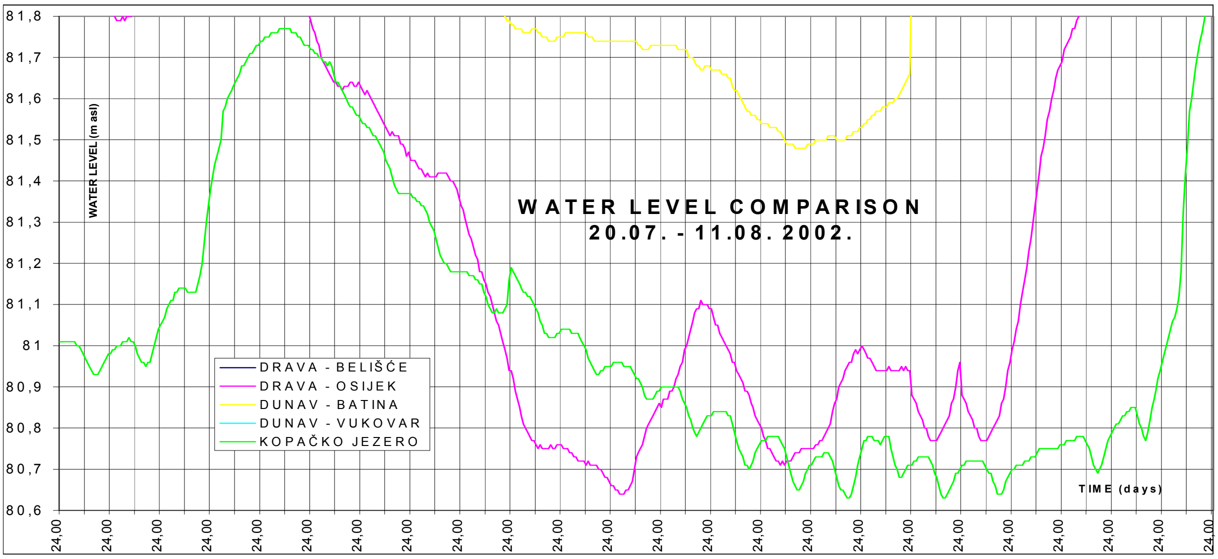


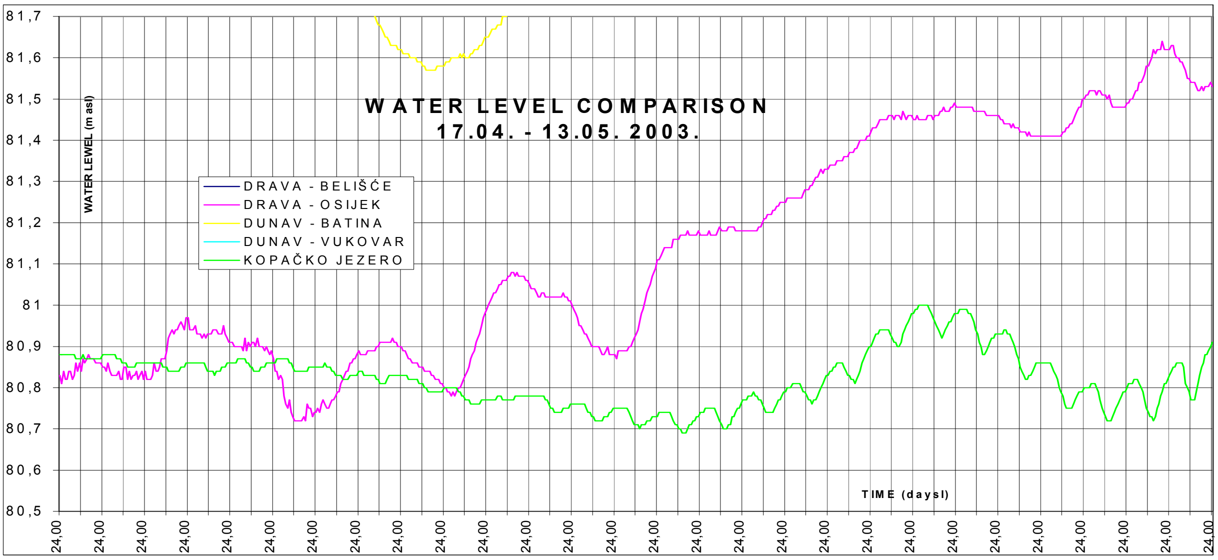




Figures 4: Comparison of the water lewels during the high floodwaves







Figures 5: Kopački rit water levels during low water periods

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An impact of a Level of Maintenance of Hydro-melioration Systems for Drainage on the Plant Crops in Croatia

Prof. Josip Marušić, Ph.D, CEng, University of Zagreb, Faculty for Civil Engineering, Kačićeva 26, 10000 Zagreb, Croatia; marusicj@grad.hr

Damir Bekić, CEng, University of Zagreb, Faculty for Civil Engineering, Kačićeva 26, 10000 Zagreb, Croatia; damirb@grad.hr

Summary

This paper is presenting major indicators of the existing level of the constructed hydro-melioration systems for surface and underground drainage in Croatia. In order to have effective drainage from agricultural and other grounds it is important to maintain hydro-melioration systems and facilities. The indicators of the maintenance of these systems for gravitated drainage are presented through different works done and their expenses. After the Serbian attack on Croatia and war in 1991 and 1992, which followed by difficult situation in agriculture, the national financial means for basin waters have been shortened and the regular maintenance of hydro-melioration systems has suffered. The consequence was low level of drainage of excess waters from agricultural and other grounds, which caused less plant crops. As sowed wheat and corn are 62% of all sown fields in Croatia, this paper is presenting data on plant crops that depend on the level of constructed and maintained hydro-melioration systems.
Key words: hydro-melioration systems, maintenance, expenses, drainage, crops, plants.

1. Introduction

It is the main task of hydro-melioration systems to build and maintain water and air regime of a field under the requirements of most satisfying development and to achieve high plant crops. This is part of the program for food production in every country. Terrain specifications of melioration grounds and requirements for optimum development of the plants are setting up solutions for construction of hydro-melioration facilities and systems for drainage and irrigation; also for realization of specific agro and technical measures and works. Detailed terrain monitoring and testing is needed, as well as analysis of data for topographical, climate, hydro, pedologic, geo-mechanical and vegetation grounds of melioration areas. The role of hydro-melioration facilities is in the drainage of excess waters from the settlements, roads and other buildings under forest vegetation.
Besides finding the most satisfying technical and financial solutions, it is important to regularly maintain hydro-melioration facilities and systems. In Croatia these systems are not constructed enough, and also they are not maintained because of insufficient financial means. Unfortunately the parts of hydro-melioration systems and facilities are destroyed during the war in 1991 and 1992. The war consequences and very bad state of Croatian agriculture from 1991 to 2003 have lowered the means for constant maintenance of hydro-melioration systems for drainage – by the Law on Waters and by the Law of Financing the Water Management. *This has brought to the low level of drainage in relation to the required level for operation of*

hydro-melioration systems in Croatia. The result was low plants crops on the hydro-meliorated fields.

2. The level of construction of hydro-meliorated facilities and systems for drainage in Croatia – 2002.

As a part of Water management project for Croatia, the systematization of data on hydro-melioration facilities has been made for 34 basins and meliorated areas. Relevant data for 4 major water areas in Croatia has been shown in Table 1.

Table 1. Main indicators of meliorated areas and the level of constructed hydro-melioration systems for drainage in Croatia.

Water area (No. of basin aras)	Meliorated field for drainage	The level of construction of hydro-meliorated systems				
		Surface drainage			Underground drainage	
		Fully	Partially	Not-built	Fully	Partially
Sava – ha (13) %	955.334 57,1 (100)	348.363 48,1 (36,5)	107.164 33,0 (11,2)	499.807 80 (52,3)	71.213 58,5	7.280 26,8
Drava-Dunav – ha (7) %	625.439 37,4 (100)	362.240 50,0 (57,8)	204.696 63,1 (32,7)	59.503 9,5 (9,5)	48.197 39,6	19.889 73,2
Primorje and Istra – ha (6) %	43.020 2,6 (100)	1.760 0,2 (4,1)	3.035 0,9 (7,1)	38.225 6,1 (88,8)	1.760 1,5	0
Dalmacija – ha (8) %	48.999 2,9 (100)	12.386 1,7 (25,3)	9.757 3,0 (19,9)	26.845 4,3 (54,8)	314.000 0,3	0
Total of Croatia – ha (34) %	**1.673.792 100 (100)**	**724.748 100 (43,3)**	**324.562 100 (19,4)**	**624.381 100 (37,3)**	**121.484 100**	**27.168 100**

In the areas with fully and partially constructed hydro-meliorated systems for surface drainage it is important to know the significance of the following hydro-meliorated facilities:

1. Length of main water ways — 2 594 km
2. Length of meliorated canals III and IV — 26 357 km
3. Concrete pipe drains, diameter 50-200cm — 21 659 objects
4. Concrete table-like drains with openings from 200-1000cm — 1 466 objects
5. Concrete and rock stairs high 80-120cm — 1085 objects
6. Half-automatic pipe corks with diameter 50-200cm — 506 objects
7. Other hydro-technical objects on meliorated canals — 1466 objects
8. Pumping stations — 75 objects

The most complex and the most expensive hydro-melioration objects are 75 pumping stations with the total capacity of 320,9 m^3 /s and their power of 22.470 kW. They serve for a mechanical drainage of excess waters from 276.000 ha of lowlands, naturally very fertile meliorated areas. Even though there is a high cost for a construction and maintenance of pumping stations, they proved to be needed for drainage and for a regulation of water and air regime in agricultural lowlands where there is no possibility for drainage through gravitation.

It is important to emphasize that properly constructed and maintained hydro-melioration systems for surface drainage are precondition for the operation of hydro-melioration systems and facilities for underground drainage. They are fully constructed on 121.484 ha (14,8%) and partially on 27.169 ha (3,3%). From totally built in 53.089.480 drainage pipes on PVC, 94,8% are with a diameter of 50, 65, and 80 mm, and only 5,2% are with a diameter of 100, 125, 160, 180 and 200mm. The area of 673.697 (81,9%) from the total of 822.350 ha still does not have hydro-melioration systems for underground drainage.

The indicators for the level of construction and significance of the hydro-melioration systems of surface and underground drainage have to be valued in relation of totally meliorated areas (1.673.792 ha) with a totally sowed areas in Croatia. The average of sowed fields from 1976 to 1990 was 1.334.224 ha/yearly, and from 1991 to 2000 only 970.642 ha/yearly. Significant decrease in sowed areas from 1991 was the result of the Serbian attack on Croatia from 1991 to 1997, and the worsened state of agriculture and economy in Croatia.

3. The expenses of the maintenance of the hydro-melioration systems for gravitated surface drainage

Beside a need for a construction of new hydro-melioration systems, it is important to maintain the old ones. In accordance with the most represented project realization elements of melioration canals of III and IV ranges (as well as for a construction under order no 3.), the relevant data are given in the Table 2.

Table 2. Total expenses of the maintenance of the hydro-melioration systems (HMS) for surface drainage (1. row – kn/ha; 2. row – EUR/ha; EUR=7,45kn)

MK-III range		MK-IV range		Total expenses of the maintenance of HMS			
depth-m	surface m²/m	depth-m	surface m²/m	300m 33,3m/ha	275m 36,4m/ha	250m 40,0m/ha	225m 44,4m/ha
2,00	9,20	1,50	7,40	156,0 26,9	165,9 22,3	177,5 23,8	183,8 24,7
2,10	9,56	1,60	7,76	158,9 21,3	169,0 22,7	180,8 24,3	194,6 26,1
2,20	9,92	1,70	8,12	161,8 21,7	172,1 23,1	184,2 24,7	197,3 26,5
2,30	10,28	1,80	8,48	164,7 22,1	175,5 23,6	189,5 25,4	202,8 27,2
2,40	10,64	1,90	8,84	167,6 22,5	178,3 23,9	190,9 25,6	205,6 27,6
2,50	11,0	2,00	9,20	170,5 22,9	181,4 24,4	193,9 26,0	209,3 28,1
2,60	11,36	2,10	9,56	173,4 23,3	184,5 24,8	197,6 26,5	212,9 28,6

An average cost for the maintenance of hydro-melioration systems for gravitated surface drainage are from 156,0 to 212,9 kn/ha or 20,9 to 28,6 EUR/ha, and that is equal to 3,9%-6,1% of expanses for their construction. Relevant expenses are based on the following unit prices:

- one-time mown and bank strips (1,5 times yearly): 0,14kn/m² (0,019 EUR/m²), 36,0%
- one-time mown of a bottom of a canal (1,5times yearly): 0,57 kn/m² (0,077 EUR/m²), 12,0%
- cleaning of a canal bottom (every 4 years): 6,05 kn/m² (0,81 EUR/m²), 40%
- other maintenance work participates with 12 % in total expenses of maintenance of hydro-melioration systems for gravitated surface drainage

For regular maintenance services of fully constructed HMS for gravitated surface drainage in the area of 724.749 ha it is necessary to secure financial means from basin water payments in the amount of 133.716.190 kn or 17.948.482 Euros. In order to regularly maintain partially constructed HMS for surface drainage in the area of 324.662 ha it is necessary to secure financial means from the basin water payments in the amount of 44.803.360 or 6.013.873 Euros. It is also important to know that 64.394.850 kn is needed for the regular maintenance and work of 75 pumping stations. The maintenance of local waterways (the part of MK-II range) in basin and meliorated areas demands the amount of 84.440.700 kn. *From the above written indicators it is clear that 327.355.100 kn or 43.940.282 Euros is needed to maintain hydro-melioration systems for surface drainage.* From 1991 to 2001 only 18% to 42% financial means from basin water payments are realized for the expenses of minimum maintenance requirements for HMS of gravitated surface drainage. Special financial payments have to be secured for restoration and maintenance of swift current water ways, and for erosion protection of basin and meliorated areas. For the operation of hydro-melioration facilities and systems, it is essential to have the preconditions for it. The preconditions consist of regular maintenance and construction of an annexe and new protective hydro-technical objects for a protection from flood waters and swift current water ways.

4. Wheat and corn crops in the areas with different level of constructed and maintained hydro-melioration systems.

Based on systematically analyzed data from 1976 to 1990 and 2000, the major indicators of realized wheat and corn crops in partially and fully constructed hydro-melioration systems for surface and underground drainage are given.

Table 3. Realized wheat and corn crops from 1976 to 2000.

Hydro-melioration systems for drainage – a level of construction and maintenance	Realized crops (t/ha)					
	wheat			corn		
	min	average	max	min	average	max
1. Surface and underground drainage – constructed						
a) regularly maintained 1981-1990	**5,70**	**7,41**	**8,74**	**6,89**	**8,03**	**9,98**
b) partially maintained 1991-2000	4,75	6,23	7,22	5,94	6,75	8,18
c) not maintained 1991-2000	3,93	4,36	5,95	4,89	5,82	6,88
2. Surface drainage – fully constructed						
a) regularly maintained 1981-1990	**4,22**	**5,19**	**6,03**	**5,11**	**6,12**	**7,18**
b) partially maintained 1991-2000	3,25	4,13	4,57	3,99	4,68	5,56
c) not maintained 1991-2000	2,54	3,11	3,72	3,26	3,72	4,49
3. Surface drainage – partially constructed						
a) regularly maintained 1981-1990	**3,37**	**3,92**	**4,16**	**4,06**	**4,38**	**5,06**
b) partially maintained 1991-2000	2,60	2,97	3,34	3,08	3,29	3,69
c) not maintained 1991-2000	1,98	2,32	2,68	2,46	2,67	2,97
An average for Croatia 1976-2000	3,04	3,80	5,02	3,84	4,47	5,16
An average for Slavonija and Baranja 1976-2000	4,04	4,97	6,50	4,26	5,68	7,08

Beside above mentioned data it is essential to have in mind that there are also wheat and corn harvest areas, and the main indicators are following:

	1976-1990	1991-2000	distinction
Total of the sowed area in Croatia	1.334.224 ha	970.642 ha	-363.582 ha
Wheat harvest areas in Croatia	323.106 ha	218.645 ha	- 104.401 ha
Corn harvest in Croatia	502.131 ha	383.899 ha	-118.232 ha
Wheat harvest areas in Slavonija and Baranja	143.841 ha	107.980 ha	-35.801 ha
Corn harvest areas in Slavonija and Baranja	204.015 ha	156.442 ha	-47. 573 ha

The data for Slavonija and Baranja are excluded because of the high level of their constructed hydro-melioration systems for surface and underground drainage comparing to the other meliorated grounds in Croatia. Significant data is also that from 1956 to 1990 in Slavonija and Baranja 490.484 ha of the ground is covered with reccommasation and hydro-melioration, and this is 77,7% from the whole hydro-meliorated fields (631.648 ha) in Croatia in the same time period.

From 1976 to 1990 in the totally sowed area in Croatia of 1.334.224 ha/year, 825.237 ha/year of a field or 61,9 % was under wheat and corn. In an average sowed area of 970.642 ha/year, from 1991 to 2000, 602.544 ha/year or 62,1% was under wheat and corn. From this data it is visible that other plants are contributing with 38% in sowed fields in Croatia. The level of hydro-melioration systems construction and its maintenance have about the same influence on other plants in Croatia, which areas are smaller then the ones of wheat and corn. This is firstly related to barley, sunflower, sugar-beet, soya beans and for all the vegetables, as well as for plants used for cattle-nutrition. From the small wheat and corn crops it is clear that the construction and the maintenance of hydro-melioration systems have to be improved, this way the higher crops for whole plant cultivation will be realized. The confirmation for this statement is data presented in Table 3, and numeric and graphic indicators on the pictures 1. and 2. Unfortunately, all the documented indicators and suggestions that there is a need for an annexe and construction of new hydro-melioration facilities and systems for surface and underground drainage are not taken into consideration or realized from 1991. The result of this is a small crops from the most of the plants, and an increase in deficit in agriculture because of high food import and low food export in Croatia.

In the framework of mentioned indicators belongs a fact that major part of hydro-melioration objects that have been destroyed during the war has been reconstructed until 2002. Due to lowering of the basin waters financial means there is no maintenance of both war destructed and remained hydro-melioration objects. Construction of an annexe and the maintenance of hydro-technical objects are significant for damages done by water and for flood protection of rivers and torrent water ways. *Based on the positive terrain characteristics and on pedologic ground components, Croatia has a high chance to satisfy needs for its own food production and for an export of it. This is possible to realize through a construction of an annexe and through regular maintenance of existing, and construction of new hydro-melioration systems and objects.*

5. Summary

From the total needs to build on 1.673.792 ha, the hydro-melioration systems for surface drainage are fully constructed on 43,3%, partially on 19,4%, and they are not constructed on 37,3%. *Besides creating most suitable technical and financial realization solutions, it is necessary to maintain hydro-melioration objects and systems on time. From 1991 the financial means from water basin payments cause irregular maintenance of hydro-melioration systems and objects together with an insufficient level of their construction.* Now the hydro-melioration systems work poorly on the drainage of agricultural and other fields. The consequence of this are increased expenses in plant cultivation and low number of plant crops.

In the areas with fully constructed and regularly maintained hydro-melioration objects and systems for surface drainage an average wheat crops are 5,19 t/ha, and corn crops are 6,12 t/ha. In the are with partially maintained hydro-melioration systems wheat crops are 4,13 t/ha and 4,68 t/ha, and in an area without maintenance are 3,11 t/ha and 3,72 t/ha. The expenses for a regular maintenance are from 20,9 to 28,6 EUR/ha, which responds to equivalent wheat value from 0,156 to 0,213 t/ha – and this is from 3,0 % to 4,1 % of average wheat crop (5,19 t/ha). Lowering of a wheat crop in the areas without regular maintenance of hydro-melioration systems is 2,08 t/ha, which is for 13,33 or 9,77 times more then equivalent wheat value (0,156-0,213 t/ha) for the works of regular maintenance (20,8 to 28,6 EUR/ha).

Detailed indicators on a relation of expenses for regular maintenance and wheat and corn crops on a fully constructed and maintained, and with partially maintained and not maintained hydro-melioration systems for drainage are presented in Tables 2. and 3. With the fact in a mind on realized wheat and corn crops (t/ha) and their value in Euros (EUR/ha), it is very visible and of big importance to maintain hydro-melioration systems and objects for drainage. Here, it is also important to do an analysis of higher expenses for ground preparation and plant cultivation in partially maintained areas and in the fields where hydro-melioration systems are not regularly maintained at all.

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Development of Quay Structures for Container Harbours

Bolesław Mazurkiewicz

Gdańsk University of Technology, Department of Marine Civil Engineering, ul. G. Narutowicza 11/12, 80-952 Gdańsk, Poland, bmazur@pg.gda.pl

Abstract
The development of modern container terminals, particularly for new generation of container ships', requires a proper preparation of waterfront structures, as well as the whole terminal area, in relation to storage requirements, cranes, crane tracks, etc.
The paper describes the development of modern quay wall structures fulfilling the requirements concerning the increased loads, particularly from heavy container cranes, and requirements concerning scour protection against ships' propellers induced currents.

1 Introduction

Recently, the main requirements concerning harbour structures, particularly quay walls for container terminals, may be presented and summarized under three thematic headings, namely:

Structure: consideration of all factors influencing the safety, stability and long working life for possible environmental conditions and expected loadings, as well as influences of berthing ships.

Quality: the long working life of the structure should be assured through high quality of materials used, as well as through proper construction procedure during the erection period.

Costs: the economic viability for the whole assumed working cycle of the structure is an essential part of design, construction and operation period.

The proper design of the whole structure should be based on a wide analysis of the:
- soil conditions, particularly from the point of view of the assumption of the bearing capacity of elements of the structure,
- impacts of water pressure, particularly in ports with significant changes of water level in the harbour basin and in the soil behind the quay wall structure,
- impacts of berting ships', particularly during approach of the ship to the quay wall and during mooring procedure,
- influence of the ship's propeller induced currents acting on the harbour bottom and on the whole structure during the berthing and mooring manouevres,
- influence of the concrete superstructure on the distribution of earth pressure acting on the whole quay wall.

Considering the recently available design and calculation methods one has to state that they have to be analyzed from the point of view of the following questions:
- is the design still the right one today?

- do the fundamentals used in the stability calculation match up to the latest state of technology and sciences?
- does actual site stability of existing structures diverge from what was calculated?

It is of course evident that the full answer to these questions can be obtained only during certain investigations on structures and in the soil eventually through introduction of very reliable calculation methods e.g. using finite – element- methods, as well as through trials on models and performance of numerical calculations to follow up fresh data discovered in the course of measurements.

Generally, it is know that numerous measurement data records are available giving the possibility of an analysis of cases for which these measurements were made. However, the number of investigated cases is relatively very small and thus they don't allow on certain generalization. This situation indicates that the modern berthing structures should be designed through introduction of all possible influences connected with soil conditions, water level changes, berthing procedures depending on the type of ships, distribution and value of surcharge, car and rail transportation requirements, as well as influence of different types of cranes and unloading equipment particularly important for container terminals.

2 Types of structures

Considering the actual requirements concerning the type and magnitude of ships which have to be used in the near future, one has to state that the predominant vessel is the container ship predicted to have a capacity of 12,500 TEU. For this type of ship the required depth at the quay wall is 19.65 m in relation to average sea water level, assumed N.A.P = ± 0.00 m. Introducing the quay wall upper structure level on + 4.0 m one obtains a structure hight above sea bottom level of about 24.0 m. Subsequently for a container terminal the surcharge should be assumed as equal to 40 kPa at the first 35 m and 60 kPa in the next paved areas. The container crane beams have to be installed in a distance of 5.0 m and 35.48 m from the quay wall mooring line.

Assuming that for the required quay wall depth the structural depth is about – 23.0 m, the types of structures presented in Fig.1 might be analyzed as proper for the described case. These structures are:

- block wall,
- floating caisson wall,
- L-shape wall,
- gravity concrete wall,
- terre armee wall,
- piled superstructure (jetty) wall,
- combined sheet pile wall,
- slurry trench wall,
- double sheet pile cofferdam wall,
- concrete wall with SI – anchors.

Using a multiple criteria analysis, which comprises following design aspects:

- costs of structure and its construction,
- maintenance,
- construction possibilities,
- risk for overloading,

- risk for collision,
- flexibility,
- construction time,
- durability, reuse,

it was obtained (de Gijt et al., 2003) that the block wall, floating caisson wall, L-shape wall and gravity concrete wall are very good structures but the costs of these structures are from 1.67 to 2.25 times higher than the costs of combined sheet pile wall. All other structures are also more expensive, while the terre armee wall, the piled structure wall and the cellular cofferdam wall are negative from the point of view of risk for overloading, risk for collision and flexibility. Very competitive are the combined sheet pile and the slurry trench walls, while the last one can be constructed in shorter time. An interesting solution is the concrete wall with SI – anchors. It fulfills all requirements although it is more expensive than the combined steel pile wall (1.18).

Taking into consideration the above conclusions and requirements concerning above all the protection of the quay wall from the scour induced by ships' propellers, it is possible to state at the time being that the combined sheet pile wall can be recommended, particularly due to the recent development of different types of quay wall elements e.g. sheet piles, anchor piles, superstructures, etc. It does not mean, however, that other types of quay walls are not used. As an example, the recent use of block quay walls in the Mediterranean area might be mentioned (Zdansky, 2002).

3 Recommended quay wall structure

Taking into consideration the above conclusions it has to be stated that the recommended quay wall structure should fulfill the following requirements:

- assure high bearing capacity of the front part of the quay wall to take over the high crane loads particularly from the waterside leg of the container crane,
- allow on the difference in height of over 27 metres to the bottom for the purpose of calculation,
- protect the whole structure against sccouring caused by ships' propellers induced currents.

Assuming in addition that the subsoil is characterized by sharply varying soil conditions and correspondingly differing parameters on soil mechanics, the main task is to design a structure which will be stable for all loading conditions, as well as safe from the point of view of scouring which has to be reduced through structural measures.

The first type is presented in Fig.1g. It is a combined sheet pile wall very often used in the Port of Rotterdam (de Gijt et al., 2003). It is a proven and reliable structure also when the aspects of corrosion and scouring protection are considered. It is characterized by low-level superstructure slab with pile foundation to reduce soil pressure on sheet piling. The reduction of the threat of scouring is achieved through the inclined wall directing the propeller induced current to the surface, thus reducing considerably the current acting on the bottom. Bottom is additionally protected by a layer of stones laid on geotextiles.

The second type is presented on Fig.2 and was used among other for the Container Terminal Altenwerder in Hamburg (Miller, 2003). It is also characterized by a low-level superstructure slab on piles which allow on the reduction of soil pressure on the sheet piling. A mean feature is the possibility for reduction of the threat of scouring, namely, through a row of fender piles driven in front of the sheet pile wall in a distance of 4 m. A kind of chamber is created in which the ships' propeller induced currents are dumped due to a turbulence occuring between the fender pile wall and sheet pile wall made as combined wall of two HZ 975B H-sections and sheet piles AZ 13-10. The fender piles were made of tubes with a diameter 1219,2 mm and wall thickness 16mm. The distance between tube axes was 4,92 m. An open cross-section with hollow space below the superstructure slab was introduced to reduce water overpressure. The horizontal forces are taken over through raking piles up to 47 m long giving stable anchorage and minimizing the horizontal deformation of the whole structure. The crane tracks are on separate foundations.

Concerning crane loads it has to be mentioned that during terminal operation the following loads will occur:

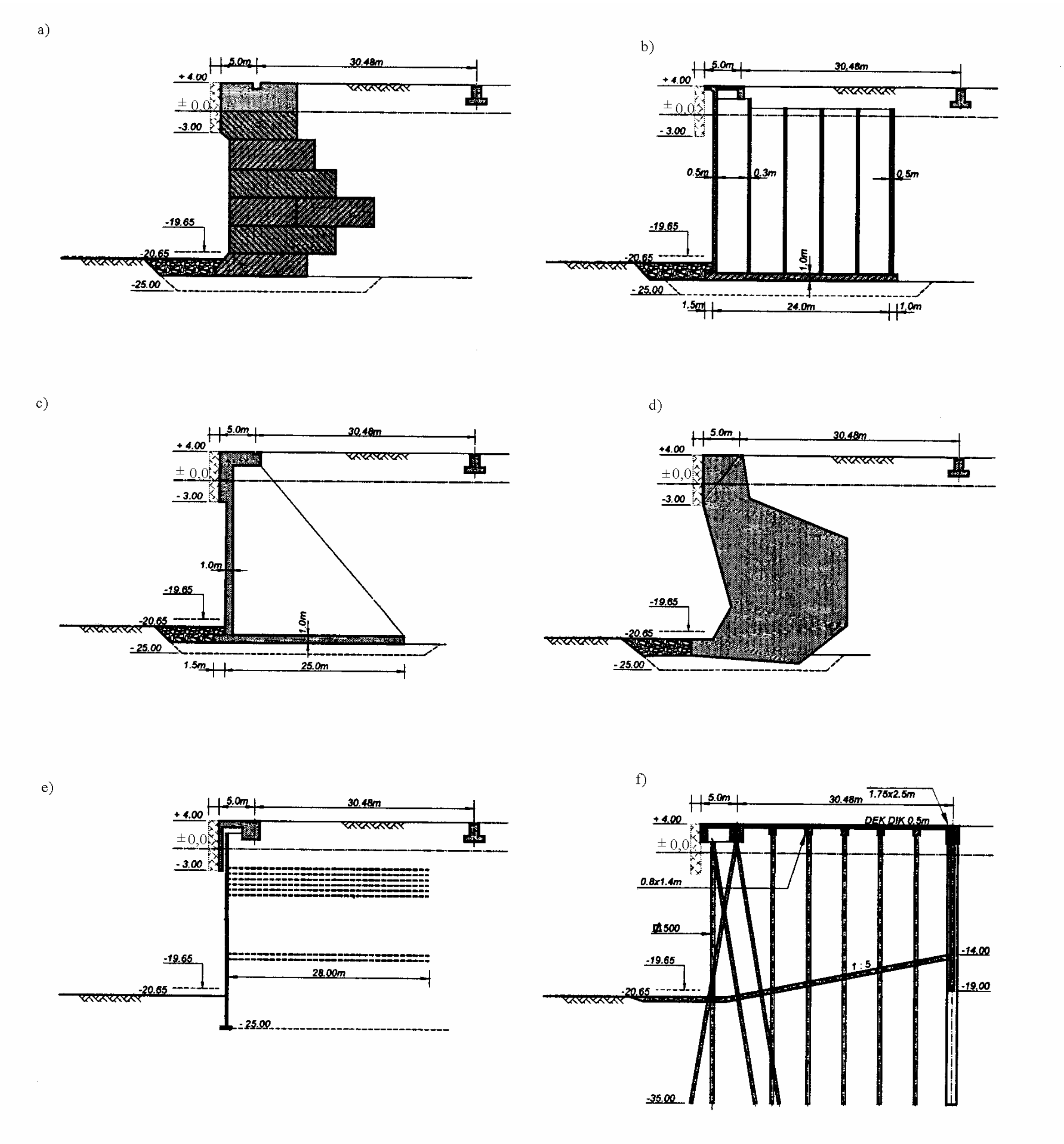
- loads induced by the crane's weight,
- loads induced by loading and unloading of container to /from a ship's hull,
- loads induced by wind on the structure of the crane,
- loads influenced by acceleration, driving and deceleration of crane.

From several investigations it can be concluded that the operational loads are within the ranges calculated by the manufacturer. This means that the stability calculations of the whole structure together with the crane tracks are based on real loads values.

Recommending the combined sheet pile wall as the main structural element of the quay wall for large depths and considerable surcharge i.e. in the application for a container terminal, it seems valuable to refer to numerous measurement data records which indicate that:

- the moment curve of the sheet piling corresponds at its base to a partially fixed or clamped wall and shows a rigid corner frame on the upper edge at the point of incorporation in the quay wall superstructure;
- the design of the anchor pile can be determined by the application of the occuring loads on the tension pile into the subsoil via friction action; it shall be, however, considered that in the case of filling inserted behind the new sheet pile wall after driving of all structural elements, the tension rises to a certain amount which can be generally assumed to be lower than the design load although it is also possible that due to driving of all other piles supporting the concrete superstructure and settlements of the fill, the pile tension loads can increase above the design load; thus it is recommended that in designing of the anchor a load of a maximum of 70% of admissible stresses should be introduced;
- the crane track beams can be designed and constructed as jointless beams on the whole required length of the structure;
- the classical earth pressure theory can be further applied; several calculation metod using e.g. finite element methods FEM, indicate small differences, particularly when the material model input and the network structure are properly chosen.

Concluding and recommending for future use the combined sheet pile walls, it is evident that further investigations of these structures seems to be necessary to obtain a stable and economical structure at all. These investigations should concern above all the determination of the deformation in the sheet piling, determination of the development of soil pressure on the active and passive side of the sheet piling and determination of the load and deformation behaviour particularly of inclined anchor piles. It has to be also mentioned that the protection of the whole structure against scour can be achieved through significant increase of sheet pile driving depth. An increase of sheet pile length of about 4.0 m seems to be sufficient to sustain the total stability in the case of development of scour holes.



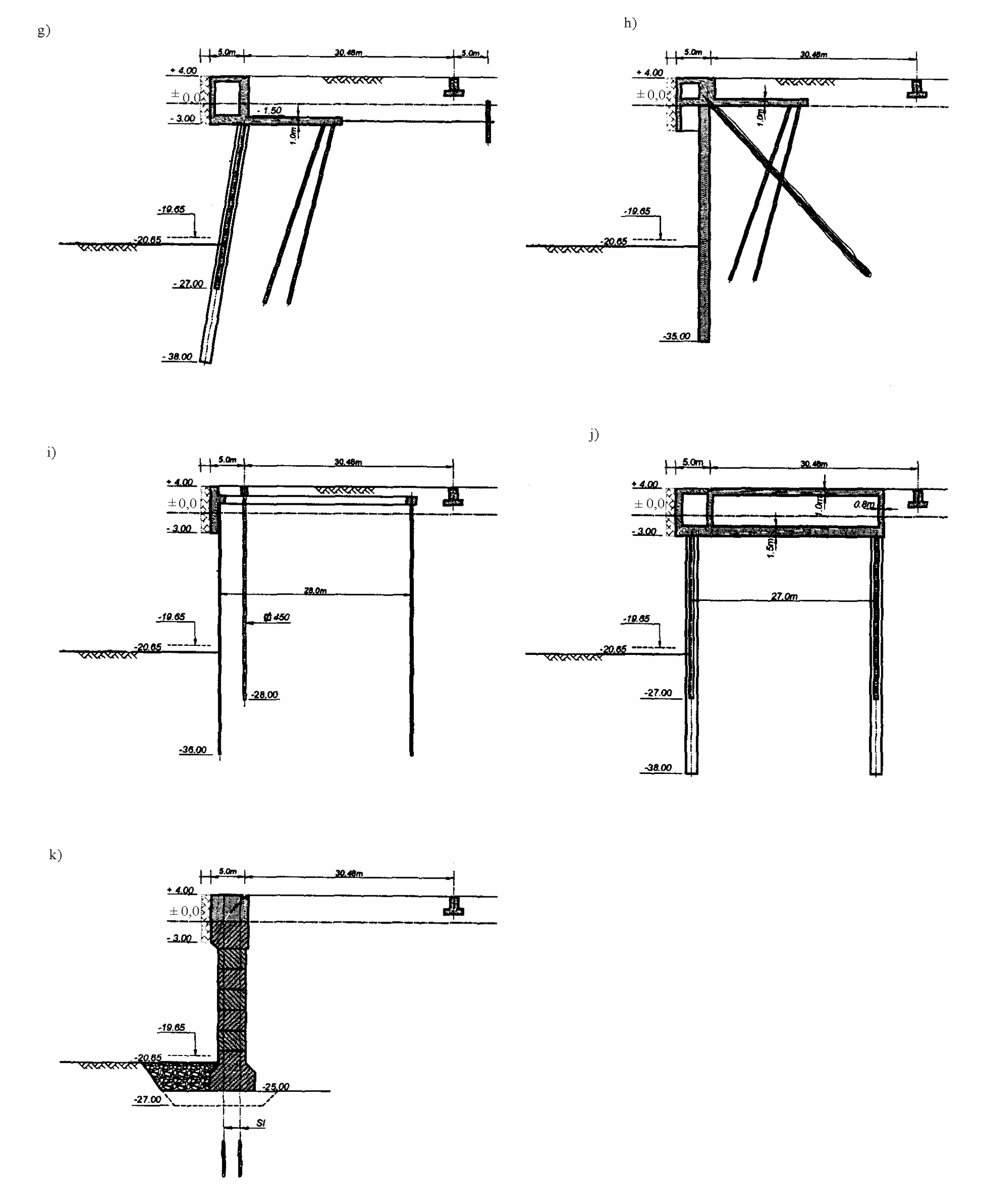


Fig.1 Quay wall structure types: a-block wall, b-floating caisson wall, c-L-shape wall, d-gravity concrete wall, e-terre armee wall, f-piled superstructure (jetty) wall, g-combined sheet pile wall, h-slurry trench wall, i-cellular cofferdam wall, j-double sheet pile cofferdam wall, k-concrete wall with SI-anchors (de Gijt, et al., 2003).

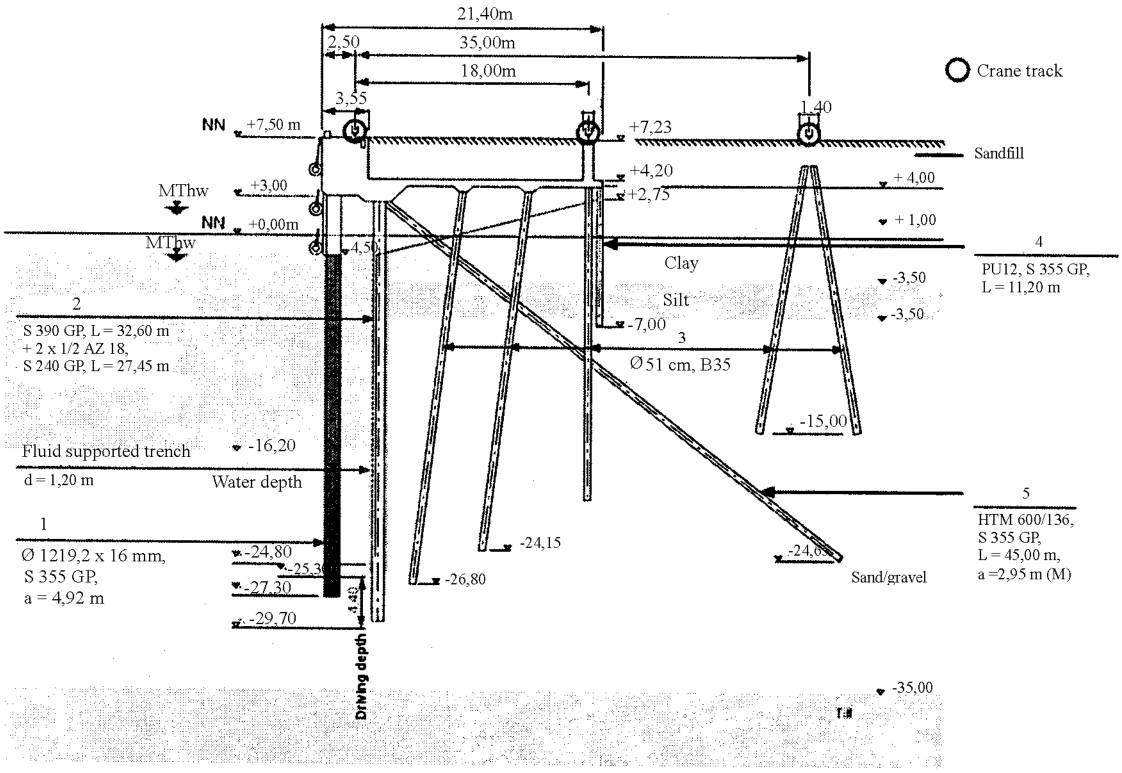


Fig.2 Combined sheet pile wall: 1-fender piles, 2-combined sheet piles, 3-prefabricated concrete piles, 4-sheet piles, 5-anchor piles (Miller, 2003)

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Numerical Methods for Investigation of Soil Settlements due to the Ground Water Well Supply System

Lena A. Mihova & Ivailo.J. Ivanov

Dep. of Geotechnics, University of Architecture, Civil Engineering and Geodesy, 1 Christo Smirnenski blv., 1000 Sofia, Bulgaria; e-mail: iji_fte@uacg.bg

Abstract

Soil deformations due to water level lowering have been studied for the case of groundwater extraction from a draw well. The development of the depression surface is determined according to Theis formula. Deformations appear because of soil's bulk density changes in the transition from water-saturated to natural condition. A 3D simulation has been performed according to the finite-element method (FEM) with process discretisation over time. The E-module changing depending on the stress-strain state changes of the soil massif has been considered. An evaluation of an approximate 1D solution has been made.

1. Introduction

In a number of countries in the world, studies have been carried out for the relationship between the groundwater discharge and the soil deformations ensuing from it. Such a problem stands before a number of countries and regions in the world where large quantities of groundwater are extracted. Significant soil settlements in USA, Mexico, Israel, caused by lowering of groundwater levels have been reported. This problem is especially important when deformations appear on the surface, since this may cause collapsing of buildings and structures in urban areas. For the solution of the problem the groundwater extraction is usually reduced but in this case the problem becomes a bilateral one because reducing the discharged water quantity in order to reduce deformations causes disturbances in the water supply of the respective settlement.

Similarly, in Bulgaria, and especially in Sofia, subgrade deformations appear due to increased groundwater extraction. Several areas being around big water-intake systems are reported to suffer increased settlements.

There are still big difficulties in forecasting deformations due to their complex nature. In a recent article [6] authors tried to simulate soil deformations and the influence of soil layers with different deformation characteristics (existence of a low-deformable layer) on them. This paper attempts to develop the study, seeking a relationship between 3D- and 1D- solutions for development of soil deformations with discharging.

2. Formulation of the task and preconditions

A single well has been considered, having a constant discharge capacity in an infinite-in-plan free-flow water-bearing layer (Fig.1) with a thickness H. The reduction of the pressure s_w due to water discharge is determined according to the logarithmic version of the Theis formula for a quasi-stabilised regime of seepage in pressure layers [4,7].

$$s_w = \frac{0{,}183Q}{T} \lg \frac{2{,}25at}{r^2} \tag{1}$$

where Q - water discharge capacity of the well; T - permeability of the layer = $H.k$; k - coefficient of seepage; t - time; r – radial distance from the well; a – coefficient of level transmission.

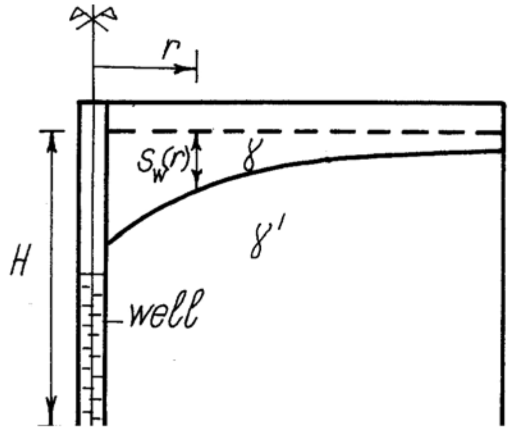


Figure 1. The wellbore

Formula (1) can be used for free-flow layers if the maximum lowering reached during the whole period of discharge does not exceed 20% of the water-saturated thickness of the free-flow layer. The following preconditions have been set for the solution of the problem:
The seepage parameters of the water-bearing layers are averaged in plan and section.
The maximum lowering of the groundwater level s_w for the period of study does not exceed 20% of the layer thickness.
The soil massif is regarded as a non-linearly deformable isotropic medium with the following deformation characteristics: tangential module E_t and Poisson coefficient $v = const.$ The module E_t is determined according to the hyperbolic model of Duncan & Chang [1, 2] using the following formulae:

$$E_t = KP_a\left(\frac{\sigma_3}{P_a}\right)^n \beta^2 \tag{2}$$

$$\beta = 1 - \frac{R_f(\sigma_1 - \sigma_3)(1 - \sin\varphi)}{2\sigma_3 \sin\varphi + 2c\cos\varphi} \tag{3}$$

$$\varphi = \varphi_0 - \Delta\varphi \lg\left(\frac{\sigma_3}{P_a}\right) \tag{4}$$

where $K, n, R_f, \Delta\varphi$ are hyperbolic parameters.
In water discharge, the water seepage does not reach the critical rate when a hydraulic destruction of the soil occurs.
Deformations in the soil massif are observed when the bulk density of the soil changes in case of transition from water-saturated condition to natural condition.
Deformations develop over time in a direct relationship with the depression surface change.
The stress-strain state of the soil massif is axi-symmetrical.

3. Numerical simulation

3.1. Three-dimensional solution (model)

Since the water level lowering s_w depends on lgt, it follows that $\Delta s_w/\Delta lgt = const.$ Process discretisation is made with respect to time, and for different moments of time $t = t_0, \ldots t_i, \ldots\ldots t_n$ the value of s_w is determined. The moments t_i are selected in such a way that the intervals $lg(t_i - t_{i-1}) = lg(\Delta t_i)$.

For each time interval Δt, loading is obtained from the difference in the soil weight in natural and water-saturated condition $(\gamma - \gamma`)$ of the soil strip between depression surfaces in the moments t_{i-1} and t_i. The depth of the deformable soil massif (active zone) H_a is determined following the recommendations in case of large-area loads on the subgrade [3, 5]. The tangential deformation module E_t of formula (1) is determined according to the reached level of stresses σ_1 and σ_3 in the soil massif due to the soil weight (static load). The final solution is obtained through superposition.

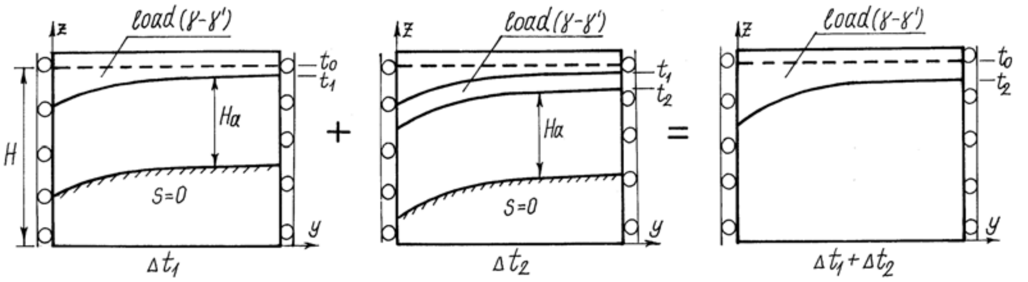


Figure 2. Algorithm of 3D solution

The algorithm of the solution is shown in Figure 2, the finite elements method (FEM) for axi-symmetrical problem being used for this purpose.

3.2. One-dimensional solution (model)

For a vertical located at a distance r from the well, the stress value is determined as:

$$Pz_i = s_{wi}(r)(\gamma - \gamma`) \tag{5}$$

It is assumed constant for the whole thickness of this vertical. The settlement of the points of the soil massif's surface, located at a distance r of the well in the moment t_i is determined according to the formula:

$$S_i = \frac{Pz_i Ha}{E_0} \tag{6}$$

where E_0 is the averaged deformation module within the frames of H_a. In case of availability of different soil layers with thicknesses h_j and modules E_{0j}, E_0 is determined according to the expression:

$$E_0 = \frac{\sum_{j=1}^{n} Pz_i h_j}{\sum_{j=1}^{n} \frac{Pz_i h_j}{E_{oj}}} \tag{7}$$

4. Numerical results

A calculation has been made for a homogenous soil massif of loose clayey sand with averaged values of the physical and mechanical characteristics [8]: $\gamma = 18\ kN/m^3$, $\gamma`$ (under water) = $9.7\ kN/m^3$, $e = 0.70$, $\varphi = 28^0$, $c = 20\ kPa$, $\nu = 0.3$, $E_0 = 10000\ kPa$; hyperbolic parameters – $K = 260$, $n = 0.65$, $R_f = 0.8$; seepage parameters – $k_f = 2.3\ m/24h$, $a = 10000\ m^2/24h$, $Q = 5\ l/s$. The deformation behavior of the soil massif has been studied at a layer thickness $H = 20\ m;\ 40\ m$. The static level before the discharge is accepted to coincide with the upper surface of the water-bearing layer (the soil massif). The change of the depression curves is shown in Figure 3.

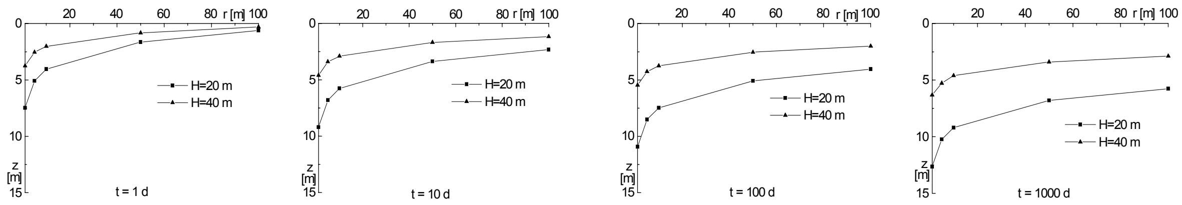
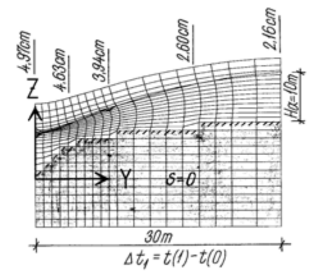
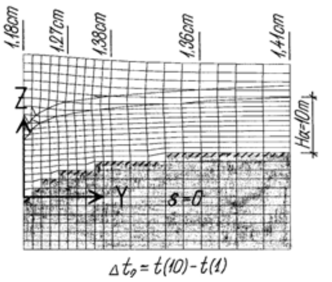


Figure 3. The depression curves for a different thickness H of the water bearing layer

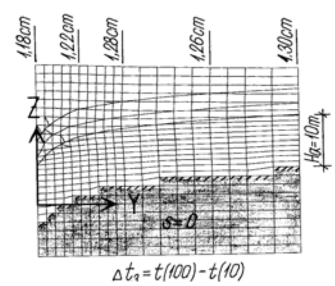
Figure 4 indicates the deformation schemes of the digital 3D model of soil massif according to FEM with module E_t at H=20 m for moments of time $t = 1d, 100d, 1000d.$ The active zone $H_a = 10\ m$ for each time step.



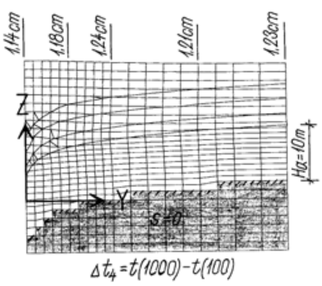
a)



b)



c)



d)

Figure 4. Deformation schemes of a soil massif for 3D solution

Figure 5 illustrates a juxtaposition of surface settlement solutions, as follows:

a) 3D solution for a soil massif with E_t,

b) 3D solution for a soil massif with $E_0 = 10000\ kPa$,

c) 1D solution with $E_0 = 10000\ kPa$.

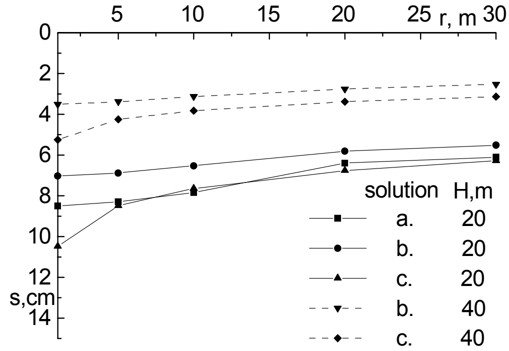


Figure 5. Stabilised settlement *(t = 1000 d)* for the following solutions:

a) 3D solution for a soil massif with E_t,
b) 3D solution for a soil massif with E_0,
c) 1D solution for a soil massif with E_0.

The following conclusions may be drawn hereof:

1. Up to 65% of the settlement is realised in the first 10 days from the beginning of the discharge.
2. The deformations in the soil massif follow the development of the depression surface, while the zone of the massif above the water level line is not deformed practically.
3. Option a) with variable module E_t leads to 10-20% higher values of settlement compared to Option b/ with E_0 since the bigger differences are in the zone of the most curvature of the depression well (in this case at a distance $r = 5\text{-}15\ m$ from the draw well). This is so because $\sigma_3 = const,$ and σ_1 grows, thereby leading to smaller values of E_t.
4. With the 1D model higher values of the settlement in a stabilised condition are obtained compared to the 3D model for a massif with E_0 (Fig. 5). For both thicknesses of the permeable layer - $H = 20\ m$ and $H = 40\ m$, respectively for different values of the permeability, the following relationship between the settlement options W can be accepted:

$$W_{3D} = mW_{1D} \tag{8}$$

where he values of the coefficient m depend on r and are shown in Table 1.

Table 1

r (m)	m
1	0.67
5	0.80
10	0.83
> 10	0.85

For such values of soil- and seepage parameters, the 1D model can be used for an approximate evaluation of the settlement for $r > 10\ m$.

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Water hammer analysis in two pipes in series

Marek Mitosek
Warsaw University of Technology, Fac. of Environmental Engineering, ul. Nowowiejska 20, 00-653 Warsaw, Poland, Marek.Mitosek@is.pw.edu.pl

Romuald Szymkiewicz
Gdansk University of Technology, Fac. of Hydro- and Environmental Engineering, ul. Narutowicza 11/12, 80-952 Gdansk, Poland, rszym@pg.gda.pl

Abstract
Some results of investigations of unsteady flow in two pipes in series are presented. The data provided by an experiment carried out for steel pipe were compared with the results of calculations. To solve the system of water hammer equations the modified finite element method was used. An example of application shows that varied diameter of pipe fundamentally changes the head oscillations comparing with ones in simple pipe. The most remarkable difference deals with the frequency of head oscillations.

1 Introduction

The unsteady flow in pipe is described by the following system of equations [6, 10]:

$$\frac{\partial V}{\partial t} + V\frac{\partial V}{\partial x} + g\frac{\partial H}{\partial x} + \frac{f}{2D}V|V| = 0 \tag{1}$$

$$\frac{\partial H}{\partial t} + V\frac{\partial H}{\partial x} + \frac{c^2}{g}\frac{\partial V}{\partial x} = 0 \tag{2}$$

where: x – space co-ordinate, t – time, V – velocity in pipe, H – piezometric head, f – friction factor, D – inside diameter of pipe, g – acceleration of gravity, c – velocity of pressure wave. The pressure wave velocity is expressed as follows:

$$c = \frac{1}{\sqrt{\rho\left(\frac{1}{K} + \frac{D}{Ee}\right)}} \tag{3}$$

where: ρ – density of the liquid, K – bulk modulus of elasticity of the liquid, E – modulus of elasticity of pipe-wall material, e – thickness of pipe wall.
Eq. (1) represents momentum conservation while Eq. (2) — the mass conservation. For these equations, an initial-boundary problem is formulated. It is solved in the domain: $0 \le x \le L$, $t \ge 0$ (where L — length of pipeline). Appropriate initial and boundary conditions have to be imposed. Namely, at $t = 0$ a steady flow is assumed. It enables to determine the functions $V(x, t = 0)$ and $H(x, t = 0)$ over entire pipeline ($0 \le x \le L$). Since from each boundary (at $x = 0$ and

$x = L$) only one characteristic enters the domain of solution ($C^+ = 1/(V + c)$, $C^- = 1/(V - c)$) one function, $V(t)$ or $H(t)$, should be imposed at these ends.
To solve Eqs (1, 2) the method of characteristics is commonly applied. There are many numerical algorithms of solution basing on this method. Very popular version was proposed in [8]. It is suitable for calculation a fixed grid. The method of characteristics usually uses the linear interpolation to calculate velocity and pressure between the nodes [1, 8]. For this reason it requires to respect the CFL condition for stability and unfortunately usually it produces numerical diffusion. The calculation should be carried out with the Courant number close to unity. This value also minimises the numerical diffusion. However this condition is difficult to fulfil for complex pipes with variable characteristics or for networks in which wave speed varies. Consequently one can face some troubles because of varying Courant number. For this reason many attempts have been made to find suitable methods.
Instead of commonly applied classical method of characteristics, a modified finite element method can be applied. The proposed approach, successfully applied for unsteady flow in open channel and transport equations, seems suitable for water hammer equations as well. The method involves two weighting parameters and it is absolutely stable. It enables to carry out the calculations with varied Courant number and to reduce the numerical errors. The proposed method was used to solve the water hammer equation for pipes in series.

2 Solution of the water hammer equations by the modified finite element method

Modification of the finite element method leads to more general form of algebraic equations approximating the governing equations. The modification deals with process of integration and it holds for the linear basis functions. Its particular cases are the standard finite element method and the well known schemes of the finite difference method.
According to the Galerkin procedure [11], the solution of Eqs. (1, 2) has to satisfy the following condition:

$$\int_0^L \Omega(f_a,\cdots)\mathbf{N}dx = \sum_{k=1}^{M-1} \int_{x_k}^{x_{k+1}} \Omega(f_a,\cdots)\mathbf{N}dx = 0 \tag{4}$$

where: Ω – symbolic representation of Eqs. (1, 2), f_a – approximation of any function $f(x, t)$ occurring in Eqs. (1, 2), $\mathbf{N}$ (x) – vector of linear basis functions, L – length of pipeline, k – index of node, M – number of grid points.

For the linear basis functions, calculation of one integral in expression (4) over an element of length $\Delta x = x_{k+1} - x_k$ is made in following way [9]:

$$\begin{aligned}&\int_{X_k}^{X_{k+1}} \left(\frac{\partial V_c}{\partial t} + \frac{1}{2}\frac{\partial V_a^2}{\partial x} + g\frac{\partial H_a}{\partial x} - \left(\frac{f}{2D}V|V|\right)_c \right) N_k dx = \\ &= \left(\omega \frac{dV_k}{dt} + (1-\omega)\frac{dV_{k+1}}{dt}\right)\frac{\Delta x}{2} + \frac{1}{4}\left(-V_k^2 + V_{k+1}^2\right) + \\ &+ \frac{g}{2}\left(-H_k + H_{k+1}\right) + \frac{\Delta x}{4D}\left(\omega f_k V_k |V_k| + (1-\omega) f_{k+1} V_{k+1} |V_{k+1}|\right)\end{aligned} \tag{5}$$

$$\int_{X_k}^{X_{k+1}} \left(\frac{\partial H_c}{\partial t} + V_c \frac{\partial H_a}{\partial x} + \frac{c^2}{g}\frac{\partial V_a}{\partial x} \right) N_k dx =$$

$$=\left(\omega\frac{dH_k}{dt}+(1-\omega)\frac{dH_{k+1}}{dt}\right)\frac{\Delta x}{2}+\frac{1}{2}(\omega V_k+(1-\omega)V_{k+1})(-H_k+H_{k+1})+\frac{c^2}{2g}(-V_k+V_{k+1}) \quad (6)$$

$$\int_{X_k}^{X_{k+1}}\left(\frac{\partial V_c}{\partial t}+\frac{1}{2}\frac{\partial V_a^2}{\partial x}+g\frac{\partial H_a}{\partial x}+\left(\frac{f}{2D}V|V|\right)_c\right)N_{k+1}dx=$$

$$=\left((1-\omega)\frac{dV_k}{dt}+\omega\frac{dV_{k+1}}{dt}\right)\frac{\Delta x}{2}+\frac{1}{4}\left(-V_k^2+V_{k+1}^2\right)+$$

$$+\frac{g}{2}(-H_k+H_{k+1})+\frac{\Delta x}{4D}\left((1-\omega)f_kV_k|V_k|+\omega f_{k+1}V_{k+1}|V_{k+1}|\right) \quad (7)$$

$$\int_{X_k}^{X_{k+1}}\left(\frac{\partial H_c}{\partial t}+V_c\frac{\partial H_a}{\partial x}+\frac{c^2}{g}\frac{\partial V_a}{\partial x}\right)N_{k+1}dx=$$

$$=\left((1-\omega)\frac{dH_k}{dt}+\omega\frac{dH_{k+1}}{dt}\right)\frac{\Delta x}{2}+\frac{1}{2}((1-\omega)V_k+\omega V_{k+1})(-H_k+H_{k+1})$$

$$+\frac{c^2}{2g}(-V_k+V_{k+1}) \quad (8)$$

were: Δx – distance step, ω – weighting parameter ranging from 0 to 1.

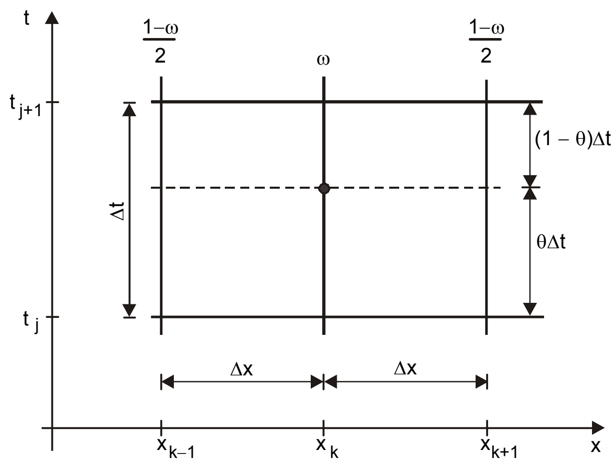

Fig. 1: Numerical grid for the finite element method

The grid points used by the method are presented in Fig. 1. When all integrals in each element are summarised accordingly to Eq. (4), the global system of ordinary differential equations over time is obtained. It has the following form:

$$\mathbf{S}\,\frac{d\mathbf{X}}{dt}+\mathbf{C}\,\mathbf{X}=\mathbf{0} \quad (9)$$

where: **S** – constant and symmetrical matrix, **C** – variable and asymmetrical matrix, $\mathbf{X} = (V_1, H_1, V_2, H_2, ..., V_M, H_M)^T$ – vector of unknowns set up from nodal values of V and H, T – symbol of transposition.

The matrices **S** and **C** have dimensions of $M \times M$ with bandwidth equal to 7. To time integration the following two level method is applied:

$$\mathbf{X}_{j+1} = \mathbf{X}_j + \Delta t \left(\theta \frac{d\mathbf{X}_{j+1}}{dt} + (1-\theta) \frac{d\mathbf{X}_j}{dt} \right) \tag{10}$$

where: θ – weighting parameter ranging from 0 to 1, Δt – time step, j – index of time level.

It leads to the system of non-linear algebraic equations

$$(\mathbf{S}+\Delta t\theta \mathbf{C}_{j+1})\, \mathbf{X}_{j+1} = (\mathbf{S}-\Delta t(1-\theta)\, \mathbf{C}_j)\, \mathbf{X}_j \tag{11}$$

which has to be completed by imposed boundary conditions. They are as follows:

- at the beginning of the pipe (k = 1) a constant piezometric head H_1= const is imposed,
- at the end of the pipe ($k = M$) the velocity of outflowing water can be calculated depending on the gate opening using the approach proposed in [8]; however in this paper it is assumed that the gate is closed immediately then the condition $V_M(t) = 0$ for $t > 0$ can be imposed,
- at a connection of pipelines of different properties, the continuity equation and the energy equation must be satisfied similtaneously.

In the presented method two weighting parameters ω and θ have occurred. Taking ω = 2/3 the finite element method in its standard form is obtained, whereas for ω = 1 this method coincides with the finite difference one. When θ = 1/2 the method becomes the Crank-Nicolson scheme (equivalent to the implicit trapezoidal rule), whereas with θ =1, it is the implicit Euler scheme. To solve the system (11) an iterative method, for example the Newton's one, has to be used. The stability analysis carried out by Neumann method [7] leads to the following conditions:

$$\theta \geq 1/2 \text{ and } \omega \geq 1/2 \tag{12,13}$$

These relations ensure unconditional stability. For properly chosen values of two weighting parameters the applied method becomes more accurate comparing with standard finite element one.

3 Comparison of experimental data and results of calculation

An experimental apparatus for investigating water hammer events in pipelines is shown in Fig. 2. The apparatus is composed of two straight pipelines (1) having different diameters, pressurised tank (2) and ball valve (4) mounted at the end of the pipe. The valve was closed manually by hand. Valve closure time T_c, measured to a thousandth of a second, was from 0.018 to 0.025 s in all tests. The measurements and further analysis of pressure head characteristics $H(t)$ referred to simple water hammer in which the water hammer wave reflection time (pressure wave period) T was always greater than the value closure time.
Pressure was recorded by means of a measuring system consisting of strain gauges (5), extensometer amplifier (6) and a computer (7) with AD/DA (20 MHz) card. The signal from the transducer was sampled with a frequency of 2,000 Hz. The gauges, with measuring ranges up to 1.2 and 2 MPa (with measurement uncertainty ±0.5%) had linear performance characteristics in the whole range of measured pressure with a correlation coefficient R of at least 0.999. Pressure transducers are located at two points along the pipeline (Fig. 2).
The pressure was recorded during the process of water hammer for different velocities V_0 of steady water flow (varied from 0.3 to 2 m/s in the experiments). The velocity V_0 at the end of pipeline was measured by the volumetric method (with measurement uncertainty ±1%).

The tested pipes (1) were fed from a reservoir (2) which itself was filled from the pipe network. The water pressure p_s at the installation was constant (p_s = 0.65±0.01 MPa). The pressure was reduced by means of a pressure reducing valve (3). The stream pressure value p_0 at the pipe outlet and the water flow velocity V_0 were adjusted in order to avoid column separation events at water hammer [3]. The experiments were carried out for pipelines which contained two parts having different diameters. The are described in details in Table 1.

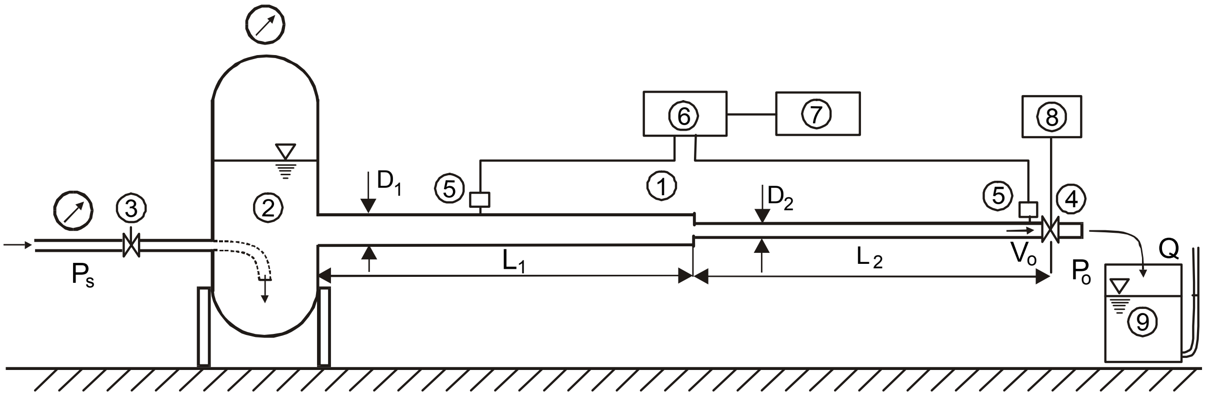

Fig. 2: Experimental installation

Table 1: Pipe characteristics

Parameter	Pipeline 1	Pipeline 2
Length L [m]	26.5	24.5
Inside diameter D [m]	0.042	0.0215
Thickness of pipe wall e [m]	0.003	0.0023
Roughness height k [m]	0.00008	0.00008
Wave's velocity c [m/s]	1328.0	1368.0

After an initial steady state had been established, a transient event was initiated by a rapid valve closure.
In Fig. 3a the graph of pressures measured at the downstream end of second pipeline is presented. These results were obtained for time of closure T_c = 0.019 s, initial pressure head at downstream end of pipe H_o = 48 m and initial velocity at the same point V_o = 0.431 m/s.
The calculations of the water hammer phenomenon were carried out by proposed version of the finite element method for the friction factor f calculated using the Colebrook-White formula [2]. The results obtained for the data which minimised the numerical diffusion with wave celerities defined by Eq. (3) are presented in Fig. 3b. Comparing both graphs one can notice an essential disagreement. From experiment results that frequency of the head oscillations vary with time and after several periods it becomes practically constant. Conversely in the results of calculation no change of frequency is observed.
An analysis of wave frequencies carried out for recorded head oscillations shows that the wave speed strongly vary with time. Namely in presented case it ranged from 1368 m/s to 755 m/s. In Fig. 3c the results obtained for lower value of c are presented. Between the graphs 3a and 3c similar disagreement is observed as well.
Although many available publications concern the unsteady flow in pipelines, unfortunately only few of them touch the studied problem. For example some information concerning an influence of variable pipe's characteristics on head oscillations are given by Mitosek and

Malesinska in [4] and by Mitosek and Chorzelski in [5]. Similarly some suggestions on this problem are presented by Wylie and Stretter while discussing the vibrations of pipe system [10] and water hammer events in system of pipes in series. Generally one can accept that variable pipe characteristics as diameters and materials partially reflect the waves travelling in pipes and consequently an initial wave speed is affected. Unfortunately at this moment an explanation of mentioned phenomenon is unknown. On the other hand it seems that this problem has important practical meaning since it concerns the pipe networks as well. One can suppose that in pipe systems the wave reflections in the pipe junctions change the run of head oscillations comparing with the results obtained by standard analysis in which a constant wave speeds are assumed.

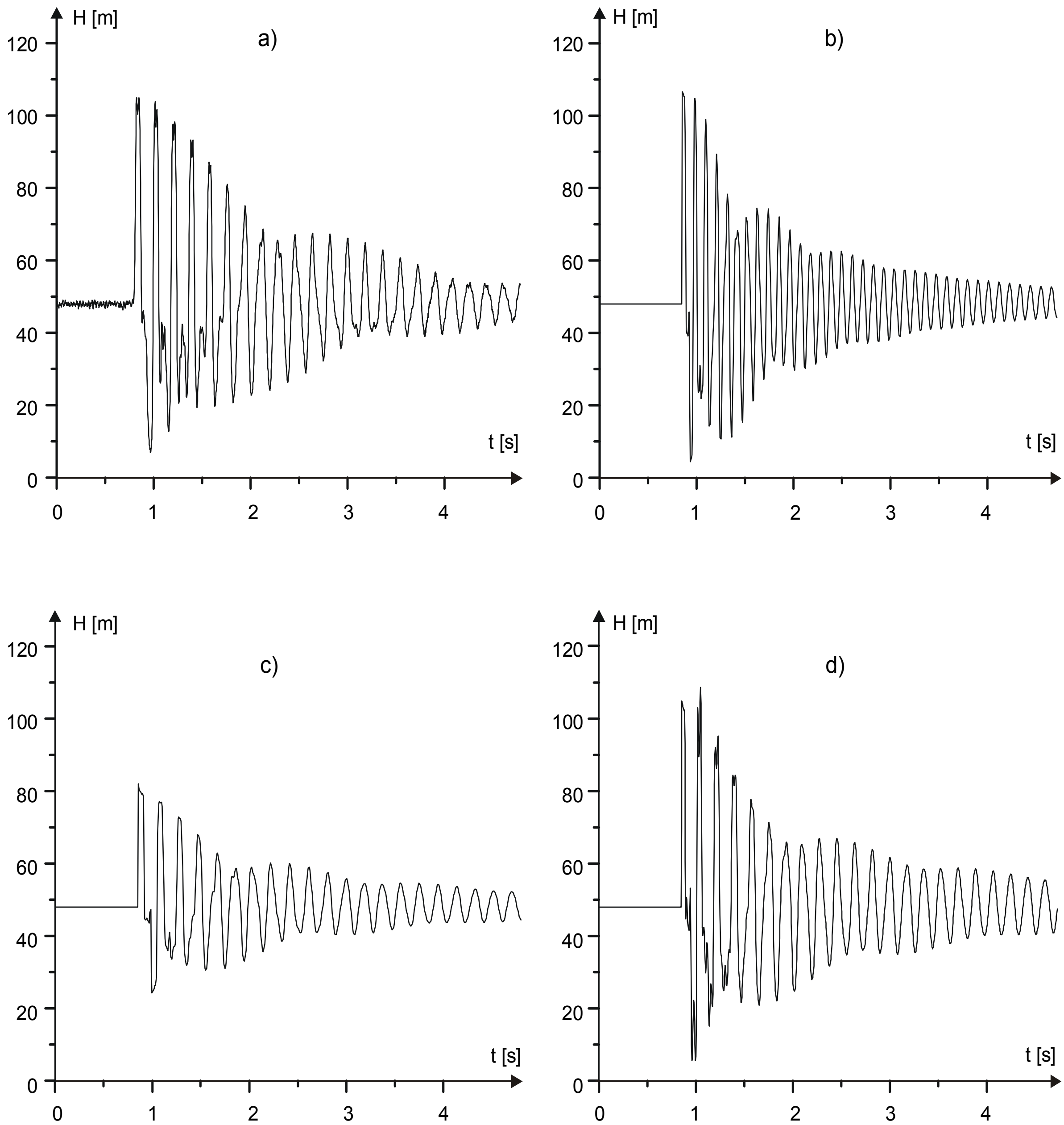

Fig. 3: Observed (a) and computed (b, c, d) head oscillations for two steel pipelines in series at downstream end

The remarkable improvement can be achieved while accepting variable wave celerity. Namely one can take the following formula:

$$c(t) = c_f + (c_o - c_f)\, e^{-Kt} \tag{14}$$

where: c_o – initial wave celerity, c_f – final wave celerity K – coefficient.
It is a very simple manner to take into account this fact. Satisfying agreement between observed and computed head oscillations was obtained for the following set of data: c_o = 1368 m/s, c_f = 775 m/s, K = 5, ω = 0.5, θ = 0.67; Δx = 0.5 m, Δt = 0.0005 s. The Courant number C_r varied from 0.75 to 1.37. Comparing the graphs presented in Fig. 3a and Fig. 3d one can notice that in this case the applied method gives the results which are very similar to the ones provided by physical experiments. Owing to the introducing varied wave speed the evolution of frequency in time was improved. Consequently the results of calculations approached the ones provided by experiments.

4 Conclusions

The experiments carried out using the physical model showed that variable diameter in two pipelines in series remarkably changes the run of water hammer phenomenon comparing with simple pipe having constant diameter. It was confirmed as well as that its reproduction using the standard model of water hammer in form of Eqs. (1, 2) with constant wave celerity is impossible. The head oscillations recorded during the water hammer event indicates that the wave celerity varies and after several periods it takes the value which strongly differs from result given by Eq. (3). Therefore one can suppose that in fact the water hammer in system of pipes has much more complicated character than it is usually assumed.

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Effects From Association Work of Surrounding Rock and Concrete Linings at Tunnels Under Pressure

Assist. M.Sc. B.Sc. Civ. Eng. Darko Moslavac

University "St's Cyril&Methodius", Faculty of Civil Engineering – Skopje

Partizanski odredi 24, P.BOX 560

1000 Skopje, Republic of Macedonia

Email: moslavac@gf.ukim.edu.mk

Abstract

The aim of this paper is to show the effect of association work between surrounding rock material and concrete linings at tunnels under pressure, and to determine the percentage of load which is transferred to the rock. One spillway tunnel is analyzed with diameter D=9.4m and thickness of concrete lining d_c = 70cm. The excavation of tunnel is in rock material category III – favorable rocks, according to Bieniawski classification. Analyses are performed with Finite Element Method implemented in ZSOIL V.4.23. software. The surrounding rock material is modeled with finite elements with nonlinear behavior, using the *Drucker - Prager* criterion. Two analyses are made, first with real strength – deformation parameters for the rock material and second with value of this parameters approximate equal to zero. In the first case, the intensity of tension axial force in the concrete lining is N_{max}= 606KN, and in the second case when the influence of the surrounding rock material is neglected the intensity of the axial force is N_{max}= 932KN. From the results of this analysis it can be concluded that if neglecting of surrounding rock material is made in the calculation, the intensity of tension axial force in the concrete lining is 55% higher.

1. Introduction

The main goal of this paper is to present the effects of association work between surrounding rock material and concrete linings at tunnels under pressure. To be able to make comparison between the results obtained with Finite Element Method (FEM), with the results obtained using Marriot's formulae for calculating axial forces in the tunnel linings, it's necessary to simulate uniform state of stresses around the tunnel lining. To provide this state of stresses two assumptions are made. First assumption, is that the self – weights of the rock material and concrete lining is equal to zero, and the total load of the overburden is acting on the top boundary of the numerical model as uniform distributed load. With this assumption uniform distribution of the normal vertical stresses σ_V on the entire high of the numerical model is provided. Second assumption, is that the value of the coefficient of lateral pressure is equal to one $\lambda = 1$, which provide the condition $\sigma_V = \sigma_H$ to be satisfied.

2. Problem statement

2.1. Geological characteristics of rock material and geometry

One spillway tunnel is analyzed with diameter D=9.4m and thickness of final concrete lining d_c = 70cm. To eliminate the influence of the primary lining made of shotcrete and anchors it is assumed that this lining doesn't exist, in other words, the full load of the overburden is acting on the final concrete lining. The excavation of the spillway tunnel is in rock material category III – favorable rocks, according to Bieniawski classification. For further numerical analysis the following strength – deformation parameters are assumed for the surrounding rock material:

RMR = 54 – 60;

D = 3.0e6 KPa;

C = 350 KPa;

$\varphi = 36^0$;

$\nu = 0.26$;

$\gamma_{sat} = 26.5$ KN/m^3;

2.2. Loads and load cases

Loads which are taken into account in the numerical analysis are as follows:

- Outside load from surrounding rock material saturated with water with intensity $P_n = \gamma_{sat} \cdot h = 26.5 \cdot 34 = 900$ KN/m^2;
- Inside water pressure with intensity P_W = 200 KN/m^2 which acts as radial load with same intensity in all points of the tunnel.

Schematic review of numerical model and loads is shown on Fig. 1.

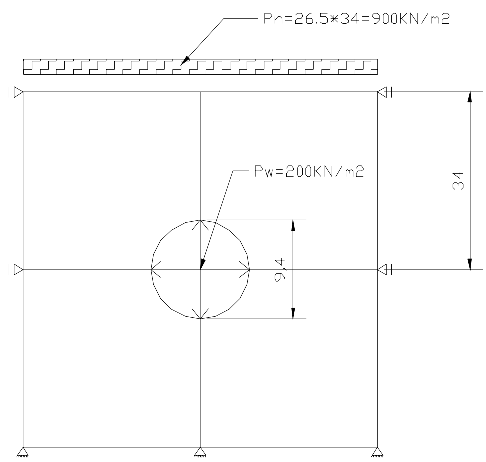

Fig. 1: Schematic review of numerical model and loads

The following load cases are analyzed:

- (LC-1); State of full reservoir and empty spillway tunnel, when only outside load Pn affects the final concrete lining;
- (LC-2); State of full reservoir and spillway tunnel under pressure, case when both loads affects the final concrete lining;

- (LC-3); State of empty reservoir and spillway tunnel under pressure. In this load case only the inside water pressure affects the concrete lining. This load case is characteristic for sections of spillway tunnel located after the grout curtain.
- (LC-4); State of empty reservoir and spillway tunnel under pressure, with strength – deformation parameters of surrounding rock material approximate equal to zero. This state is hypothetical, but it's used in the analysis to calculate the axial forces in the concrete lining with neglecting the association work of surrounding rock.

3. Numerical modeling

For mathematical modeling the Finite Element Method is used, implemented in the computer software ZSOIL. This software use the newest advantages of non – linear techniques of finite elements and plastic modeling of soil and rock materials for solving problems of stability, bearing capacity, excavation of underground structures in stages and other geotechnical problems. ZSOIL offers wide choice of constitutive models for modeling of continua behavior such as: Elastic model, Mohr – Coulomb, Drucker – Prager, Hoek&Brown, Cap – model e.t.c.

Excavation stages of underground structures are simulated with time depended existence functions defined for the elements, which can be removed or added in some specific time. The concept of time is "undimensional", and it can present 1-minute, 1-hour, 1-day e.t.c. Unloading of continua, after excavation, can be controlled by the user with application of "load time functions" which could be defined for any structure element. With this "load time functions" the three – dimensional effects of the vicinity of the front of excavation can be simulated.

Modeling of continua is made with two – dimensional quadrilateral finite elements with 4 nodes with two degrees of freedom (Ux, Uy) in each node. The plasticity behaviour of the rock material is taken into account with Drucker – Prager constitutive model for the finite elements (Fig.2).

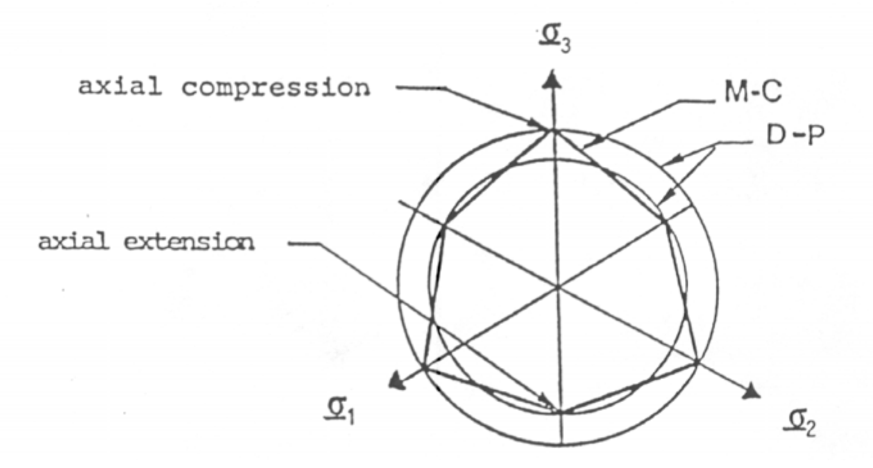

Fig. 2: Deviatory sections of Mohr-Coulomb and Drucker-Prager criteria

A Drucker-Prager criterion is defined in the space of stresses with the following equation:

$$F(\sigma) = a_\phi I_1 + \sqrt{J_2} - k = 0 \tag{1}$$

where I_1 and I_2 are main stress invariants and constants (a and k) could be defined from the geotechnical characteristics of the material (C and ϕ) with the following relations:

$$a = \sin(\phi/3) \tag{2}$$

$$k = C \cdot \cos\phi \tag{3}$$

The whole mathematical model is discretizied with 2000 FE. One part of the finite element mesh is shown on the next figure.

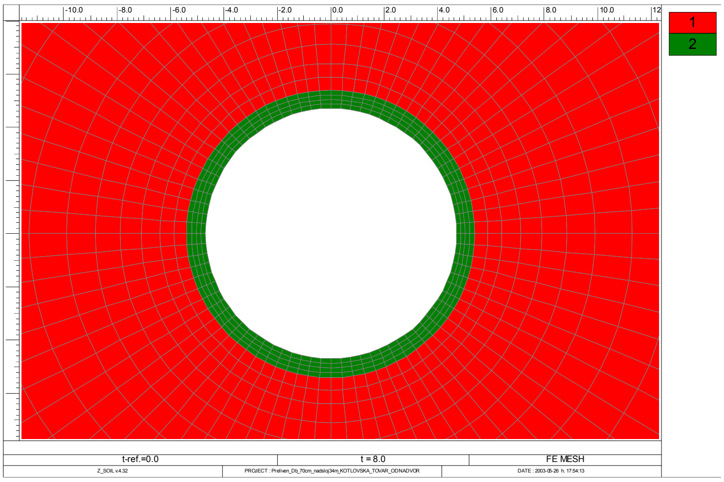

Fig. 3: Numerical model of the spillway tunnel

4. Analysis results

4.1. State of full reservoir and empty spillway tunnel (LC-1)

At this state of reservoir, the influence of surrounding rock material saturated with water is taken into account as outside load Pn = 900 KN/m^2 which is acting on the top boundary of the numerical model as uniform distributed load. Cross – sectional forces in the final concrete lining at this state are shown on figure 3.

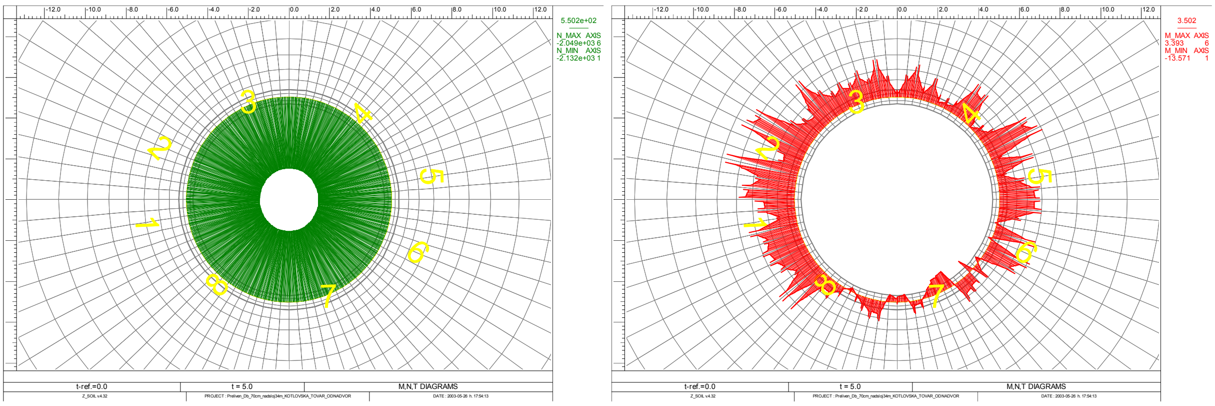

Fig. 3: Diagram of axial forces and bending moments at state (LC-1)

At this state maximum axial force in the final concrete lining is $N1_{max}$ = -2132 KN (comp.). Bending moments in the lining are relatively small (M_{max} = 3.4 KNm; M_{min} = -13.6 KNm), so in the calculation of stress – deformation state and dimensioning of the concrete section they will have very small effects.

4.2. State of full reservoir and spillway tunnel under pressure (LC-2)

This state occur when reservoir floods over the shaft spillway and in this case both loads affects the final concrete lining; Outside load Pn = 900 Kn/m^2 and inside water pressure in the tunnel Pw = 200 KN/m^2. Cross – sectional forces in the final concrete lining at this state are shown on figure 4.

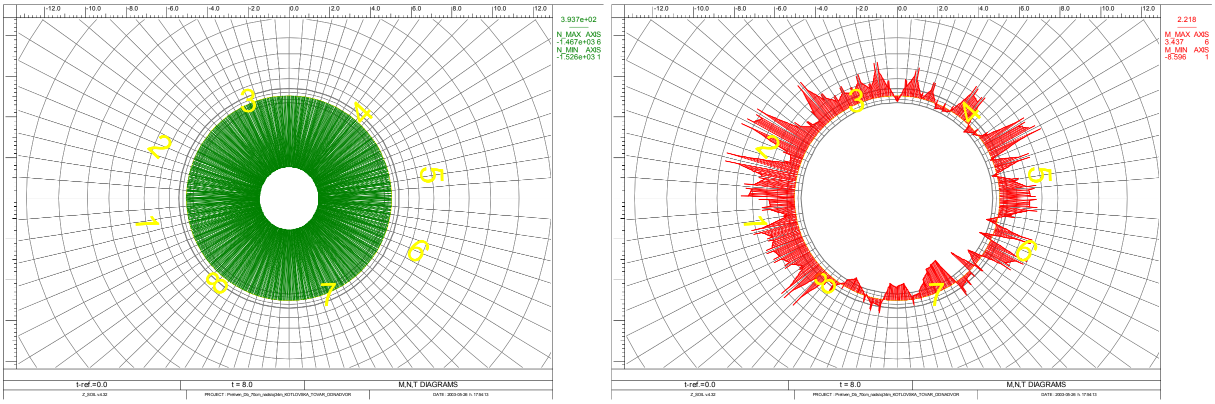

Fig. 4: Diagram of axial forces and bending moments at state (LC-2)

At this state the maximum axial force in the concrete lining is $N2_{max}$ = -2132 KN (comp.). which is smaller then $N1_{max}$, which means that the internal water pressure causes tension axial force with intensity Nz = $N1_{max}$ - $N2_{max}$ = 606 KN. Bending moments are again with small intensities.

4.3. State of empty reservoir and spillway tunnel under pressure (LC-3)

This state is characteristic for sections of spillway tunnel located after the grout curtain when reservoir floods over the shaft spillway. In this load case only the inside water pressure affects the concrete lining. Cross – sectional forces in the final concrete lining at this state are shown on figure 5.

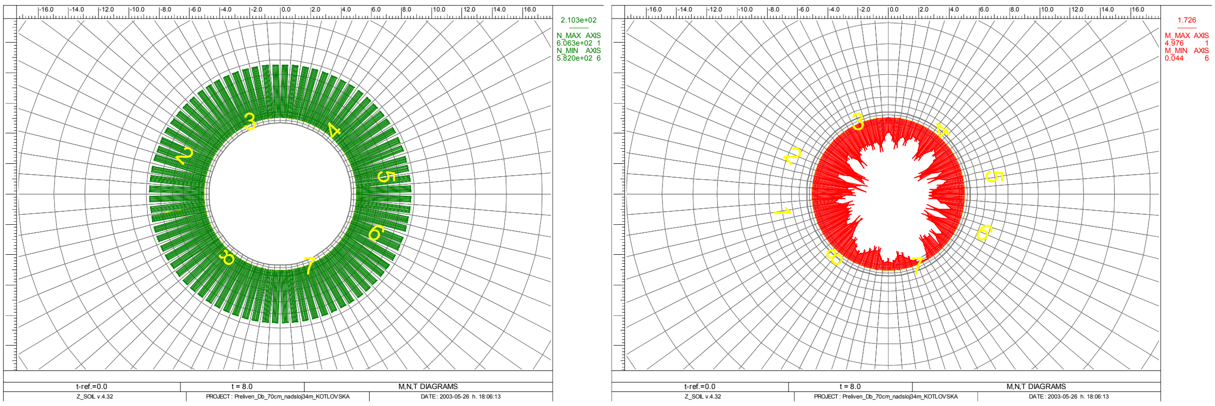

Fig. 5: Diagram of axial forces and bending moments at state (LC-3)

In this load case maximum axial force in concrete lining is tension with intensity Nz = 606KN which is identical result obtained for the tension axial force in the previous LC-2.

4.4. State of empty reservoir and spillway tunnel under pressure, with strength – deformation parameters of surrounding rock material approximate equal to zero (LC-4)

This state is hypothetical, but it's used in the analysis to calculate the influence of water pressure in the spillway tunnel with neglecting the association work of surrounding rock material. This state is simulated with adopting very low values for strength – deformation parameters of the rock material. Cross – sectional forces in the final concrete lining at this state are shown on figure 6.

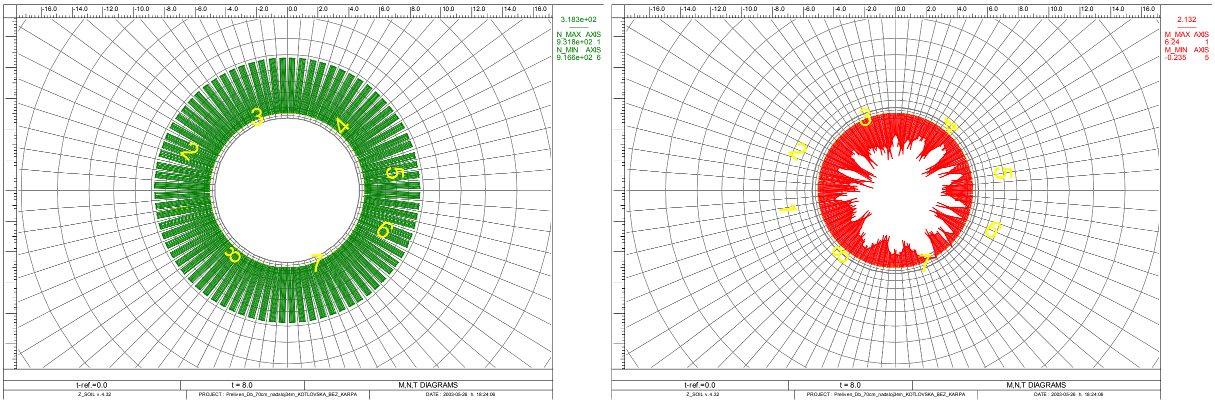

Fig. 6: Diagram of axial forces and bending moments at state (LC-4)

At this state maximum axial force (tension) with intensity Nz = 931.8KN is obtained.

If we calculate the stresses in the concrete lining according to Marriot's formulae we will obtained:

$$\sigma = \frac{Pw \cdot D}{2\delta} = \frac{200 \cdot 9.4}{2 \cdot 0.7} = 1342.9 KN/m2 \tag{4}$$

and for axial force:

$$Nz = \sigma \cdot F = 1342.9 \cdot 0.7 = 940KN \tag{5}$$

This is almost the same result obtained with ZSOIL V. 4.23. (Nz = 931.8KN).

5. Conclusion

Analysis in this paper shows that the influence of surrounding rock material in transfer of water pressure loads is large. Neglecting the association work between surrounding rock material and final concrete lining leads to axial forces with 55% larger intensity (940/606 = 1.55), and of course more expensive structures.

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Váh Navigability

Pavol Obložinský

Slovak University of Technology, Faculty of Civil Engineering, Department of Hydraulic Engineering, Radlinského 11, 813 68 Bratislava, Slovakia. E-mail: oblozin@svf.stuba.sk

Abstract:

This article was written to underline perspective possibilities of networking Slovak rivers /considered waterways/ into international network of inland´s waterway. The article concentrates on Vah waterway, that should be realised from Komárno to Žilina. Its importance, as the possibility to connect Danube with Oder or Wisla, in Poland is unreplaceable. It is the only solution of navigational connection and it is suitable supplement of railway and road route in North-South direction, via valley of Vah /The Amber Route/. It also refers to importance of the port in Bratislava and the direct navigational connection of river Vah to waterway Danube-Mohan-Rhone. Currently the Vah river is not navigable. The European Union has designated the Vah as a missing link in the European waterway systém. The European Agreement on Main Inland Waterways of International Importance (AGN) has identified the Vah as a waterway E81 in the European Inland Waterway Network. The Vah has been classified as class VIa for the lower reach from the Danube (at Komarno) to Sered/Hlohovec and class V from Sered/Hlohovec to Zilina and to the Oder too.
The Vah River flows entirely within the territory of Slovak Republic running down from the Karpaty Mountains and ending up in the river Danube near Komarno , after Danube, it is the most important river in Slovak Republic. The river is mainly used to produce the hydropower energy. Navigation is not possible on most parts of the river, which is due to the lack of passage facilities on the hydropower stations. Presently, there are only two operational locks on the river long 403 km, but interest for navigation has a length of about 250 km, from Zilina downstream to the river Danube Navigation is partly possible in the lower course, limited to the water level in Danube. However, plans exist to make the river navigable with future possible connection to Oder. The paper gives description on the present situation on the Vah River and discusses the pilot project on the Vah River navigability.
Also in my paper I would like to give some information about plans of navigability another Slovak river.

1 Introduction

At present time, the Vah River is mainly used for the production of the hydropower energy. Although through the valley of the Vah River leads an important transportation route, railways and highways, the river is not navigable along most part of its length. A system of hydropower stations, which has been built up on the Vah River during the period of the last decades consisting from 22 power stations and canals, is known as the Vah River Cascades of hydro-plants. Most of the structures are not equipped with the passage facilities allowing the shipping on the river. However, the design of the structures took into consideration the possibilities of future navigation on the Vah River and the positions of the lock chambers were planned and, in some cases, the lock heads have been constructed.
For transport purposes, the Vah has only been used in the past as a means of floating timber rafts downstream from the forest in the upper regions to the lower reaches and Dnube Valley.

This rafting was done on the floods in spring and after summer rains when the river had ample depth for the rafts to pass.
Up to present, only 2 chambers are in operation. There is an intention to make the Vah River navigable along all its length from Komarno to Žilina and to create the waterway, which is denoted as the E81 in the map of the internal waterways. After making the Vah River navigable the canal Vah–Odra could be considered to construct and to complete waterway connection between the Black and Baltic Sea, which is a missing link in the European waterways' network.
The plan of navigability another Slovak rivers is only idea for future.

2 Present condition and perspectives of the Vah River navigability

Catchment area of the Vah River is spread over 34% of the territory of the Slovak Republic. The great part of the industry is concentrated in this region with relatively high density of inhabitants. There are railway tracks and highway leading through the Vah River valley, which are remarkably over-loaded.
Presently, the navigation on the Vah River is excluded from the derivation canals and the navigation in the original bed is limited by the water condition in Danube and by small number of locks. Only recently constructed structures are equipped with locks and lock facilities, which meet the standard of VIa class (Kráľová, Selice) of the European waterway classification system. However, the necessity of the navigation on the Vah River *Fig. 2.1* and possible connection to Odra or Wisla rivers in Poland was recognized. Integration of the planned waterway on the Vah River into the system of European's waterways as Waterway E81 requires providing standard design parameters of locks, shape of basin profile and minimum height under bridges. The waterway will be build up to meet the conditions of Class VIa on the stretch Komárno (influx)-Hlohovec and Class Va on the stretch Hlohovec-Zilina, with considering future possible link to Odra from Zililina as Class Va. The parameters of the waterway are given in Table 1.
Pilot project on the Vah River navigability was carried out in 1998 [2], in which the following reconstruction work on the existing water structures and other structures have been recognized:

- The first (oldest) part of the Vah River Cascades, built up between 1932 and 1949, which consists from a diversion gate weir in Dolne Kockovce and three water power stations in Ladce, Ilava and Dubnica. The plants are equipped with locks (31x7m) for lockage of rafts, which are under reconstruction at the present time. The size of locks is to be extended to 110x12m, the standard of Va class.

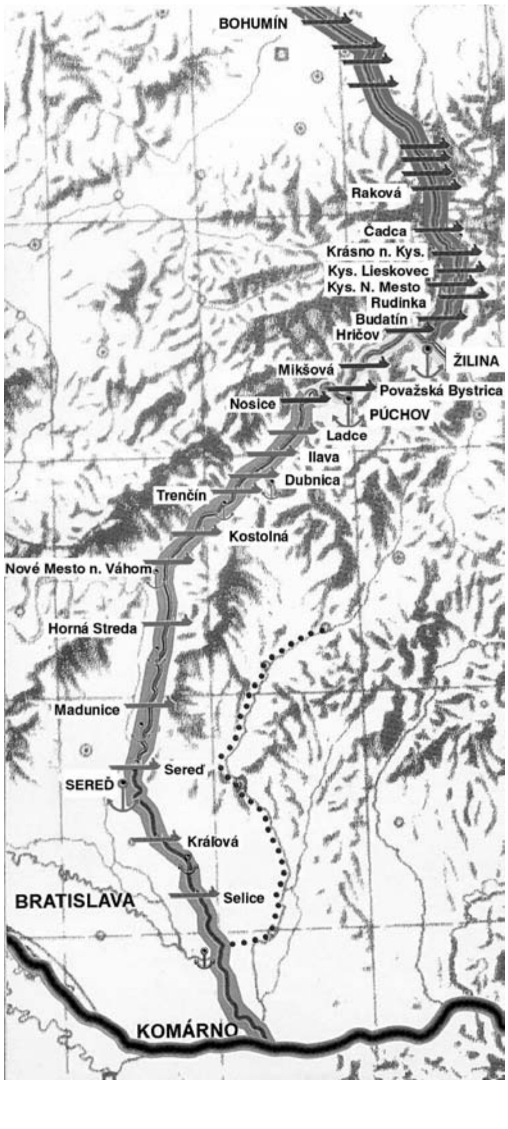

Fig. 2.1 Váh waterway E81

- The second part of the Vah River Cascades (built in 1945-54), which consists from a diversion gate weir in Trencianske Biskupice and three waterpower plants in Kostolna, Nove Mesto and Horna Streda are not equipped with the locks. However, the head of locks are partly constructed on those plants and presently used as the idle outlet. According to the former project, the size of locks was planned to be 85x12.5m. The lock length does not meet the standard of the Va class and locks will be either elongated or the navigation will be provided by newly constructed locks at different location.
- The hydropower plant Trencin (1952-56) is constructed on the waste canal from the Dubnica power station and the discharge is deflated into the reservoir crated behind the diversion gate weir in Trencianske Biskupice (beginning of the first part of Cascades). The partly constructed head of the lock, 85x12.5m, will require the reconstruction and the originally intended length has to be extended.
- The concrete dam in Nosice is equipped with a panel that can be turned into the head of the lock (originally planned size of 85x12.5m). The guide wall of the lock and the port wall have already been constructed. The passage throng dam in Nosice could be possibly provided by a ship lift, which would lead into the considerable saving of the water required for lockage to overcome the head of 23m.
- The water power plant in Madunice is also equipped with partly constructed lock, the design size of which was 85x12.5m. The existing part of the lock will be either reconstructed or new lock will be designed.
- The cascades of the power plants Hricov, Miksova and Povazska Bystrica (1958-63) are not equipped with lockage facilities at all. However, the design of the power stations allow the lock to be adjusted to the existing structures with minor structural changes.
- Recently constructed structures on the Vah River basin, the hydropower plants Kralova (1978-86) and Selice (1993-97) are equipped with fully operational locks of size 110x24m.

At present time the limited navigation on the lower course of the Vah River is possible as far as Sered, the stretch long 89 km from influx to Danube. The navigability is dependent on the water level in Danube. The minimum water level on this stretch can also be partly controlled by the discharge governed from the reservoir in Kralova. To provide navigability at the lower course of the Vah River, on the stretch from the influx to Selice, it will be necessary to construct a lock in Kolarovo, unless the hydropower station in Nagymaros (Hungary) is to be constructed. The scheme of the hydropower plants Gabcikovo-Nagymaros would cause an increase of the water level in Danube basin above the Nagymaros plant, which would also affect the water level on the lover course of the Vah River so that the requirement for the minimum shipping depth would be met.

Table 1. Classification of the Vah River Waterway

Stretch	Class of waterway	Capacity [ton]	Size of lock		Shipping depth [m]	Minimum under-bridge passage [m]
			Length [m]	Width [m]		
Komárno – Hlohovec	VI a	3200 ~ 6000	95 – 110	22,8	2.5 - 4.5	7,0 - 9,1
Hlohovec – Žilina	V a	1600 ~ 3000	95 –110	11,4	2.5 - 4.5	5,25 - 7,0
Žilina – Odra	V a	1600 ~ 3000	95 - 110	11,4	2.5 - 4.5	5,25 - 7,0

The pilot project on the Vah River navigability [2] considers that the shipping from Komárno to Zilina will be provided in:

- original basin of the river; total length of 27km
- reservoirs; total length 83km
- derivation canals; total length 130km

The total length of the waterway from Komárno to Zilina will be 240km

Ports and port facilities are not constructed on the Vah River at all. There ate two port walls in Sala and Soporno mostly used as the re-loading station for transportation of agricultural products. The pilot project on the Vah River navigability suggests constructing 13 ports in Kolarovo, Nove Zamky, Trnovec, Sered, Hlohovec, Piestany, Nove Mesto, Trencin, Dubnica, Puchov, Povazska Bystrica and Zilina. Locations of ports allow the connection on the road and railroad network.

The execution of the project of the Vah River navigability is suggested to be done in four stages:

1. stage stretch Komarno-Sered, length of 89km. The lock located in Selice provides limited navigation on this stretch (lower course of the river). The shipping conditions directly depend on the height of the water level in Danube. The navigability of this stretch can be also provided by the affected discharge from the upper water structures. However, the construction of new lock is required in order to provide the full-year navigability of this stretch in the case that the water power plant in Nagymaros is not to be finish.
2. stage Sered-Puchov, length of 120km. To navigate the stretch Sered-Puchov means to reconstruct existing structures, which consists mainly in constructing locks and lock facilities on hydropower plants in Madunice, Horna Streda, Nove Mesto, Kostolna, Trencin, Dubnica, Ilava and Ladce
3. stage stretch Puchov-Zilina, length of 40km. Construction of new locks in Nosice (ship lift), Povazska Bystrica, Miksova and Hricov.
4. stage link to Odra from Zilina, length about 89 km. The link from Zilina to Odra is suggested to be done by canalizing rivers Kysuca and Ciernanky and constructing locks and ship lifts up to the reservoir in Jablunkovsky valley, and canalizing Olsa and Bohumin, which influxes into Oder.

The required construction time of the Vah River navigability is estimated to be 20 years. The scenarios have been divided into three phases:

- phase 1: the stretch Komarno – Hlohovec,
- phase 2: the stretch Hlohovec – Zilina,
- phase 3: the stretch Zilina - Oder

Table 2. Time planning

	Year																			
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**
Phase 1																				
Phase 2																				
Phase 3																				

There are no ports on the Váh now and twelve locations of ports are being considered as a possible location for minor port facilities. These locations are in towns – Komarno, Nove Zamky, Sal, Sered, Hlohovec, Piestany, Nove Mesto nad Vahom, Trencin, Dubnica, Puchov, Povazska Bystrica a Zilina.

In general feasibility calculations take into account cost estimates on a current basis, they exclude the future impact of inflation. The cost estimate has been broken in the following six typical categories – locks/dams, bridges, regulation works, pipe lines, ports and other structures. Investment cost of connection Komarno – Zilina is 824.000,000 US$. The price is on basis of an index 1,66 compared to current prices. Investment cost of connection Zilina - Oder is 3292.650,000 US$. Cost per km for Komarno - Zilina is 3.6 million US$ and Zilina – Oder 23.4 million US$.

3 Transportation capacity of the Vah River waterway.

The transportation capacity of the Vah River waterway is the main factor determining the economical assessment of the cost and the return of investment. The capacity of locks has a direct influence on the total transportation capacity of the waterway. Because the navigation on the Vah River is not the only purpose of the river utilization, at present it is the hydropower production, it will be necessary to perform a complex assessment on the river management taking into the consideration the requirements of all users. The navigation will be provided in the derivation canals of the hydropower plants, what will results in higher requirements on the safety of the ship operation during the lockage. Those two facts, lockage time and ship operation close to the hydropower stations, will have considerable influence on the overall transportation capacity of the waterway and they have to be, therefore, carefully examined.
Table 3 gives information on the time requirement for one cycle of the lockage and calculated capacity of transportation for 24, 8 and 12 hours operational time of the lock.
This calculation shows that for presently required transportation capacity of $3.5x10^6$ ton/year the capacities of lock are satisfactory even at 8 hours operation time of locks. However, the transportation capacity of the water way is estimated to be $10x10^6$ ton/year after

Table 3. Transportation capacity of locks on the Vah River waterway

No.	Lock	Time of one filling/emptying cycle t_n [hour]	Transportation capacity at operation time of lock		
			operation **24** h/d [10^6 t/ year]	operation **8** h/d [10^6 t/ year]	operation **12** h/d [10^6 t/ year]
1	Kolárovo	0,263	213,17	17,76	26,64
2	Selice	0,287	195,35	16,28	24,42
3	Kráľová	0,452	124,03	10,33	15,49
4	Sereď	0,535	104,79	8,73	13,09
5	Madunice	0,497	56,40	4,70	7,05
6	H. Streda	0,498	56,29	4,69	7,03
7	N. Mesto	0,471	59,51	4,95	7,42
8	Kostolná	0,466	60,15	5,01	7,51
9	Trenčín	0,371	75,56	6,29	9,43
10	Dubnica	0,404	69,38	5,78	8,67
11	Ilava	0,400	70,08	5,84	8,76
12	Ladce	0,405	69,21	5,77	8,65
13	Nosice *	0,593	47,27	3,94	5,91
14	P.Bystrica	0,417	67,22	5,60	8,40
15	Mikšova *	0,623	44,99	3,75	5,62
16	Hričov	0,334	83,93	6,99	10,48

* critical condition

creating the connection to Oder, which might be extended to 20 ~ 30x10^6 ton/year after fully navigable passage Danube-Oder-Baltic Sea. In this case, the capacity of some locks will not be satisfactory enough and the lock in Nosice and Miksova will be the critical ones. On those two locks, and probably on locks in Madunice, Nove Mesto and Horna Streda, the time of filling/emptying cycle appears to be too high and the future technical measures to decrease the time of the cycle are recommended. This could be achieved either by using more sophisticated filling/emptying systems or alternative solution will have to replace the lock, like the ship lift (recommended on structure Nosice) or ship-railways passage.
There is only one location where possibly the waiting of vessels for the lockage might occur, in the 3.5 long stretch Kolarovo-Selice, however this should not have any serious impact on the total capacity of the waterway.

Projections for Slovak traffic is assesses for two of the main opportunities for Vah waterway as:

- the handle traffic generated by existing industries in the Vah regions is 250,000 t/year,
- the handle traffic which will be generated by future economic growth in those regions, it is 2,700,000 t/year.

Projections for transit traffic presents the assessment of the potential for transit traffic and presents an opportunity for Vah waterway. It has proved very difficult to obtain consistent data in future, however, there is no doubt that there are substantial levels of transit traffic passing trough. Now it can be about 500,000 – 1,000,000 t/year.
The contribution of the Vah waterway and its connections to Oder is characterized by:

- restructuring of trade relationships towards greater trade between eastern and weastern Europe,
- possible European integration on a continental scale
- continued physical development of the inland waterway network,
- environment concerns which will suppress of road haulage.

The river Vah can be developed in phases, and each phase will progressively make the Vah more attractive and able to compete in the transport market.

4 Global effects of the Vah River navigation

In order to present an assessment on the global effects of the Vah River navigability on the infrastructure, it is necessary to consider a wide range of factors, not only to consider the standpoint of water management or transportation measures. Prime contributions of building up the water way are with two maim frameworks:

- primary - water management and transportation

  The contribution of the water management factor should be viewed in more complex solution of the water supply of all surrounded area for different users and in general protection against flooding. From the transportation point of view, the waterway will be integrated into the system of the railways and roads transportation through the Vah River valley, decreasing the transportation cost of certain products and unloading the busy railroads and highways. Great contribution of the Vah River waterway is also in creating of the possible link to Oder and thus the link Baltic-Black Sea.

- secondary – impact on ecology, waterpower supply, tourism, recreation, water sports, fishing, local economical development and employment

The water transportation is one of the cleanest kind of the transportation from the ecological point of view. The navigability of the Vah River will involve the program of revitalization of

the river, reservoirs, canals and surrounding areas, which will have a positive effect on the ecology of the territory along the river. By constructing the hydropower station (in Sered) all the potential of the river will entirely utilized. The project will also have a considerable influence on the social sphere by creating new working opportunities and influence on the tourist shipping and recreation.
Economic cost benefit analysis [2] has been identified and appraised bz the following economic costs and benefits:

- capital costs have been adjusted to exclude the application of taxes which are transfer items distorting the economic cost within the Slovak economy,
- Benefits from economic activity induced by making the Vah navigable, these will accrue to the Slovak economy,
- the economic impact of tourism stimulated by the waterway, also accruing to the Slovak economy
- benefits in savings in transport costs due to lower per tonne mile costs of inland waterway transport, compared to road and rail transport,
- saving in pollution. These benefits accrue principally to Slovakia, but also to Europe at large,
- increased safety, with benefits to various European countries.

The results as economic rates of return, after phase 2 are 5,9% - 10,8% accrue to the Slovak economy and 8,6% - 15,1% accrue to the other European economies.

5 The others Slovak rivers

It is necessary to consider the navigability of the rest of Slovakian rivers and possibility for direct linking up Slovakian flows with international network of waterways.In case that river Morava will be navigable, there will be facilities to transport goods from Czech Republic on river Danube. In connection with this, it is necessary to talk about possibility of canalisation of river Maly Dunaj to Kolarovo.This could make the journey much shorter.
Canalisation of river Nitra could mean a direct connection with Vah waterway near Kolarovo and Danube near Komarno.

Fig. 5.1 Plan of Slovak waterways

Navigability of two rivers Hron and Ipel, both flow into river Danube, would mean linking up part of central Slovakia by waterway with Danube.
In case that rivers Hornad, Bodrog, Laborec and Latorica in east part of Slovakia would be navigable, it could create the link by Tisa waterway with Danube.
Navigability of these flows would create /besides Danube waterway/ a network of waterways roughly 900 km long predominantly in Nort-South direction. It was considered navigational connection of West with East /Bratislava-Kosice/ by rivers Danube-Ipel-Rimava-Slana-Bodva-Hornad.
According to real present day possibilities it is considered not only canalisation of river Vah, but also developing of Vah waterway and canalisation of rivers Morava and Nitra. Canalisation of the rest of Slovak rivers is more-less in the study, with no specified date of realisation.

5 Conclusions.

The paper gives detailed description on the present situation on the Vah River and discusses the pilot project on the river navigability, which was carried out in 1998. The analyses of the transportation capacity of the waterway pointed out that the design of the filling/emptying system on the critical locks should be performed with grate care, otherwise those locks would require to be reconstructed in the future due to low transportation capacity. It is recommended to consider the future demand on the transportation and design the waterway in such way that the life time would be from 50 to 100 years from the point of view of the transportation capacity. The waterway was designed to meet the condition of the class VIa and Va, considering the possible connection to Oder by canalizing the rivers Kysuca and Ciernanky on the class Va.
The contribution of the Vah River navigability is not only in the transportation benefits and creating possibilities to connect Baltic and Black Seas, but also in the ecological benefits, which are accompanied with the river and canals revitalization, and in social benefits by creating new working opportunities and the expected development in the water tourism and recreation.

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Croatian Experience in Exploitation of Hydrological Calculations in a Road Design Practise

Nevenka Ozanic[a], Aleksandra Deluka-Tibljas[b], Barbara Karleusa[c]

[a] University of Rijeka, Faculty of Civil Engineering, V. C. Emina 5, 51000 Rijeka, Croatia, nozanic@gradri.hr
[b] University of Rijeka, Faculty of Civil Engineering, V. C. Emina 5, 51000 Rijeka, Croatia, deluka@gradri.hr
[c] University of Rijeka, Faculty of Civil Engineering, V. C. Emina 5, 51000 Rijeka, Croatia, barbara.karleusa@gradri.hr

Abstract
Intensive building of road network started in Croatia in last few years and lots of roads are designed and build yet. Calculations, design and construction of drainage structures are an important part of road construction even not always recognized. There is no doubt that hydrological calculation of drainage constructions are essential elements for complete road design, but still in everyday design very often marginalized. Correct choice of hydrological parameters has an important influence on appropriate functioning of drainage constructions and also there is significant influence on the road economy (maintenance costs etc.) and traffic safety.
Present paper gives a brief comparative analysis of precipitation intensities from different Croatian areas, as well as analysis of their practical interpretation. Besides that this paper contains an example analysis for presentation of economic aspect of correct drainage constructions building.

Key words: road design and construction in Croatia, drainage constructions, hydrological calculations, calculation mistakes, road safety and durability

1 Introduction

Croatia is a country of only 56.538 km^2 but with very different geographical and climatic characteristics (Fig. 1). One of the confirmations of such diversity is the average annual precipitation amount ranged from 650 to 4000 mm in different parts of country.
Building of many highways and two-lane roads started in Croatia in last years and lots of roads are designed yet. In this situation every detail in road design becomes important if it is helpful for decreasing of road building costs. If we know that approximately one of four (or five) dollars in USA and one of seven (or eight) dollars in Croatia (data collected unofficially from road-designers) are spent for drainage structures the importance of properly designed drainage structures becomes obvious.
Hydrological calculations of drainage constructions are essential elements for complete road design. In Croatian everyday practice, however, projecting and building of drainage system remains of marginal significance and it is very often calculated with inaccurately definited parameters.

Fig. 1: Location map of Croatia

Calculations with imprecise parameters results in uneconomic solutions:
- if the system is over designed: in unnecessary high building costs and
- if the system is low designed: in high maintaining costs and costs for rebuilding of drainage structures during road life cycle,

Poorly projected drainage solutions are very often the main reason for distresses on road structures (embankment, pavement…). It can also effect negatively on traffic safety causing hydroplaning, for example. Both problems are noticeable in Croatia today.
The statistic data show that during year 2001 16% of all road accidents happened during rain precipitation and 30% of that number of accidents happened during winter, which is rainy part of season[1].
About 95% of all roads in Croatia have flexible pavement and distresses, which can be caused or accelerated by presence of water in pavement structure, are well known (depression, stripping, rutting etc.) and very common on Croatian roads. Integrated data about road conditions and distresses (caused by water) in general does not exist so as an indicator of road condition in Croatia we can mention data about roads in one of the most developed regions, the region of Zagreb. Collected data show that about 50 % of all roads in region of Zagreb have distresses that can be caused by the presence of water in pavement (potholes, depression lane-to-shoulder drop-off etc.) and we can only presume that the situation is very similar in other parts of the country[2].

2 Analyses

2.1 Basic Elements of Hydrological Calculations

The main purpose of this paper is not to present new hydrological calculation methods or to recommend any of already existing methods as the best to use, but to point out basic methodological errors that appear in calculations of water drainage in urban areas. The discussion in this paper is conducted according to the very simple equation that is still in use for calculation of maximal runoffs - so called rationale Lloyd-Davies method[3] This is an old and very simple, but still relatively acceptable method. The calculation of peak value of maximal runoff (Q) for selected returning period (p) is performed by the equation:

$$Q = 0{,}278\ C\ i\ A \qquad (1)$$

where:
Q - maximal runoff defined returning period (m^3/s),
C - rational coefficient,
i - precipitation intensity (mm/hour) defined duration and returning period,
A - surface of catchment area (km^2),
0,278 - factor of correction.
During the determination of parameters for the equation (1) it is necessary to decide the degree of protection for a desired object expressed as the probability of a maximal flow through appearance, e.g. its returning period. That can be done using precipitation intensity of selected returning period. For the selection of proper precipitation intensity, besides its returning period it is necessary to calculate the time of concentration (T_c) for each part of road, according to the rational method equation:

$$T_c = T_k + T_s + T_g \qquad (2)$$

where:
T_k - the surface concentration time (s),
T_s - the time of flow through secondary canals (s),
T_g - the time of flow through main canals (s).
The determination of previously mentioned flow through and surface concentration times can be performed using proper equations and numerous tables and nomograms, so further commentary of these methodologies is not necessary.

2.2 Mistakes in the determination of time of concentration
The mistake that is most frequently done in hydrological calculations for road drainage is that constant values of precipitation durations are considered and the time of concentration is not calculated at all[4]. Such hydrological calculation considers only previously adopted values of precipitation intensity, catchment area and runoff coefficients but disregards the physics of runoff in urban area.
Therefore in certain regions in Croatia, and probably elsewhere, it is usual to consider 20-minutes precipitation duration for all the analysed profiles. Such calculations result in fact that the drainage systems with shorter time of concentration are as a rule under-dimensioned and others with longer concentration time are over-dimensioned.
In Tab. 1 maximal precipitation intensity values for 10 years returning period for station Rijeka for the duration from 10 minutes (410 l/s/ha) to 2 hours (134 l/s/ha) are represented. It can be noticed that calculated intensities for different durations differ significantly from that calculated for previously mentioned 20-minutes duration time (300 l/s/ha). The differences are expressed in percents and it can be seen that the values are in range from -27% for 10-minutes period to +124% for 2-hours period. It is logically to presume that maximal flow through calculation error would be in the same range if only 20-minutes precipitation intensity would be considered in these calculations as a constant value.

Duration	**10’**	**15’**	**20’**	**30’**	**40’**	**1 h**	**2 h**
l/s/ha	410	342	300	251	220	184	134
Difference (%)	-27	-12	0	+20	+36	+63	+124

Tab. 1: The comparison between 20-minutes precipitation intensity with the duration ranged from 10 minutes to 24 hours (Rijeka - 10 years returning period)

2.3 Mistakes during precipitation intensity determination
Calculation of precipitation water drainage usually includes methods that include the

knowledge of characteristics of short-termed intensive precipitation rainfall regime[4]. Such analyses are usually based on DDP (depth of rainfall, duration, returning period) or IDP (rainfall intensity, duration, returning period) curves that are not always available to the drainage designers and sometimes the designers are not even interested for their availability. Their values on Croatian territory are very variable hydrological parameters (Tab. 2).
On the contrary, some designers avoid using the data regarding maximal precipitation intensities for calculations of maximal runoff quantities. Maximal flows through are therefore, sometimes, determined considering for example annual rainfalls quantities. Such calculations are certainly unacceptable.

Duration (minutes)	RETURNING PERIOD 2 YEARS				RETURNING PERIOD 100 YEARS			
	Bjelovar	Zagreb	Rijeka	Split	Bjelovar	Zagreb	Rijeka	Split
10	321	163	348	218	498	403	457	355
20	189	115	232	164	293	289	378	332
30	139	94	183	121	215	233	340	305
60	82	61	122	79	127	153	283	198
120	48	36	76	45	63	77	141	99
Number of years	*	82	28	24	*	82	28	24

* - Adopted analysis results without information regarding the duration of analyzed number of years

Tab. 2: The comparison of precipitation intensity data (l/s/ha) in Croatia

An analysis of data obtained from 10 pluviographs located in wider region of Zagreb in Croatia[5] show that there are no statistical connections between annual precipitation quantities and maximal precipitation intensities of any duration within analyzed range from 10 minutes to 24 hours. Correlation coefficients obtained in that study suggest weak statistical connection of these parameters only with 24 hours data. Practically, the durations of intensive precipitations that are useful for the determination of maximal flows through are much shorter than 24 hours. Just for illustration in Tab. 3 average coefficients of correlation obtained by previously mentioned analysis and coefficients for analyzed station Zagreb-Gric in a series of data from 71 years are summarized.

Duration	**10'**	**20'**	**30'**	**40'**	**1 h**	**2 h**	**4 h**	**8 h**	**12 h**	**24 h**
Zagreb-Gric	0,10	0,15	0,18	0,19	0,18	0,08	0,16	0,23	0,36	0,47
Average	0,32	0,30	0,31	0,30	0,28	0,20	0,23	0,25	0,35	0,45

Tab. 3: Correlation coefficients between maximal short-termed intensities and annual precipitation quantities

Besides that, in designer's practice, it is almost usual to get the data regarding the characteristics of maximal precipitation intensity by extrapolating from DDP curves these data that are out of the range of analyzed input parameters. Therefore, if ombrographic data regarding the real characteristics of short-termed intensive precipitations are not available they are usually extrapolated from the daily data or even from data for more than one day. Such calculations can result in great discrepancies between the real and calculated precipitation characteristics (Tab. 4).
Such analyses can have a great influence on technical realization of drainage systems that are, therefore, frequently under dimensioned (if shorter precipitation duration data are used) or

over dimensioned (if longer precipitation duration data are used).

Duration	H_h-according DDP curve	H_e-from daily precipitation data	H_e/H_h
	(mm)	(mm)	(%)
10'	24,6	42,3	243
20'	36,0	50,9	175
30'	45,1	56,7	144
40'	52,8	61,3	125
1h	66,1	68,3	103
2h	96,1	82,2	74
6h	125,0	110,2	68
12h	147,3	132,6	70
24h	173,7	156,2	72

Tab. 4: The values of maximal precipitation intensities according to DDP curve extrapolated from daily data (Rijeka - 10 year returning period)

These previously mentioned examples of mistakes that result from using the data regarding the characteristics of short-termed intensive rainfalls are not the only ones that appear during hydrological calculations of drainage systems, but they are certainly the most frequent ones. There are certainly other mistakes that include primary analysis of ombrographic data and the defining of DDP curves, but such analyses are not usually performed by rainfall water drainage system designers and therefore they should be discussed elsewhere.

2.4 Mistakes in the determination of rational runoff coefficient

A very frequent mistake during the calculation of maximal flows through is also a disregarding of precipitation's infiltration variability and consequently disregarding of the fact that function of effective precipitation is non-linear. Uncritical use of usually older methodologies for maximal flows through calculations is to adopt constant average runoff coefficients and consequently maximal discharge is functionally connected with maximal precipitation quantities for certain returning periods.

As an example for unacceptability of such calculation principals in Tab. 5 maximal precipitation values for Rijeka for the duration of 20 minutes[6] are presented. In Tab. 5 are also presented the respective calculated values of effective precipitation by SCS method that takes into account the non-linear character of infiltration during rainfall precipitation and consists of values for meadow ground and for four analyzed soil groups (A-D). From such calculated values of effective precipitation and respective runoff coefficients can be noticed that the infiltration possibility influences significantly on the quantity of effective precipitation and consequently on maximal runoff and flow through values.

Returning period	H	A		B		C		D	
	(mm)	Pf	K	Pf	K	Pf	K	Pf	K
2 years	27,8	11,5	0,41	20,7	0,78	23,4	0,84	25,1	0,90
10 years	36,0	18,0	0,50	28,6	0,79	31,5	0,88	32,8	0,91
100 years	45,4	25,9	0,57	37,7	0,83	40,8	0,90	42,2	0,93

Tab. 5: The comparison of rainfall and effective precipitations

2.5 Mistakes during catchment area surface determination

These mistakes are directly connected with previously mentioned mistakes of runoff

coefficient determination and disregarding the non-linear relation between rainfalls and runoffs. The catchment area surface determination is a task that is independent of the methodology of drainage calculations and it should not be a problem at all[7]. But the determination of catchment area boundaries of influentially areas is often followed by mistakes. The real surface of catchment area is often diminished, sometimes even considerably. Extreme rainfalls in such cases cause floodwaters considerably more than it can be predicted.
Extreme rainfall appearance of less frequent returning periods causes surface flow on both, the urbanised areas and other still not urbanised gravitating surfaces within the catchment area. Such conditions activate the complete catchment area, and such possibilities are often disregarded during calculations[8]. The mode of evacuation of water exceeding the capacity of closed canal drainage system is often not even considered.
The analyses of the degree of building up in urban areas and of the functioning of drainage systems in such extreme situations are frequently absent. Namely, building up of urban areas and roads changes the natural water running conditions. In such conditions of extreme precipitation, the roads collect surface running water and therefore, sometimes even independently from closed drainage system, contribute to the formation of extreme flows through on locations where, in natural state, high waters were not gravitating in such a degree.

3 Results

In this chapter the comparison of water drainage calculation results of one part of the road in the region of Rijeka according to rational method (Tab. 6) and its simplified modification, that is usual in our designing practice, was performed. The simplification adopts the 20-minute intensity as unique for all the profiles (regardless the calculated time of concentration) (Tab. 7). The chosen returning period is 10 years and the data of IDP curve were obtained from Rijeka pluviograph station.

Profile	**Chainage L (m)**	**Catchment area F (km^2)**	**Time of concentration tc (min)**	**Precipitation intensity (l/s/ha)**	**Max. discharge Q (m^3/s)**	**Diameter of pipe Φ (mm)**	**Total expenses of drainage (1000 EUR)**
1	350	0.90	5	542	0.42	500	58,65
2	1100	2.84	12	373	0.80	600	208,65
3	2300	5.93	21	294	1.20	700	502,65
4	2800	7.22	28	260	1.30	800	652,65

Tab. 6: Drainage calculation elements according to rational method

The results presented in Fig. 2 and Tab. 6 show that the rational method predicts real changes of precipitation intensity for different profiles e.g. for their catchment areas. For the duration of 20 minutes real intensities are higher than adopted in simplified calculation method and for longer periods the values decrease.
In Fig. 3 calculated values of maximal discharges according to both methods are presented. It can be noticed that the drainage system is under dimensioned for its part with calculated time of concentration of 20 minutes e.g. in analyzed case approximately near profile 3 that value is 21 minutes.

For longer durations e.g. for the remaining part the drainage system is over dimensioned. The differences in calculated values are significant and are presented in the last column of the Tab. 7.

Profile	Chainage L (m)	Catchment area F (km^2)	Time of concentration tc (min)	Precipitation intensity (l/s/ha)	Max. discharge Q (m^3/s)	Diameter of pipe Φ (mm)	Total expenses of drainage (1000 EUR)	Difference of Q in relationship on Tab. 6
1	350	0.90	Adopted without calculation – 20 minutes	20 – min. (300 l/s/ha) - adopted for all tc	0.23	400	58,65	-45.2 %
2	1100	2.84			0.65	600	208,65	-18.8 %
3	2300	5.93			1.23	700	502,65	+2.5 %
4	2800	7.22			1.49	900	652,65	+14.6 %

Tab. 7: Drainage calculation elements according to usual simplified methodology – for 20 minutes intensity

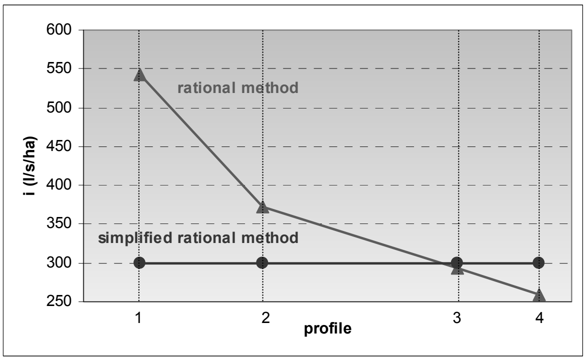

Fig. 2: The comparison of precipitation intensity

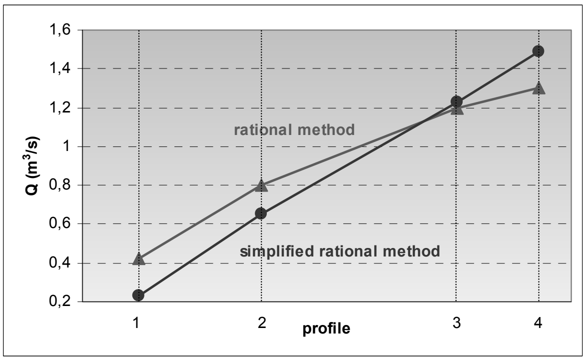

Fig. 3: The comparison of maximal discharge calculated values

In Fig. 4 the costs of the drainage system construction for the analyzed part of the road are comparatively presented. It can be seen that very close to the profile 3 (that is characterized by the time of concentration of nearly 20 minutes that is also the value of the adopted constant precipitation intensity according to the simplified method) the costs of the drainage system construction are higher if the rational method is used. For the longer times of concentration total costs of drainage system construction according to simplified method are greater because system is over dimensioned[5].

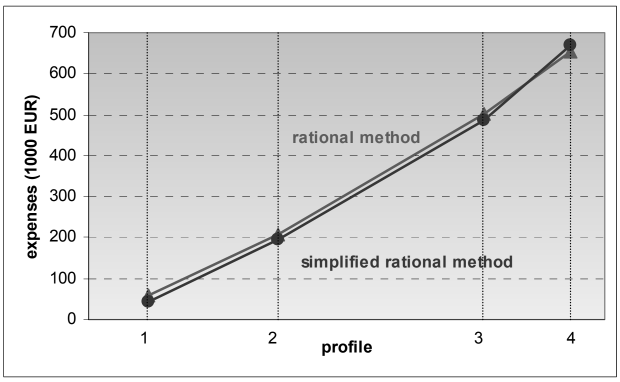

Fig. 4: The comparison of drainage costs

4 Conclusions

Designing and building of road network (highways and other roads) is in process in many developing countries in Europe and in Croatia especially so this paper and its conclusions are an effort in improving the important segment of road design practice – drainage structure design.

This paper shows that uncritical use of the simplified rational method with constant chosen precipitation intensity, that is frequent in designing practice of road drainage systems, leads to wrong results of maximal discharge calculations. For shorter times of concentration than chosen ones the system is under dimensioned and for longer the system is over dimensioned.

With such methodologically wrong approach, besides incorrect technical decision, costs of the system construction are also questionable because for longer duration than the chosen time of concentration overall costs are greater too. Therefore is necessary to accept the correct use of hydrological calculations in designing practice of the road drainage system dimensioning.

It is necessary to realised that properly designed drainage road system is an important factor in road design practice that influences pavement quality and serviceability and that properly designed road drainage system saves money during construction and exploitation of road.

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Investigation Vertical Water Balance Alluvium of the River Drava

Prof.dr.sc. Vladimir Patrcevic, Mr.sc. Siniša Maricic, Mr.sc Tatjana Mijuškovic Svetinovic
Faculty of Civil Engineering Osijek, Croatia

Abstract

It is known that through the aeration zone of the soil there is a constant exchange of water between the free of groundwater and the atmosphere. This exchange manifests itself through three basic hydrological processes:
1. Precipitation
2. Infiltration
3. Evaporation
All three processes are of fundamental importance for the feeding of ground waters. It is therefore essential for a good utilization of ground waters that we are well acquainted with the physics of these processes, with their volume in certain time-intervals, as well as with their comparative relations in different part of the year.
Groundwater losses are the result of evaporation (evapotranspiration) and the ground water is renewed through effective infiltration of the precipitate in a certain area. As a rule, evapotranspiration and infiltration are the main factors of the hydrological water balance of an area. For the total water balance of a time-span such as a whole season, a month or an even shorter period, these vertical groundwater-balance factors are far more important as feeding elements of the ground waters than the horizontal factors. The analysis of the interaction of the vertical components in the processes of the flowing of the water through a porous ground a section between the atmosphere and the groundwater level has shown that the losses of the water due to actual evapotranspiration take a specially important place among all other hydrological processes.
A most thorough understanding of hydrological processes and of the natural laws of the flow of the water from the underground into atmosphere and the other way around is possible on through direct measuring in special station equipped with a lysimeter and infiltrometer, with meteorological equipment.
The paper present results of 14-year (1989-2002) research on the described measuring station Varkom – Varaždin.

KEYWORDS:
Recharge, Infiltrometer, Infiltration, Evapotranspiration, Hydrologic system, Water balance, Research

1. Introduction

Knowing is that through the aeration zone of the soil happening a constant exchange of water between the free of groundwater and the atmosphere. Loss and recharge ground water during the season of the year, depend by process in this vertical water balance (Fig.1). This exchange manifests it self through three basic hydrological processes: Precipitation (rain, snow, dew), Infiltration (effective infiltration), Evaporation (evapotranspiration, evaporation of ground water). All three processes are of fundamental importance for the recharge of groundwater afar from river. It is therefore essential for a good utilization of ground waters that we are well acquainted with the physics of these processes, with their volume in certain time-intervals, as well as with their comparative relations in different part of the year. Ground water losses are the result of evaporation (evapotranspiration) and the ground water is renewed through effective infiltration of the precipitate in a certain area. As a rule, evapotranspiration and infiltration are the main processes of the hydrological water balance of the natural areas. For the total water balance of a time-span such as a whole season, a month or an even shorter period, these vertical ground water-balance factors are far more important as feeding elements of the ground waters than the horizontal factors. The only exception to this is a narrow area close to water flow which is dominantly influenced by the surface inflow and river-water. The analysis of the interaction of the vertical components in the processes of the flowing of the water through a porous ground - a section between the atmosphere and the ground water level has shown that the losses of the water due to actual evapotranspiration take a specially important place among all other hydrological processes. In order to determine the losses, practicians often use different approaches without any objective criteria. They most often apply empirical methods with coefficients set for some other, different climatic and geological areas, or they use evaporation data obtained on different evaporators in meteorological station. As a rule, such approaches to hydrological calculations always require the input of the factor of in certainty and significant errors.

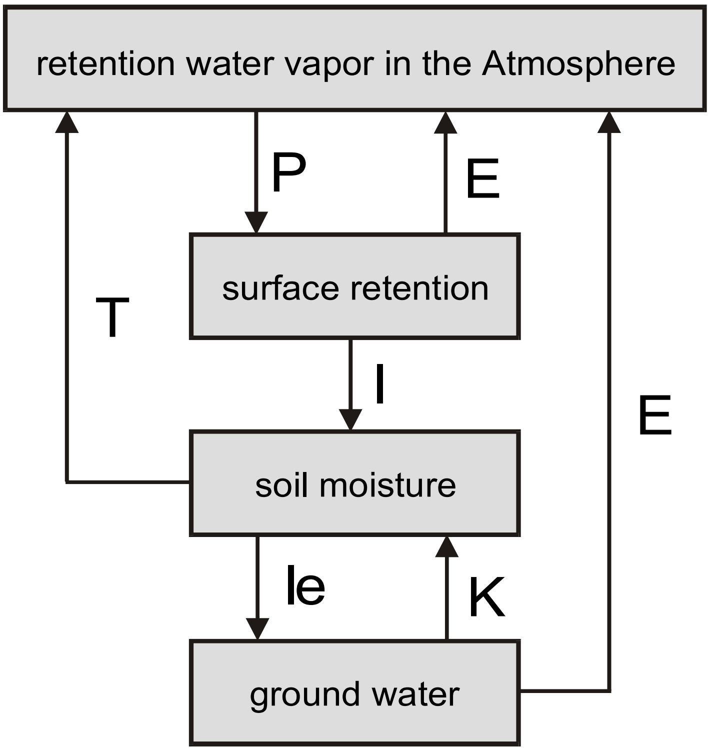

Fig. 1 Vertical water balance

where are: P - precipitation, E - evaporation, T - transpiration, I - infiltration, Ie - effective infiltration, K - capillary

The rational theory of ground water evaporation, as well as the infiltration to its free level is comprised in the laws of water and air movement in an unsaturated porous environment. A most thorough understanding of hydrological processes and of the natural laws of the flow of the water from the underground into atmosphere and the other way around is possible on through direct measuring in special measuring stations equipped with an infiltrometer with the required meteorological equipment.

The infiltrometric measuring station (Varkom) with two infiltrometers was installed (October 1988) on water supply pumping station of the town Varaždin, together with a meteorological station. The infiltrometers themselves allow direct measuring of the effective infiltration on different controlled depth-levels of the ground water to the maximum depth of 2m. Each infiltrometer has its own piezometer by which the level of ground water in the infiltrometer is controlled. Through these measuring it is possible to determine the amount of daily, real or actual evapotranspiration (Ea) from the surrounding area which is typical for the areas of the town Varaždin. By connecting the measured evaporation and infiltration with the measured physical state of the atmosphere above the measuring spot it is possible to establish certain algorithms an mathematical modes for the determining of effective infiltration and actual evapotranspiration for practical use. The methods for the measuring of the listed parameters (hydrological, meteorological, hydro geological) on the experimental station are adapted for the studying of ground water regimes of the natural areas and their results can be used in areas with similar climatic and hydrologic conditions.

2. Methodology

With respect to their time variability, the main factors influencing on recharge ground water can be generally divided into process (dynamical) and physical (stationary) factors [3]. Group of them specially in low-land agricultural regions afar from river we can listed like climatological, hydrological, hydrogeological and biological parameters.

The stationary factors describe the physical properties of the area, whereas the dynamical ones describe climatological and hydrological processes in the vertical exchange of water. In view of the nature of these processes, it can be accepted that the whole process of water exchange in the system occurs undimensionally along the atmosphere-lithosphere vertical. That space is defined by its climatic, hydrogeologic and biological characteristics. It can be divided into several zones: ground atmospheric layer, aeration zone of lithosphere, and saturation underground zone.The most dynamic parameters that also have the most significant influence on the ground water regime are hydrological and climatological parameters because of their pronounced dynamic variability in space and time. Their effect is reflected in the permanent oscillation of the water level below the earth's surface, and in the change of soil moisture above that level. Hydrogeologic parameter are not so susceptible to changes in shorter periods of time. Thus, on smaller areas they can be taken as unchangeable depending on the geological characteristics and composition of soil. Biological parameter on smaller areas can be also taken as unchangeable and dependant on vegetation cover and its seasonal changes.

By directly measuring main input and output parameters of ground waters on a set hydrological system of vertical water balance, and by hydrological analysis of all the relevant factors of water balance, it is possible to closely determine the fundamental characteristics of the system. By this we may come to better understand the main hydrological processes that naturally determine vertical fluctuations between atmospheric and ground water.

3. Investigation

By the direct measurement of the main input and output parameters of the vertical balance of ground water during a 14-year period (1989-2002), the information concerning the basic characteristics of the *Varkom* hydrologic system was obtained (Fig.3). The system is set so that it describes the natural regime of the vertical water balance of ground water to a depth of 200 cm of alluvium, and it corresponds to the circumstances existing in the northwest regions

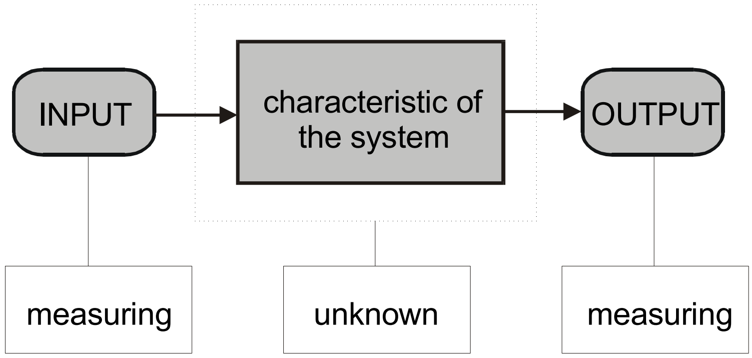

Fig. 2 Investigation work at the hydrologic system

of the Republic of Croatia, in the catchment area of the Drava River. The input parameters of the system measured at the *Varkom* test station are basic climatological parameters, represented by precipitation and air temperature. The system output parameters are hydrologic parameters represented by the measurement of the change of ground water level in the system piezometers, by the measurement of effective infiltration, and by the measurement of the loss of water from the system due to evaporation. The geological composition of the system's zone of aeration corresponds to homogenous sandy grovels, with the soil effective porosity $\eta = 0.18$. The humus layer has a thickness of 30 cm on the surface, with thick, low grass during the whole year.

The main results of the 14-year research work carried out at the *Varkom* test station are as follows. At the beginning of summer season (June) the ground water level decreases, although the secondary precipitation maximum is recorded for that particular month. The lowering of the level continues until October, when the recharging of the system due to precipitation starts, and the water level in the piezometer rises. The highest gradient of ground water level decrease in the infiltrometer system is recorded for the month of August. The loss of water from the infiltrometer system due to a capillary rise of water from the saturation zone, and its evaporation in the period from June to October, which amounts to 35 mm, cannot be compensated by the precipitation measured at the infiltrometer station during the research period.

4. Mathematical Modeling

The 14-year research work on the hydrologic system *Varkom* has shown that the actual evapotranspiration (*Ea*) and effective infiltration (*Ie*) are the main elements of the vertical water balance of the system. The analysis was made with the aim to define the nonlinear regression model that describes the hydrologic processes in the infiltrometer system with a

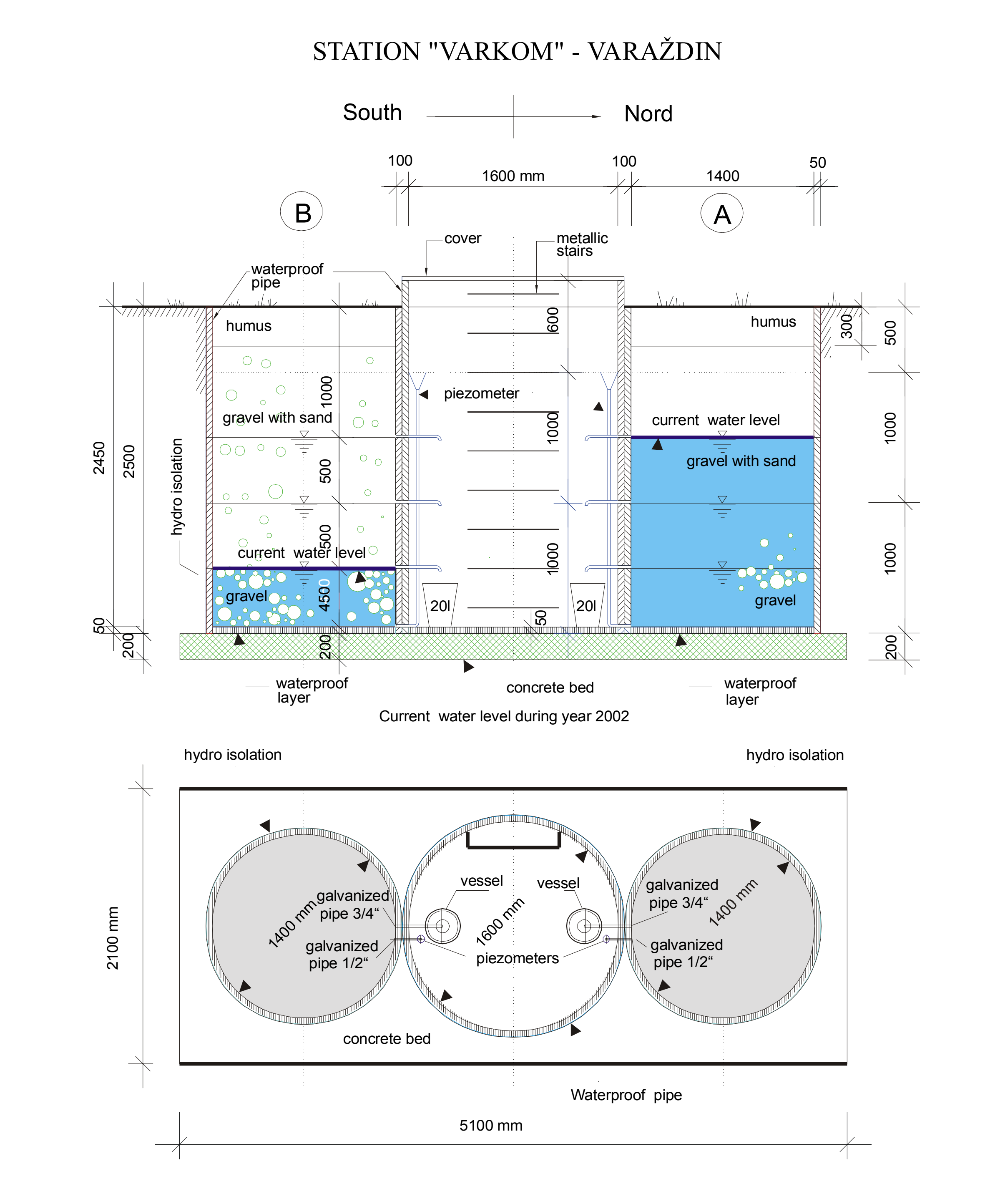

Fig. 3 Hydrologic system (Varkom)

ground water depth of 200 cm. The basic assumption was that the established infiltrometer system of *Varkom* station is a closed hydrologic system. Thus, the general equation of vertical water balance could be used. Therefore, the model computation was done for monthly values with the assumption that within an average hydrologic year there is no change of water content in the infiltrometer system. Taking into consideration two basic preconditions, i.e. that the presence of moisture and energy in the system is essential for the occurrence of the evapo-transpiration process, by the analysis and synthesis of the measurement results, the model *Varkom* [2] was obtained. The model *Varkom* for the determination of the actual evapotranspiration (*Ea*) of the hydrologic system at the *Varkom* infiltrometer station has the following form:

$$Ea = w\,P\,t^{\,n}\ (mm)$$

where are:
w = parameter (0,173); n = parameter (0,55); P = monthly precipitation; t = monthly air temperature.

Inserting the evaporation and infiltration coefficients into the general equation of vertical water balance, the final model for the determination of effective monthly infiltration of the hydrologic system of the vertical water balance of the Drava River alluvium was obtained. It has the following form:

$$Ie = P\left(1 - w\,t^{\,n}\right)(mm)$$

The model is very suitable for the use in everyday hydrologic practice, since it is only necessary to know monthly precipitation and mean monthly air temperature in the area for which the infiltration value is to be determined. The verification of the model was done [1] by simulation for the whole period of the hydrologic research work on the test station.

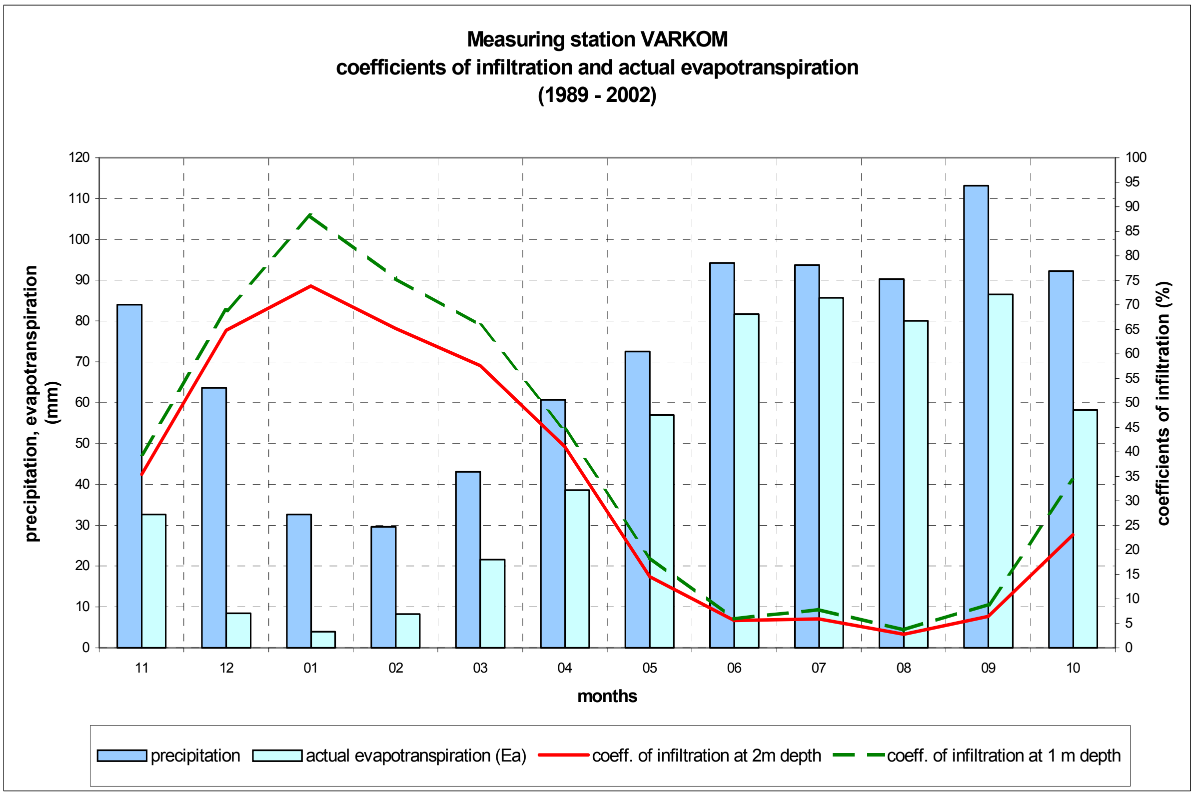

Fig. 4 results of 14-year (1989-2002) research

The significantly high correlation index of the obtained regression model (R=0.71) along with the mathematical and statistical parameters of nonlinear regression (s =0.23, Cv=0.28, Cs=0.46) indicate that the model is acceptably adjusted to the values measured on the infiltrometer station. The model shows the greatest deviation for the month of September, when the trend of ground water level lowering ends, and starts the period of saturation zone recharging by infiltration. The model is restrained by the assumption that there is no evapotranspiration (*Ea=0)* during the periods when the mean air temperature (*t*) is equal to or lower than 0°C. At low air temperatures evapotranspiration is minimal. Thus, the given restraint of the model does not introduce any significant inaccuracy into calculations.

5. Conclusion

Simulating the infiltration model for the period of research work, the model relative error of 9 % was obtained. This indicates that the mathematical model *Varkom* can be successfully used in hydrologic practice for the computation of effective infiltration and better initation rechargeable of ground water particularly in low-land agricultural regions afar from river. The model can be applied in the areas with the hydrogeological and biological characteristics similar to those relevant for the test station, and with the climatic conditions corresponding to those existing in the northwest regions of the Republic of Croatia.

6. References

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Finite Element Modelling of Chromium Mass Transport Through the Soil

Ljupcho Petkovski[1], Stanislava Dodeva[2]

[1] Faculty of Civil Engineering - Skopje, Partizanski odredi 24, PF 560, 1000 Skopje, Republic of Macedonia, petkovski@gf.ukim.edu.mk
[2] Public Water Management Enterprise "Water Management of Macedonia" - Skopje, III Makedonska brigada 10, 1000 Skopje, Republic of Macedonia, dodeva@water.org.mk

Abstract
This paper presents the results of the investigation of the transport of pollutants under the industrial dump of CEC Jugohrom. Finite element method was used for the study which was realised in two stages. The first stage is calibration of the model for analysis of the transport of contaminated material through the ground, using hydro-geological properties, boundary hydrodynamic conditions and recorded annual quantities of chromatic mass. In the second phase, the calibrated model is used to predict the pollution, and to select measures for an optimal protection of the environment.
Key words: finite element method, transport of polluting material, advection, dispersion.

1 Introduction

Industrial dump site of the Chemical Electro-metallurgy Combine (CEC) "Jugohrom" is located very close to River Vardar, the most important water resource of Republic of Macedonia. The basic raw material for the CEC "Jugohrom" is the chromium ore, while the residues of the technological process are: (1) white dust (surafine), (2) slag (carbure) and (3) bi-chromatic sludge. These residues are mixed and disposed at the industrial dump. The first two matters are inert, while the bi-chromatic sludge dissolves in water and is a serious polluter. It pollutes the ground with chromium mass, which contains very poisonous polyvalence chromium.

In the last 30 years there were approximately 40,000 m^3 $year^{-1}$ of disposed material, a quarter of which is bi-chromatic sludge. Due to the high pollution danger and the importance of the area (there is only 13 km to Rashche - the main drinking water supply source for the city of Skopje with about 600,000 inhabitants), a comprehensive hydrogeologycal research was performed and a permanent monitoring of the soil pollution with chromium mass was set in place.

From the measurements of the chromium mass pollution of Vardar river in 1997, the quantity of contaminated material transported through the groundwater to the river, is Q_m = 704 kg $year^{-1}$. The estimated length of influence from the dump site to the river is L = 200 m, which gives the specific flow of pollutants per unit length:

$$q_m = Q_m*1000/(365*L) = 704*1000/(365*200) = 9.64 \text{ g day}^{-1} \qquad (1)$$

The basic hydraulic characteristics of the ground were obtained from hydro-geological site investigation. The characteristic values of the coefficient of filtration and the porosity are: $k = 1.3\ 10^{-6}$ m s^{-1} = 0.112 m day^{-1} and n = 0.40.

2 Applied Model

The finite element method (FEM) is a process of approximation of problems of the continuum [Zienkiewicz O.C., 1975] where (a) the continuum is divided into a finite number of parts whose behaviour is described by a finite number of parameters, and (b) the solution of the whole system follows the same rules used in the discrete problem. Using FEM for hydrodynamic analysis of porous media [Connor J.J., Brebbia C.A., 1980], and analysis of the transport of polluting materials [Geo-Slope CTRAN/W v5, 2001; Geo-Slope SEEP/W v5, 2001], a wide range of problems can be solved. Simple problem solutions can be used for quick identification of a contaminated zone or the source of pollution. Generally, the complex process of movement of polluting material can be divided into transport and diluting.

The transport process comprises two basic processes: progress (advection) and parallel mixing and spreading (dispersion). The advection process is a simple migration of the contaminated material (after the pollution enters the ground), which corresponds to the flow of the ground water. The dispersion process consists of two components. The fist component, the mechanical dispersion, depends on the changes of the flow rates in the porous medium and can be expressed by using the coefficients of longitudinal and lateral dispersion (with respect to the flow direction). The second component, the molecular diffusion, is proportional to the saturation and may occur in conditions where the filtration rate is equal to zero. Involving the advection and dispersion leads to a time difference between the arrival of the pollution front and its main body (50% of the flux of the polluted mass), which increases with the distance from the source of the pollution. The coefficient of hydrodynamic dispersion D can be obtained from:

$$D = \alpha v + D^{*} \tag{2}$$

where:

- v is mean linear velocity of the flow of the pore water, $v = U/n$, $U = k \cdot i$ is Darcy's velocity and n is porosity.
- α is dispersivity of the porous medium, and
- D^{*} is coefficient of molecular diffusion, which increases with the increase of Θ ($\Theta = V_w/V$, V_w is volume of water, V is the total volume), the capability of the soil to store water, which is a function of the pore pressure. Hence, for a saturation level $s = 1.0$, or for a relative pore pressure $P = 0.0$, $\Theta = n$.

In the case of transport of reactive and/or radioactive materials there is a loss of the mass of the pollution due to adsorption and/or radioactive decay. It is assumed that process of adsorption (which is proportional to the concentration of the pollutants) is the most important factor for the decrease in pollution. The dilution due to chemical reactions between the pollutants and the soil particles or the pore water causes retardation of the migration of pollutants.

The fundamental equation for description of the transport of polluting materials through porous media is the partial differential equation of second order, derived from the equilibrium conditions of the mass of the analysed element of the porous medium. The one-dimensional form of the advection-dispersion equation is:

$$\Theta D \frac{\partial^2 C}{\partial x^2} - U \frac{\partial C}{\partial x} - \lambda \Theta C - \lambda S \rho_d = \left(\Theta + \rho_d \frac{\partial S}{\partial C} \right) \frac{\partial C}{\partial t} \tag{3}$$

where:

- x is distance in the X direction, t is the time co-ordinate;
- $C = M/V_w$ is concentration of he pollutant, defined as a ratio between the mass of the polluting materials and the volume of water in the porous medium;
- $S = M_s/\rho_d$ is adsorption, or a ratio between the adsorbed mass and the dry density of the soil ($\rho_d = \gamma_d/g$);
- $\lambda = \ln 2/T$ is the coefficient of radioactive decay, T is the half period of the radioactive material decay.

The first element on the left hand side of the equation (3) is the transport due to dispersion, the second is the advection, the third and the fourth represent the loss due to radioactive decay in the polluting mass in the fluid and the solid phase, respectively. The right hand represents the storage of the pollutant mass in the fluid and the solid phase, due to changes in the concentration of the contaminated material.

3 Analysis of the Results

The first task in the investigation of the transport of polluting material in the ground, from the dump site to the river, is to determine the representative value of the long-term (30 year) concentration of the pollutants at the contact between the dump site and the medium, leading to a $q_m = 9.64$ g day^{-1} pollutant flow in the Vardar river. For this purpose, the ground between the dump site and the river is modelled by using a 795 elements, 864 nodes mesh (Fig. 1). Using a stationary plane analysis and the potential function shown in Fig. 1, the specific filtration flow was calculated ($q_w = 1.45\ 10^{-2}$ m^3 day^{-1} m^{-1}).

The numerical analysis was performed for a period of 240 years, with a time step 100 days. The concentration of the pollution at the input (under the dump site) was assumed to be constant, whereas the output, the filtration plane at the river bed, was assumed as free (no accumulation of the contaminating material). The adopted values for the longitudinal and transversal dispersion ($\alpha_L = 5$, $\alpha_T = 5$) and the molecular diffusion ($D^* = 0.01$) were used to calibrate the value of the concentration at the input ($C_{in} = 12750$ g m^{-3}), which would lead to an output flow of contaminating material of $q_m = 9.64$ g day^{-1}. The 30 year contamination of the medium between the dump site and the river, using the characteristic pollution at the base under the dump site, is shown in Fig. 2.

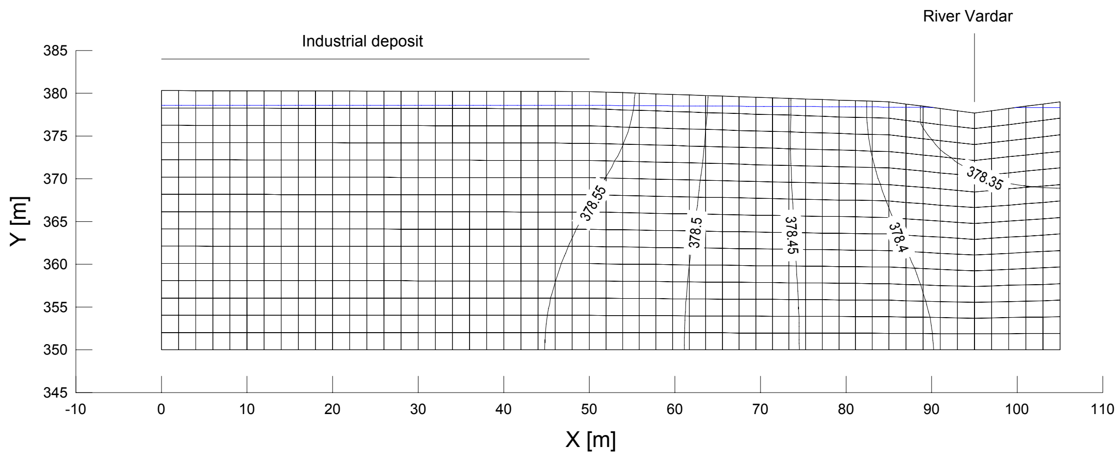

Fig 1: Finite element mesh and total head H m

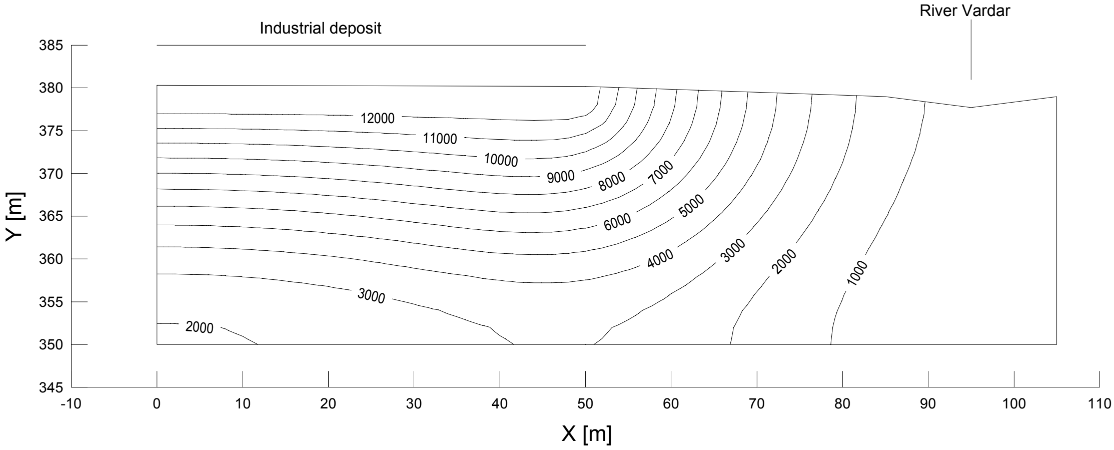

Fig 2: Pollutant Concentration (C gr m^{-3}), year 1997

Below (Fig. 3, 4) is shown the prediction of the pollutant concentration (in C gr m^{-3}) with and without dislocation of the dump site (in 2001), as the most radical measure for ground protection.

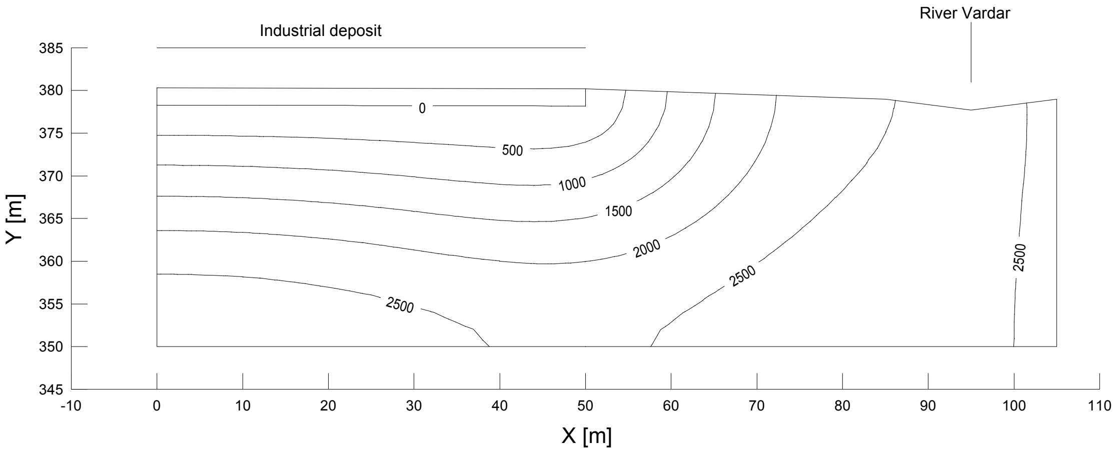

Fig 3: Pollutant Concentration (C gr m^{-3}), year 2061, with dislocation of the dump site in 2001

The concentration of the pollution in the Vardar river bed (Fig. 5, 6) and the flow of contaminating material in the control section just before the river, two important features can be observed. First, even if the dump site is relocated immediately, due to the present contamination of the ground, the pollution of the river will increase in the next 60 years, up to a level of Q_m = 2700 kg year^{-1}, before it starts to decrease. The present level of pollution (Q_m = 700 kg year^{-1}) will be reached after 210 years (Fig. 7). Second, if no measure is undertaken, the increase of the chromatic mass in the river will continue and after 250 years it will approach asymptotically a value of Q_m = 15000 kg year^{-1}.

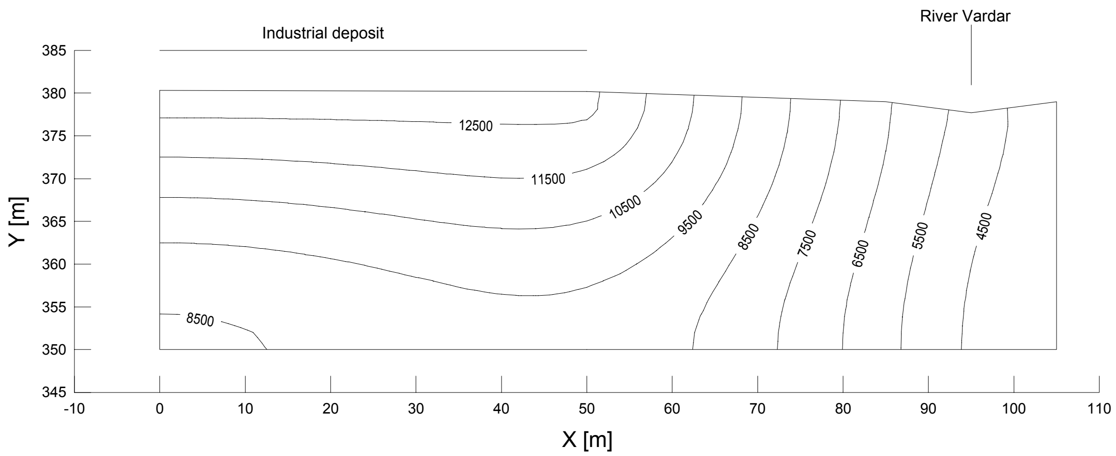

Fig 4: Pollutant Concentration (C gr m^{-3}) year 2061, without dislocation of the dump site

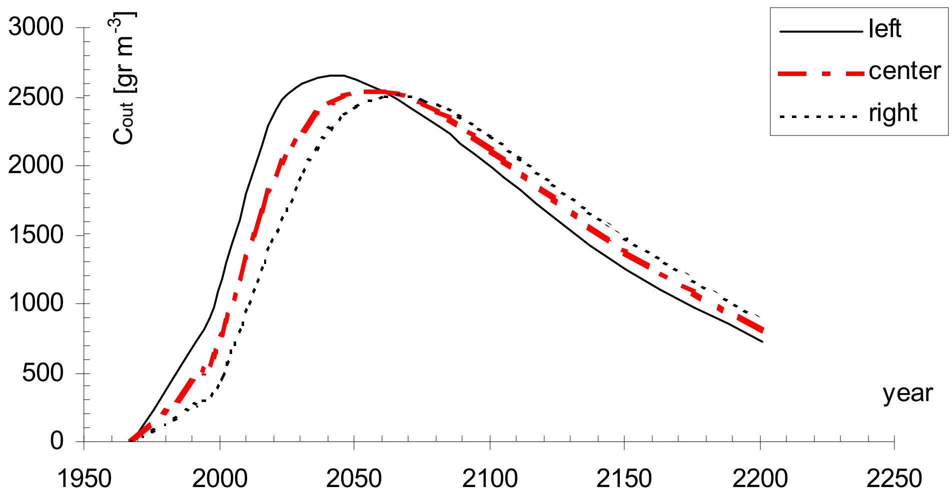

Fig 5: Pollutant Concentration in Vardar river bed (C_{out} gr m-^{3}), with dislocation of the dump site in 2001

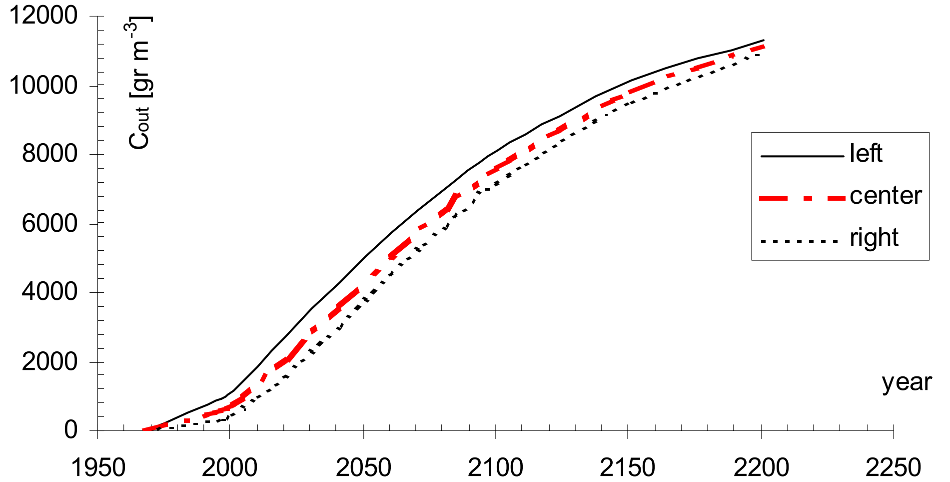

Fig 6: Concentration of Vardar rive bed (C_{out} gr m-^{3}), without dislocation of the dump site

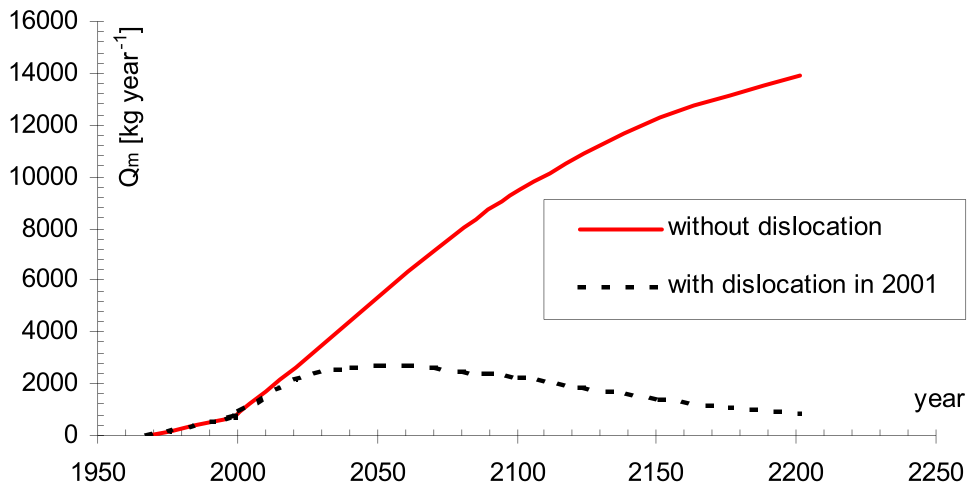

Fig 7: Contaminant flux in Vardar river (Q_m kg year^{-1}), with and without dislocation of the dump site in 2001

4 Conclusions

The first step in taking efficient measures for protection of the water resources is the evaluation of the potential for quantitative and qualitative pollution of the water. The finite element method is a useful tool for analysis of the transport of pollutants in the ground. The analyses using an FE package such as CTRAN/W integrated with SEEP/W can be used in the planning of engineering structures from the aspect of the increasing demands for better environmental protection.

The numerical simulations for evaluation of the contamination of the ground between the CEC Jugohrom dump site and the Vardar river, show that in the past 30 years the contamination of the ground with chrome mass is of such intensity that even if the dump site is completely removed, due the character of the ground water flux, the maximum pollution can be expected in 60 years. If the calculated maximum flow rate of polluting material through the ground, together with the influx of chrome mass from the surface water, exceeds the allowed concentration of pollutants, the only viable solution would be construction of a water tight diaphragm and treatment of the ground water before letting it out into the recipient.

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Selection of Procedure for Solving the Water Resource Management Task for a Serial Hydropower System

Ljupcho Petkovski[1], Ljubomir Tanchev[2]

[1] Faculty of Civil Engineering - Skopje, Partizanski odredi 24, PF 560, 1000 Skopje, Republic of Macedonia, petkovski@gf.ukim.edu.mk
[2] Faculty of Civil Engineering - Skopje, Partizanski odredi 24, PF 560, 1000 Skopje, Republic of Macedonia, tancev@gf.ukim.edu.mk

Abstract

In this paper are presented the investigation and the selection of the most favourable procedure for determining the energy effectiveness of a two-dimensional, serial waterpower system. The selection was carried out by comparison of the results obtained by using a simulation-optimisation modulus based on Network Flow Programming and an optimisation modulus based on Dynamic Programming. The numerical experiments were carried out for a real water resource system in the planning stage. The system comprises two accumulation water power plants Matka2 and Matka, on River Treska, in the Republic of Macedonia.
Key words: water resource management, hydropower system, dynamic programming, network flow programming.

1 Introduction

The resource management model is developed for two phases of the planning of the water resource systems. In the first stage, the water resource system is approximated by analytical formalisation of the relations in the subsystems and the interactions with the environment of the real system, which are assumed to be relevant for the management process. In the second stage, the procedure which is best adapted to the mathematical abstraction of the real water resource system is selected and applied to solving the resource management task. In this phase, three different approaches are applied: (i) a simulation approach, (ii) an optimisation approach and (iii) a combined approach [USACE ED, 1987. "Management"; Wurbs R.A., 1994.]. The approach which offers the best quality solution of the management task is selected by comparing the results.

According to the computational model, which depends on the character of the users incorporated in the multi-purpose water resource system, the methods for solving the management task [USACE ED, 1997. "Hydrologic"] can be grouped as:

a. methods for quantitative management of the water resource: (1) flood defences [BOSS, 1999. "DAMBRK"; Hydrologic Engineering Center, 1998. "HEC-5"; Petkovski L., 2000.], (2) time distribution and quantities to be supplied to the priority users (ecological minimum, water supply to the population and industry, irrigation), (3) determination of electric power generation and guaranteed power in critical periods [USACE ED, 1985. "Hydropower"]; and
b. methods for qualitative management of the water resources [Petkovski L., Dodeva S., 1999.].

In the initial stage of the water resource system planning a precise identification of the users is necessary for the selection of the appropriate method and for determination of the level of significance of the results from the analyses. In the considered water resource management task, the significance of the results refers to the energy efficiency of the hydro system with respect to the strict ecological and water resource criteria applied to the downstream flow.

The water resource management task for the analysed hydro-system Matka2 - Matka (M2M) comprises determination of the energy efficiency of the system for variations of the installed flow of the hydro-power plants (HPP) Matka2 and Matka, for a given hourly flow rate through HPP Kozjak, over a period of 50 years (Figure1).

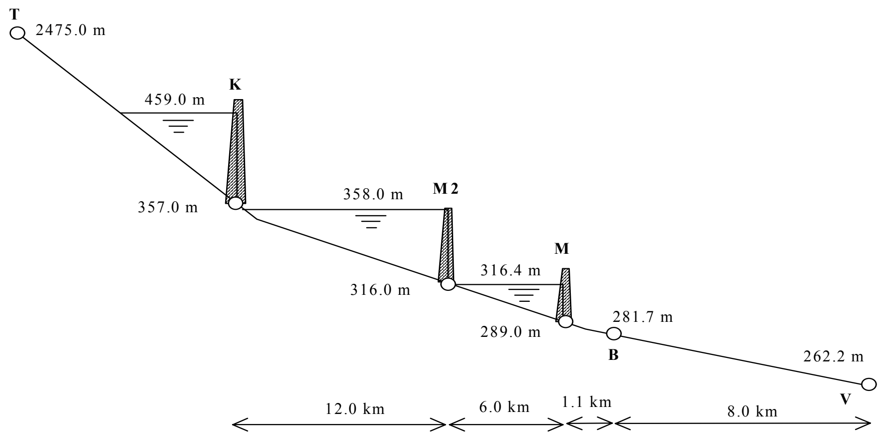

Fig. 1: Schematic presentation of the profile of Treska River from the source to the Vardar River (K-Kozjak, M2-Matka2, M-Matka)

The hourly work regime of the frontal HPP Kozjak, with installed flow rate $Q_K^{INS} = 100.0$ m^3/s, over a period of 50 years, is determined from the needs of the ESM (Electric Power Generation of Macedonia). According to the treatment of the basic hydrological input data, this task can be categorised as a water resource management solution with a deterministic approach, assuming that in the future period of exploitation the flow rates will be realised with the same statistical parameters and the same probability distribution law. The main characteristic of the M2M hydro-system is the distinguished non-uniform regime of the HPP Kozjak during the day and the relatively small usable volumes in the reservoirs Matka2 and Matka. Hence, the water resource management analysis must be carried out with a very small time discretisation step, in order to obtain acceptable accuracy so for the effectiveness of the system. In addition to the energy limitations (hourly power needs, daily energy needs and engaged power as a function of the level of the reservoir [USACE ED, 1985. "Hydropower"]), the work regime of the M2M HPP is influenced by the strict ecological criteria for the river downstream from the existing Matka dam: the ecological minimum flow, and the allowed increases and decreased in the flow rates.

2 Simulation Model

The hydraulic characteristics of the flow through the M2M system can be introduced in the management task for evaluation of the energy efficiency only by using a simple simulation model [Hydrologic Engineering Center, 1998. "HEC-5"], with a hydrological-hydraulic modulus for non-stationary effects of the flow [Hydrologic Engineering Center, 1998. "HEC-

HMS"] and simple operational management rules which can be used in the exploitation period. The usable volume of the reservoirs is divided on upper and lower zone. The lower zone is to be used in accordance with the energy and ecology requirements, with the objective of achieving the high level at the end of the period under consideration. The upper zone is to be used for retention of the flood waves from the upstream reservoir, in order to achieve the low level at the end of the considered period. In cases when the level of the reservoir is at the lower zone level, the water can be used only for peak energy needs and for satisfying the ecological minimum. If, due to increased inflow, the level of the reservoir moves to the upper zone, then the system is transferred into a phase of maximum use (in order to avoid overflows), respecting the energetic and ecological limits and, depending on the hour in the day, primary or secondary energy will be generated. The drawback of the simulation model is that the maximum of the objective function in the considered period cannot be achieved. This drawback can be attenuated by using the simulation-optimisation model [BOSS, 1999. "ARSP"], based on the Network Flow Programming, developed by Durbin & Kroenke in 1967, with an assumption that the input can be predicted only in the current time interval. Then, following a given (assumed) operation policy the criterion functional can be maximised.

Below is given the basis of the network programming formulation. The special structure of the network flow optimisation technique, which solves a subset of generalized linear programming problems, achieves a solution to a water resource allocation problem more quickly than a traditional linear program formulation and with less round-off error. The objective of the network programming application (Equation 1) is to minimize a cost function, which reflects the benefits derived from a particular operating policy, while satisfying all flow constraints and the continuity of the mass within the water resource system. Mathematically, the network programming formulation is stated as:

$$\min Z = \Sigma ijCij \text{ for all i and j} \tag{1}$$

subject to the following constraints:

$$\Sigma j\ qij - \Sigma j\ qij = 0 \text{ for all i} \tag{2}$$

$$Lij \leq qij \leq Uij \text{ for all i and j} \tag{3}$$

where:

Z is the objective function to be minimized, the sum of the cost of flow in all arcs in the network in this case,
C_{ij} is the total cost of flow in the arc from node i to node j = $c_{ij}q_{ij}$,
c_{ij} is the cost of each unit of flow in the arc from node i to node j,
q_{ij} is the total flow in the arc from node i to node j,
L_{ij} is the lower flow bound in the arc connecting node i to node j and
U_{ij} is the upper flow bound in the arc connecting node i to node j

The first set of constraints (Equation 2) ensures that the overall continuity in the network is satisfied. The second set of constraints (Equation 3) guarantees that, for a feasible solution to exist, the flow in all the arcs will fall between the pertinent minimum and maximum limits.

3 Optimisation model

The best operational method for solving optimisation tasks in the water resource planning of systems with reservoirs and HPPs, multi-stage structure and non-linear criterion functional is the Dynamic Programming [Yakowitz S., 1982.]. The method of Dynamic Programming (DP), developed by Richard Bellman in 1957, consists of application of the intuition approach to optimum: "The optimum strategy has such a property, that regardless of how we have arrived to a certain stage of the management, the next step must follow the optimum strategy, which applies from that stage to the end". The mathematical formalisation reduces this principle to setting a system of recurrent equations. Due to the excessive generality of the DP method, its use is not standard, i.e. a unique computer program has to be developed for every water management task.

For the serial two-dimensional water resource system M2M (Matka2-Matka) the system of functional equations will be:

$$f_i(V1_i, V2_i) = \max_{U1i}\left\{D1e_i(Y1_i^e, V1_i^{sr}) + \max_{U2i}\left[D2e_i(Y2_i^e, V2_i^{sr}) + f_{i-1}(U1_i, U2_i)\right]\right\} \tag{4}$$

where:
M=1,2 is the number of the subsystem: 1-upstream, 2-downstream subsystem;
$UM_i=VM_{i-1}$ is the initial state of the M-th subsystem in the i-th stage – co-ordinate of the management;
VM_i is the end state of the M-th subsystem in the i-th stage – co-ordinate of states:

$$VM_i = UM_i + XM_i - YM_i \tag{5}$$

XM_i,YM_i are the summary input and output of the M-th subsystem in the i-th stage and
DMe_i are the gains from the M-th subsystem in the i-th stage.

In the practical application of the DP apparatus for successive solving of the sets of recurrent equations a numerical (approximate) method is used. The accuracy of the method depends on the discretisation of the states. For an adopted accuracy of increase of 100,000 m^3, for the usable volumes of the reservoirs, 17 and 11 computational nodes are needed for the reservoirs Matka2 and Matka, respectively. In the solution of this optimisation task, the problem with the dimensions is crucial. If the members of the M(M+1) dimensional vectors of the optimal management are stored as real, single-precision (4 bytes long) values, then for the analysed 50 year period 50*365*24 = 438,000 stages will require a space of 2*0.438*17*11*4 = 655.248 MB. The size problem can be partially reduced by functional decomposition of the system and use of iterative DP, where in the i-th iteration only the values of the M-th subsystem are stored. The program execution would be accelerated by numerical decomposition of the DP and use of incremental DP (first applied in water resource analysed by W.Hall in 1969 year), with successive enhancement of the criterion functional in a strip-like corridor around the previously optimal trajectory of states. However, due to the enormous number of stages, it would be difficult to achieve an acceptable solution.

Here, we would like to emphasise the role of the powerful numerical apparatus of the DP in the water resource planning. The use of DP as an optimisation method in water resource management tasks is targeted towards determining of optimal rules for management of the system and determining the effectiveness of the system under optimal (ideal) management. This value will be used as an upper limit in the calibration of the sequential model for real management of the water resource system [Karamouz M., Houck M., 1982.; Petkovski L., Tančev L., 1998.; Yang X., Parent E., Michel E., Roche P.A.,1995.]. Hence, the DP is usually applied on large reservoirs with seasonal (monthly) regulation of the flow, for long periods of analysis (in order to incorporate realistically the phenomenon of piling-up of dry and rainy years). In the case of the M2M hydro-system, first, the optimal trajectory of states (defined by the optimisation method), due to the small usable volumes, would be of little help in the coding of the optimal rules. Secondly, the trajectory would contain a serious drawback since the optimisation method cannot include the effects of the non-stationary flow, which have a significant influence on the behaviour of the considered system.

The M2M system is characterised by small volumes in the usable space of the reservoirs, which can be used for hourly flow regulation during the day. In the exploitation of the M2M the hour-by-hour variations of the inflow regime (concentrated output from the Kozjak HPP) can be predicted with a high level of certainty. Therefore, in solving the management task (for determining the energy effectiveness) the concept of the "characteristic day in the month" can be used. This concept will lead to equal initial and end level within a day, over the entire period of analysis (otherwise, the balance equation of the reservoir would not be satisfied). The use of the characteristic day in the month, in this particular management task, means optimal management of the M2M system for 50*12=600 independent time intervals (days).

In the mathematical formalisation, the criterion for optimisation of the objective function of the management task is a sum of Dei – the gains from the energy generation of Matka2 and Matka HPP and Dni – the degree of satisfaction of the ecological minimum flow downstream from the Matka HPP. The price of the electric power is variable. The value of the electric power (according to data from ESM) is the highest between 6:00 - 13:00hr and 17:00-20:00hr during the day. The gains in the energy generation are proportional to the production of pondered energy Epi, which is a sum of night Eni and pondered daytime p*Edi (more valuable) energy, with a coefficient of ponder p=c_d/c_n=2 equal to the ratio between unit price of the expensive and the cheap energy. The criterion for optimisation is maximisation of the objective function, which can be achieved with successive solution of the set of recurrent equations.

4 Analysis of results of the numerical experiments

The energetic efficiency of the M2M hydro-system for selection of the most suitable procedure for solving the management task was determined for two variants, which were obtained by using the limit installed flow for the upstream Matka2 HPP and the downstream Matka HPP. The m2m-11 variant has $Q_{M2}^{INS} = 50.0\ m^3/s$ and $Q_M^{INS} = 19.5\ m^3/s$, whereas the m2m-43 variant has $Q_{M2}^{INS} = 80.0\ m^3/s$ and $Q_M^{INS} = 39.5\ m^3/s$.

Tab. 1: Annual production Ey and pondered annual production Ep of electric energy of the M2M system, depending on the accuracy of the solution

No. of nodes		accuracy	m2m-11		m2m-43	
Iz1 Matka2	Iz2 Matka	IJ	Ey MWh/y	Ep MWh/y	Ey MWh/y	Ep MWh/y
9	6	54	77,343	128,624	96,244	161,952
17	11	187	87,276	150,128	104,164	180,303
17	21	357	91,430	154,916	104,893	181,809
21	21	441	92,011	156,477	104,865	183,204
21	**26**	**546**	**91,551**	**156,102**	**104,844**	**184,073**
33	26	858	92,232	158,629	104,880	186,412

The first task of the numerical experiment with the optimum management of the M2M system (using a DP-based software) is to adopt the accuracy (tolerance), i.e. the increase in the discretisation of the states, which will offer sufficient accuracy of the results. The increase of the volumes in the optimisation task depends on the time step and on the maximum output from the subsystem. This task is unique for every particular water resource system. For the two limit values of the installed flow through M2M (variants 11 and 43) and for different accuracy indexes (a product of the number of calculation nodes for the usable volumes of the M2M reservoirs) the mean annual values of electric energy generation were calculated. The results shown in Table 1 lead to a conclusion that for an accuracy of IJ=546 the increase of the energetic efficiency of the system (expressed by the increase of the mean annual energy production) is negligible. Hence, the comparisons between the optimisation and simulation model were carried out by using an optimisation model with 21 and 26 computation nodes for the usable volumes of the Matka2 and Matka reservoirs.

Below are presented the results obtained with the simulation model based on Network Flow Programming (NFP), with monthly and weekly time frames, with and without the effect of evaporating of the reservoirs, for the two analysed variants. These results are compared with the results obtained from the optimisation model by using Dynamic Programming (DP) with sufficient accuracy.

Tab. 2: Annual production of electric energy Ey and averaged energy losses due to overflow Yo and evaporation Ye from the reservoirs, for the m2m-11 variant

		Ey [GWh/year]		Yo [m^3/s]		Ye [m^3/s]	
Method	time-step	Matka2	Matka	Matka2	Matka	Matka2	Matka
DP	hour	62.237	29.314	0.943	6.922	0.000	0.000
NFP	month	64.623	31.895	0.730	7.227	0.000	0.000
NFP	month-evap	64.579	31.886	0.729	7.207	0.015	0.008
NFP	week	64.754	31.930	0.695	7.213	0.000	0.000

Tab. 3: Annual production of electric energy Ey and averaged energy losses due to overflow Yo and evaporation evaporation Ye from the reservoirs, for the m2m-43 variant

		Ey [GWh/year]		Yo [m^3/s]		Ye [m^3/s]	
method	time-step	Matka2	Matka	Matka2	Matka	Matka2	Matka
DP	hour	65.136	39.708	0.133	1.969	0.000	0.000
NFP	month	66.996	41.566	0.091	1.784	0.000	0.000
NFP	month-evap	66.944	41.522	0.091	1.781	0.015	0.008
NFP	week	67.014	41.689	0.090	1.710	0.000	0.000

5 Conclusions

The comparison of the results for the mean annual electric energy production of the M2M system, shows that approximately equal values of the electric energy production are obtained by using an optimisation and a simulation method, with different time frames for the solution of the management task. The 1-hour time step of the discretisation is necessary, not for evaluation of the total amount of energy, but for determining the ratio between the daytime (expensive) and the night (cheap) energy. This ratio is important for determining the pondered annual energy production, which needs to be valued economically. Due to the small surface areas of the reservoirs, in this particular case, the free surface evaporation losses can be neglected.

Considering the two available methods for solution of the management task for the M2M system, the following drawbacks can be noticed. The main drawback of the simulation method is the lack of a mechanism for optimisation of the criterion functional with a precise prediction of the inflow (for the following 24 hours). A significant drawback of the optimisation method is the inability to incorporate the hydraulic characteristics of the stream, relevant for the adopted discretisation time-step and the ecological criteria for the variations of the downstream flow. These hydraulic and ecological parameters cannot be incorporated in the optimisation model because they are in contradiction with the separable character of the criterion functional of the DP apparatus. The key factor in the selection of the management procedure in this particular management task (with hourly management) is the relatively high precision prediction of the non-stationary, concentrated output from the Kozjak HPP in the following 24 hours. Hence, the best approach is to solve the management task for the M2M system by using an optimisation model.

Finally, it has to be noted that, despite the existence of a number of general purpose simulation packages which can be directly applied in analyses of water management systems with different configurations (depending on the number of reservoirs and the different types of subsystem interactions – serial and/or parallel), there are categories of management tasks for which better quality results are obtained by using specific optimisation models, (applicable only for that particular water resource system). This implies development of unique, custom-built software. The planner of complex water resource systems, in addition to the (1) knowledge of the physics of the process of management with water resources, (2) familiarity with the capabilities and the applicability of the existing general purpose simulation programs and (3) theoretical background in the methods of operational investigations; must posses programming skills for development of a unique optimisation model which will be best suited to the identified water management task.

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Hydrology of detention basins as constituents of flood protection system of Zagreb city

Josip Petraš & Davor Malus

University of Zagreb, Faculty of Civil Engineering, Water Research Department, Kačićeva 28, 10000 Zagreb, Croatia; jpetras@grad.hr ; dmalus@grad.hr

Abstract

The procedure for the hydrologic determination of the volume of detention basins planned to be designed as a part of the system for the protection of the city of Zagreb against mountain stream floods is presented. The old hydrologic standards from 1982 are mentioned. A new methodology is proposed in 1992. This methodology takes into consideration the interactive effects of artificial and natural constituent elements of the flood protection system. For defining effective runoff the SCS method is applied. After that, the maximum discharge is defined using Ven Te Chow's expression, and for defining hydrographs the proposed methodology uses Goadrich's expression and the procedure of superposition.

Key words:
detention basins; flood protection system; effective runoff; water waves; composite hydrograph.

1 Introduction

Zagreb, the capital of Croatia, is situated in the Sava River valley at the foot of mountain Medvednica which is rich in torrential streams. That is the reason why it is double flood endangered. From the valley side there is a danger of high Sava River waters, and from the mountain side a danger of torrential floods from the streams of mountain Medvednica. These streams transversely intersect the city on their way to the Sava River. Therefore, the problems of Zagreb city flood protection are separately solved. On one hand, the problem of the protection of Zagreb against the Sava River floods is a part of the flood protection system in the central river flow, and on the other hand there is the problem of protection against the mountain stream floods, which is separately solved. The interaction of these two kinds of problems has been considered by carrying out hydrologic analyses of simultaneous occurrence of the Sava River high waters and the high waters of mountain streams. These analyses have shown that the extremely high waters of the Sava don't coincide with the extremely high waters of torrential streams. The subsystem of Zagreb protection against the flood waters of the Sava is so designed and built that it provides protection with a safety factor of 99.9 per cent, i.e. it is designed for high waters of a thousand-year return period, whereas for the system of protection against mountain streams a safety factor of 99 per cent is selected, i.e. the high waters of a hundred-year return period have been chosen for the determination of the dimensions of the elements of that subsystem. These flooding safety factors have been conventionally selected, although they should have been established by carrying out cost-benefit analyses in which the prevented flood damage determines the benefits of the designed protection structures.

The evolution of hydro-technical ideas in solving the problem of the protection of Zagreb against mountain stream floods started some 40 years ago with a dilemma concerning the solution concept which involved two different variants. As the first variant the solution involving the increase of the capacity of mountain stream channels on their way through Zagreb, so that they would be capable to accept the high waters of a hundred year return period The idea was rejected at the very beginning as unacceptable. This was due to the fact that these channels are included into the Zagreb sewage system as the main sewers. The majority of them were previously designed as closed underground conduits mainly under the streets of Zagreb, and sometimes also below buildings or some other structures. In such circumstances their redesigning to the dimensions necessary for flood protection was a priori rejected.
In 1982 the first Water Management Master Plan of Zagreb City (WMMP) [1] was made. In that plan the solution of flood protection which involved a system of mountain detention basins, for the purpose of significant water quantity retardation in the mountain part of the watershed during excessive rainfall, was considered and adopted.
In 1992 the new WMMP[2] was made. According to this document the construction of 45 detention basins is envisaged. These basins are planned to be built on the locations not yet covered by urbanisation, and that is mainly on the points of connection between the mountain part of the watershed and the, so called, Zagreb terrace. With such positioning of detention basins it is possible to control the runoff especially in those areas in which, due to great inclination of the terrain, the majority of high water waves is generated. The hydrologic aspect of such situation is characterised by the fact that on such small watershed areas (l km^2 to 40 km^2) there is the occurrence of flood waves during short rainfalls of great intensity. This well-known hydrologic phenomenon confirms in the circumstances of Zagreb streams the appropriateness of the concept of runoff regulation by constructing detention basins. Otherwise, it would be necessary to design in the urbanised parts of the city very large and expensive stream canals or conduits, in which most of the time there would be relatively little or no water.

2 Hydrologic standard from 1982

In the Water Management Master Plan from 1982[1], the methodology for hydrologic computation of high waters in small watersheds of Zagreb streams was defined. That methodology didn't take into consideration the fact that the detention facilities located on the watersheds of Zagreb streams with the task of detained high water waves, make together with the watercourses integral hydrologic system.
The need for the applying of a systematic approach in carrying out the hydrologic analyses of these detention facilities follows from their synergetic effect on the reduction of high water waves. In the WMMP from 1992[2] this fact was taken into consideration.
The analyses of the synergetic effect of the detention facilities have resulted in the knowledge that the methodology of hydrologic computations presented in the WMMP 1982 is not suitable for the determination of necessary storage volume of the planned detention basins, nor for the determination of dimensions of conduits or watercourse channels.
Therefore, in the WMMP 1992 a new methodology for hydrologic computations is elaborated and proposed. It is adjusted to the systematic approach to the problem of the determination of storage volume of the planned detention basins and necessary dimensions of conduits and watercourse channels. The authors of this paper were participating in the conceiving and elaborating of this methodology.

The WMMP1982 gave a uniform hydrologic standard for the determination of high water waves from the watersheds of Zagreb torrential streams. As a result of performed analyses, the regional relation for the computation of maximum discharges is given in the analytical form as $\mathbf{Q_{RP} = aF^n}$. This relation was established on the basis of the rational method.
For that purpose the relevant effective rainfall intensities were previously determined by the SCS method. The standard from 1982 also contains the relation of the type $\mathbf{V_{RP} = f(F;RP)}$ for the determination of the volume of high water waves ($\mathbf{V_{RP}}$) depending on the return period (**RP**) and watershed area (**F**). This relation was established using the method developed by Prof. Srebrenovic, a hydrologist from Zagreb, on the basis of the previously calculated $\mathbf{Q_{RP}}$ according to the aforementioned relation. Thus, both of these relations belong to the same rainfall event..
Such a level of hydrologic treatment may be appropriate for the water management master plan and is sufficient for the determination of location and reserving of the space necessary for individual elements of the flood protection system within the physical plans of regional development. However, for the determination of the final design dimensions of particular structures that will act within the flood protection system, and as such should be the result of the optimisation of the relations existing between the relevant elements of the system, the above mentioned level of hydrologic treatment is not sufficient.
In support of that statement the application of the given regional relations $\mathbf{Q_{RP} = aF^n}$ and $\mathbf{V_{RP}= f(F;RP)}$ while designing a detention-basin system in a certain watershed can be considered. The relation $\mathbf{Q_{RP} = aF^n}$ is obtained as an envelope of maximum discharges calculated applying the rational method for all the watersheds of Zagreb streams. The runoff coefficients are determined for several characteristic watersheds on the basis of hydrometric data, and are then used in the computation of discharges in other watersheds, according to the criteria of the similarity of watersheds. Thus, for each watershed and for each return period, the relation $\mathbf{Q_{RP} = aF^n}$ gives maximum discharge value. However, the water wave relevant for the determination of the detention basin storage volume need not necessarily be the one with the maximum discharge. Relevant is the wave which passing through the detention basin requires the greatest storage volume, and this can be the water wave at which the maximum discharge does not belong to the envelope of maximum discharges, but has a very large volume, which is determined by rainfall intensity and duration. In small watersheds the rainfalls of shorter duration and higher intensity give high maximum discharge values, whereas the rainfalls of somewhat longer duration and smaller intensity result in greater volumes of water waves. This fact was neglected in the WMMP 1982.

3 Methodology for hydrologic analyses recommended in the WMMP 1992

The relations $\mathbf{Q_{RP} = aF^n}$ and $\mathbf{V_{RP}= f(F;RP)}$ from 1982 mean that for each watershed a rainfall of certain duration and intensity which gives maximum discharge $\mathbf{Q_{max}}$ and maximum water wave volume $\mathbf{V_{max}}$ is relevant for designing flood protection structures. Concerning the planned design of the flood protection system with detention basins in the watersheds of Zagreb streams, the question of the appropriateness of the given standards in the determination of hydrologic parameters of the structures within that system arose. Since the respective watersheds above the detention basins are different in area, it means that by using the given relations while finding the relevant hydrographs for the determination of the storage volume of detention basins and their evacuation facilities, the hydrographs of different rainfall duration would be obtained. Such hydrographs would give unrealistic hydrologic information about the total situation in the watershed, if having in mind the fact that the system is influenced by the detention basins. So obtained hydrographs do not belong to one (and the same) rainfall event, and therefore from the standpoint of the system approach it can be said

that incompatible hydrographs are in question. These hydrographs can not be thus reliably used for the determination of the storage volume of detention basins or for the determination of dimensions of other elements of the integral flood protection system. Therefore, the conclusion was reached that the hydrologic standards from 1982 should be changed in order to provide the compatibility of hydrographs. The need for their compatibility follows from the fact that all the objects within the same flood protection system work integrally, and thus the effect of a particular structure on the functioning of the system as a whole can be only analysed under condition that the real hydrologic model of the system exists.
The fulfilment of the condition that the hydrologic analysis should result with the real hydrologic model of the system of detention basins and watercourse channels in the watershed, can be realised by the application of an adequate precipitation-runoff model of particular precipitation. Such models which enable defining of runoff hydrographs on the basis of real or some estimated precipitation satisfy the condition of the compatibility of hydrographs, because all the hydrographs of particular precipitation are compatible. This particular approach was applied in the hydrologic analysis of Zagreb streams during the elaboration of the WMMP 1992. The hydrologic analysis is based on the concept that can be illustrated by the following block diagram:

	subsystem A		subsystem B	
P	**Transformation of gross precipitacion into net precipitacion**	$\mathbf{P_e}$	**Time distribution of net precipitation – defining of real hydrographs**	$\mathbf{Q_{(t)}}$

This concept makes the framework for the systematic approach to the problem of the determination of the dimensions of basic elements of a flood protection system within an integral hydrologic system in the watershed. According to this concept, it is necessary to define the **subsystem A** that reduces gross precipitation **P** into effective precipitation **Pe**, and the **subsystem B** that gives the time distribution of effective runoff in the form of real hydrographs.
A necessary precondition for the carrying out of such hydrologic analysis is the correctly defined input into the system, and that is the estimated precipitation of selected duration (**D**), of adequate intensity (**I**), and of a certain frequency of occurrence (**F**). On the basis of a large amount of pluviographic data, the **RDF** (Rate-Duration-Frequency) curves, presented in WMMP 1992, were defined for the Zagreb area. A special requirement which is to be met by the mentioned subsystems **A** and **B** is that they should be determined in such a way to enable the defining of the hydrographs for the particular return period **RP** to which the input precipitation belongs.

Definition of effects of subsystem A

In WMMP 1992 the **SCS**[3] method is selected for the definition of the effects of subsystem A. For the entire area considered in the WWMP 1992, the runoff curves were calculated according to the SCS method for particular regions that with respect to runoff conditions represent relatively homogenous wholes. The homogeneity of particular region was assessed on the basis of the similarity between vegetation cover and soil pedological characteristics.
All together 21 homogenous regions were defined, and the same number of the graphic presentations of surface runoff coefficients α =**Pe/P** were made. For each region three curves were calculated on the basis of antecedent moisture conditions. The obtained results were checked on several available hydrometric profiles of Zagreb streams, using the recorded data on discharge and related precipitation intensity. One example of the runoff curves calculated for the watersheds of Zagreb streams is presented in Fig. 1.

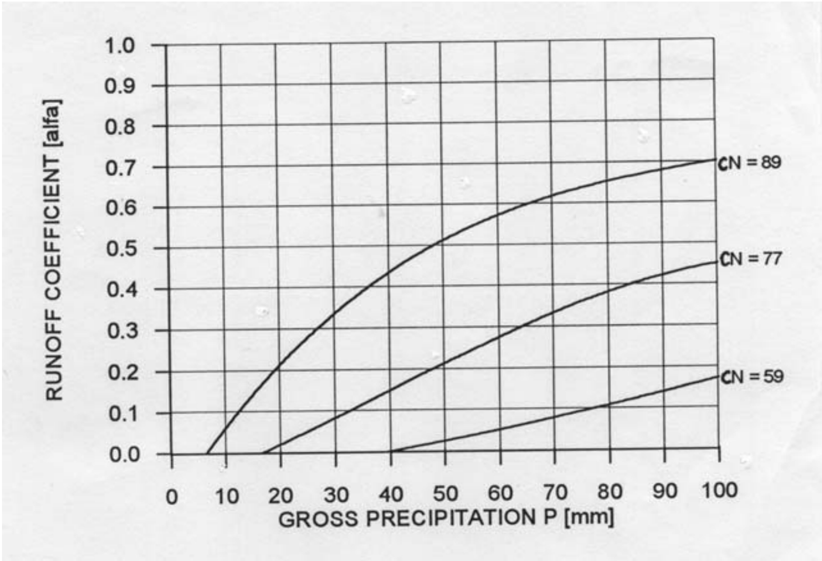

Fig. 1: An example of runoff curves α **=Pe/P** for *Sesvete region*

Definition of effects of subsystem B

For the definition of actual hydrographs a procedure based on Goadrich's[4], [5], expression is proposed:

$$\mathbf{Q(t) = y(t)Q_{max}} \quad (1)$$

where:

$$\mathbf{y = \exp -\lambda(1-x)^2 / x} \quad (2)$$

$$\mathbf{\lambda = (Q_{max} * t_{p1}) / W} \quad (3)$$

$$\mathbf{x = t / t_{p1}} \quad (4)$$

$$\mathbf{t_{p1} = 0{,}5\ t_k + t_p} \quad (5)$$

The meaning of the individual parameters is shown in Fig 2.

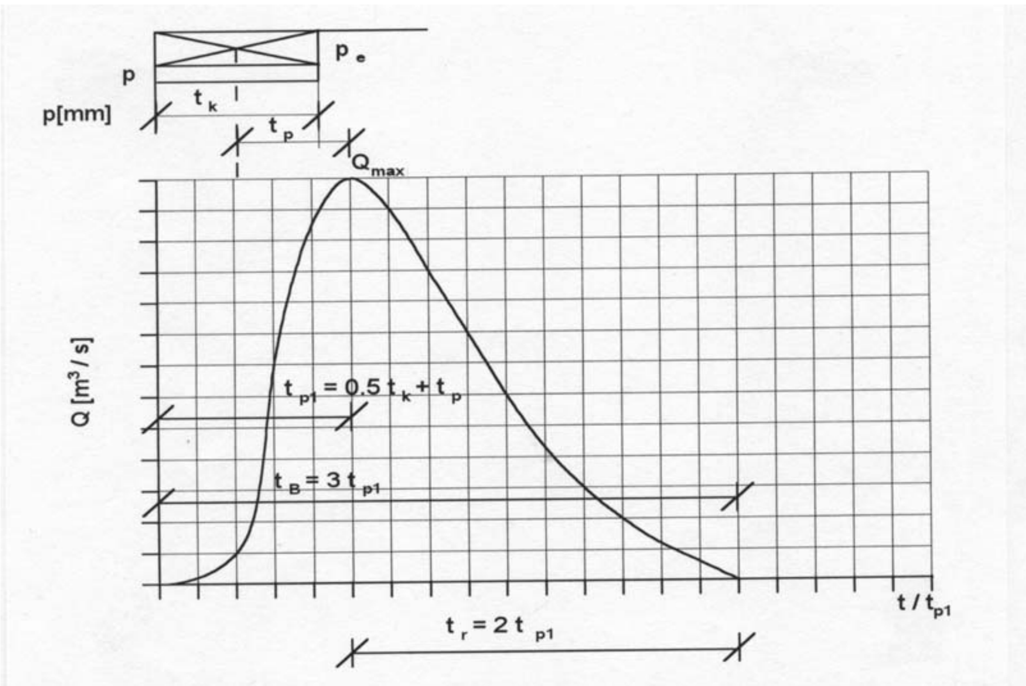

Fig.2: The parameters of the hydrographs of Zagreb streams

For the definition of the shapes of hydrographs it is necessary to determine the relation between the rising segment of a hydrograph (t_{p1}) and its recession segment (t_r). On the basis of the water waves observed on the available water-gauge, the relation tp/tr=1/2 was accepted for all Zagreb streams. For the definition of the hydrographs of high water waves according to Goadrich it is necessary first to determine maximum discharge $\mathbf{Q_{max}}$ on the basis of assumed rainfall duration $\mathbf{t_k}$. Also the water wave lag time $\mathbf{t_p}$ and water wave volume $\mathbf{W=P_e*F}$ have to be determined. For the determination of water wave volume, earlier calculated runoff curves for 21 homogenous regions of the watersheds of Zagreb streams were used.

In the **WMMP 1992** for the calculation of maximum discharges the procedure according to Ven Te Chow[4], [5], was proposed:

$$\mathbf{Q_{max} = 16{,}6\ F*i*y*k} \qquad (6)$$

where:
16,6 = dimensional factor
F = watershed area (km^2)
i = net precipitation intensity [mm/min]
y = climatic factor (for Zagreb area y=1 is accepted)
k = hydrograph-peak reduction factor

The core of this procedure is determined by factor **k.** For estimation this factor Ven Te Chow recommend the expression:

$$\mathbf{k = -0{,}00303 + (0{,}84902\ t_k / t_p) - 0{,}17747\ (t_k / t_p)^2} \qquad (7)$$

In WMMP 1992 for estimation lag time $\mathbf{t_p}$, according to prof Srebrenovic[6] , the next expression is recommend:

$$\mathbf{t_p = 0{,}49 * [\ L*U / S^{0,5}]^{\ 0,38}} \qquad (8)$$

where:

L = length of watershed in **m** measured along the main water channel,
U = distance of the reference profile of water channel from the watershed center,
S = average slope of watershed.

By means of this mathematical apparatus, it is possible to calculate for the precipitation of various intensity and duration the compatible maximum discharges in the channels of Zagreb torrential streams, on the locations selected for the construction. of detention facilities, as well as on the locations referent for the determination of the necessary dimensions of the channels.

4 Designing of compatible high water hydrograps for reference profiles of flood protection system

The presented method for the defining of high water hydrographs enables the designing of hydrographs on those locations (profiles) of water channels towards which the watershed gravitates, without any built structures that would essentially change the natural runoff regime. In the event when in the watershed upstream of some significant location (i.e. reference profile of a water channel) detention basins act, it is necessary to define a composite hydrograph on such a profile, with the condition of the influence of detention basins on the change of the natural runoff regime. For the definition of such hydrographs in the WMMP 1992, a simple procedure of superimposing water wave hydrographs from particular hydrographic branches of the planned flood protection system is provided. Since this system consists of detention basins with their evacuation facilities and of natural water channels which are to be artificial changed, the hydrographs from particular hydrographic branches of the system will consist of the transformed hydrographs at the outlet of detention basins and hydrographs from the subwatersheds between the detention basins. The transformed water wave hydrographs at the outlet of detention basins can be obtained by the hydrologic and hydraulic analysis of the effects of a detention basin and its evacuation facilities on the incoming water wave. The hydrograph of the incoming water wave for the first (the most upstream) detention basin is obtained according to the earlier described Goadrich's procedure, whereas the incoming water wave hydrograph for the detention basins in a sequence is obtained by superimposing the transformed hydrograph of the previous detention basin and the hydrograph of the water wave from the subwatershed between two detention basins. The hydrograph of the water wave from the subwatershed is also obtained according to the described Goadrich's procedure. For the simplified procedure of the superimposing of water waves only the translational shift of the detention basin hydrographs on the time axis with respect to the duration of the wave flowing in the channel to the reference profile is taken into account. The effect of water wave flowing in the channel on the reduction of the ordinates of hydrographs is neglected. Such proposal was made since relatively small watersheds with short channel length between reference profiles are in question, and also the detention basin outlet hydrographs are very flat and with a relatively long time basis, so that the effect of flow in the channel on the reduction of hydrographs' ordinates can be neglected.

In order to illustrate the application of the presented methodology, a short description of the procedure for the defining of the hydrograph of high water with a return period of 100 years for the reference profile No.5 shown in Fig.3 will be given. In Fig.4 the subsystem of *Bliznec* stream and *Stefanovec* stream with three planned detention basins from the complex of the Zagreb city flood protection system is illustrated.
According to Fig.3 in order to solve the defined task, it is necessary:
- to define the hydrograph of high water wave that enters *Jazbina* detention basin,
- calculate the transformation of water wave due to the effect of the retention basin and its evacuation facilities,
- determine the duration of the transformed water wave flowing from the retention basin to the reference profile,
- determine the hydrograph of water wave from the subwatershed between the detention basin and the reference profile,
- carry out the calculation of water waves superimposing, i.e. calculate the composite hydrograph.

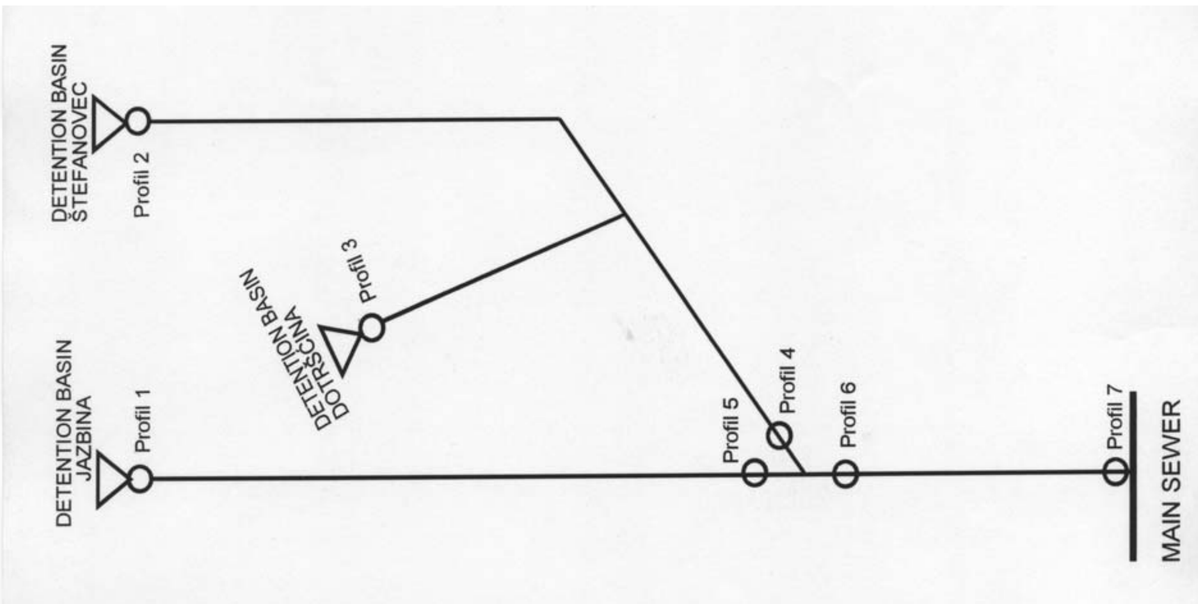

Fig.3: The subsystem of *Bliznec* stream and *Stefanovec* stream

Since the defining of the hydrograph of high water with a return period of 100 years is set as the task, in the task-solving procedure the RDF curve of 100-year return period from the WMMP 1992 is used. From this curve, the intensities of several selected rainfall durations (for which the hydrographs on the selected profile No.5 will be defined) are read off. It is necessary to select several rainfall durations, because at the start we do not know which rainfall will result with a maximum hydrograph on the selected profile. Actually, it is also necessary to decide whether to define the hydrograph of a 100-year high water wave according to the parameter of wave maximum discharge (Q_{max}) or according to the volume parameter (W_{max}). However, in both cases it is necessary to solve the task for several rainfall events, since at the start we do not know either which rainfall will result with a maximum discharge, or which one will give a maximum volume of the water wave. After reading off from the RDF curve the data of the rainfall events for which the task will be solved, the procedure follows as presented below:

Step 1 - effective precipitation **Pe** is determined for each selected rainfall using the runoff curves that are designed for the watersheds of Zagreb streams as described previously (see Fig.1);

Step 2 - for each selected rainfall maximum discharge (Q_{max}) is calculated according the equation (6), separately for the watershed that gravitates towards the detention basin *Jazbina* and separately for the subwatershed;

Step 3 - for each selected rainfall the hydrograph of the water wave from the watershed gravitating towards the detention basin, and the hydrograph of the water wave from the subwatershed directly gravitating towards the reference profile No.5 are calculated according equation (1);

Step 4 - the transformation of the hydrograph for each water wave (determined in the previous step) which passes through the retention basin is calculated hydraulicly - based on the detention basin volume characteristics and the hydraulic characteristics of water evacuation facilities;

Step 5 - duration of the transformed water wave flowing from the detention basin to the reference profile is calculated according to open channel hydraulics rules;

Step 6 - the water wave from the subwatershed is superimposed with the transformed water wave from the detention basin for each rainfall selected in step 1, taking into account the translational shift of the transformed hydrograph on the time axis that is equal to the duration of wave flowing from the retention basin to the reference profile;

Step 7 - the functional dependence of the maximum discharge on the rainfall duration $Q_{max} = f(t_k)$ have to be defined on the basis of obtaind superimposed hydrographs, i.e. on the basis of the pairs of values (Q_{max}; $t_{k)}$ to each selected rainfall, for which the superimposed hydrograph is determined. The maximum discharge is read off from the superimposed hydrograph. Then, the maximum of the function $Q_{max} = f(t_k)$ have to be find, and for this maximum the associated rainfall duration could be read off (determined). With this rainfall duration the whole procedure of finding the superimposed hydrograph on the reference profile have to be repeated. This last hydrograph represents the solution of the set task, with respect to the maximum discharge.

Step 8 - the solution of the set task with respect to the maximum wave volume could be find on the similar way like in step 7, only instead of Q_{max} the W_{max} have to be considered;

The composite hydrograph for the profile No.5 obtained by the described procedure with respect to the parameter of maximum discharge Q_{max} is presented in Fig.4.

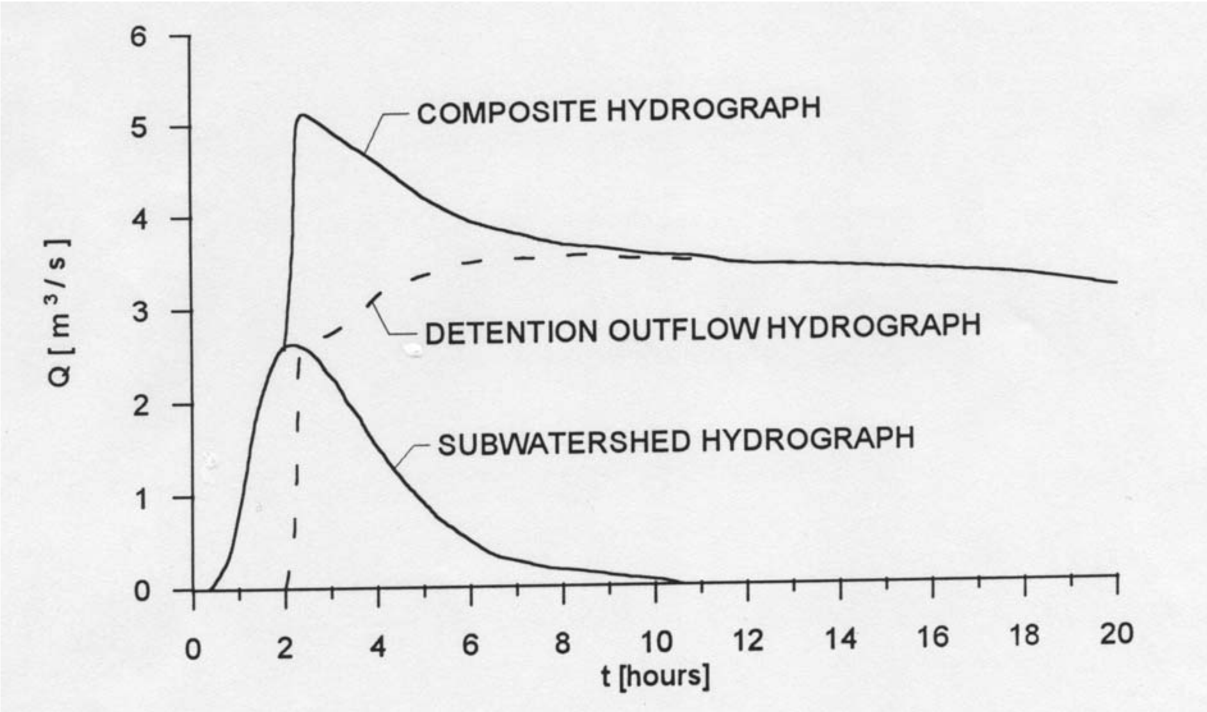

Fig. 4: Composite hydrograph on the reference profile No.5 of *Bliznec* stream

5 Conclusion

The presented methodology of hydrologic analyses was elaborated for the needs of designing hydrotechnical structures within the system of Zagreb city protection against the floods of mountain streams. With the use of this methodology it is possible to determine on the reference profiles of the system the compatible hydrographs of the rainfalls of the same duration, which was not the case in the earlier hydrologic standards of the Zagreb City Water Management Master Plan (WMMP) from 1982. The authors of the paper are of the opinion that the presented methodological procedure can be recommended for the carrying out the hydrologic analyses of water resources systems consisting of water regime regulation structures (storage reservoirs and/or detention basins) and water channels (canals, and watercourses). Of course, while carrying out such analyses, all the specific features of particular systems have to be taken into consideration.

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10000 Zagerb, Jurisiceva 10, Croatia. YU ISBN 86-7059-011-5

Vulnerability Assessment of the Water Resources

Cvetanka Popovska

University of Ss. Cyril and Methodius, Faculty of Civil Engineering, Department of Hydraulics, Hydrology and River Engineering, 1000 Skopje, Macedonia, popovska@gf.ukim.edu.mk

Abstract

This paper deals with climate change impact and vulnerability assessment of the water resources in the Republic of Macedonia.Water balance components are influenced by many factors. Climatic factors are presented with the rainfall characteristics, their time and space distribution and with the evaporation components. Climatic changes impact on the hydrological cycle components has been analyzed with the measured historical hydrological and meteorological parameters for the selected stations in the representative river basins for the period 1950-2000. Trend lines of the observed annual discharges in the main rivers and water levels in the natural lakes have been obtained and almost all of them have a descending trend. According to the performed analysis it can be stated that the climatic changes as a result of global warmth will cause significant changes in hydrological cycle and with that in water resources and water economy planning as well. National action plan and implementation of adaptation measures will be of high significance for long-term planning of the water resources.

1 Introduction

In Macedonia no organized examination of the climate change impact on the water resources has been carried out until now. Macedonia is a small country by area, but has different climatic characteristics. In one part the climate is continental, and in other part mediterranean with varieties between the two types. Climatic differences can be generally described through annual amounts of rainfalls, that are 400mm in the Ovche Pole region up to 1.300mm in the western Macedonia. From the hydrological aspect the differences are described with average annual runoff coefficients in range of 0,50-0,10.
In Macedonia, last decades flood flows have occurred twice, almost in the whole territory, in November 1962 and in January 1979. Local appearances of the flood flows have been noticed in other periods as well. Also it can be stated that the torrential floods are becoming more and more frequent. There were several droughty periods in the Republic of Macedonia, and the most characteristic is the period 1982-1992, when large number of springs and wells dried.
In the second half of the XX century, large number of regional water supply systems, irrigation and drainage systems, reservoirs, hydropower systems and other hydro technical structures have been built in Macedonia. Water transfer from one to another river basin has been performed in some of the systems. Construction of these systems and structures and their exploitation conditions have significant impact on the surface and ground waters regime. Human activities in the nature have direct impact on the local water balance and on the local climatic changes that can influence also the global climate changes. Therefore, it can be

stated that the human activities and the global climate changes have common influence on the water resources.
The objectives that have to be attained with this investigation of the vulnerability of the country to the climate changes are: (i) determination of the water regime in main rivers, (ii) determination of the water regime in natural lakes, (iii) determination of the change in water regime and (iv) adaptation measures and draft action plan with strategies and activities.

2 Water Resources

Water resources term is closely related to the water source areas that have utilization value. For long term planning of the quantity and maintenance of the quality, observations and measurements are required, as well as forecast and operation modelling where climate impact examination has significant contribution as well. Climate variety in our country can be seen through the general data for physical-geographic, climate-meteorological and hydro-geographic characteristics.
Hydro-geographic territory of the Republic of Macedonia belongs to the following basins: river Vardar and river Strumica basins which gravitate towards Aegean Sea (22.351km^2 or 86.9%), river Crn Drim basin gravitates towards the Adriatic Sea (3.318km^2 or 12,9%) and the basin of river Juzna Morava belongs to the Black Sea (44km^2 or 0,2%). The longest river is Vardar (302,6km), with basin area of 22.456km^2 , with average rainfalls of 660mm and with total annual discharging of 4,56$\cdot$10^9 m^3. The basic hydro-geographic characteristics of the main rivers in Macedonia are shown in Table 1.

Table 1 Hydro-geographic haracteristics of the main river basins

River	A (km^2)	H_{av} (m)	L_r (km)	S (‰)	Q (m^3/s)	$\forall$ (m^3)$\cdot 10^6$
Vardar	22.456,0	793,0	301,6	2,12	144,9	4.564,35
Treska	2.068,0	1.010,0	138,3	10,54	24,2	762,30
Pchinja	2.840,7	758,0	136,4	10,62	12,6	396,90
Bregalnica	4.306,8	722,0	225,0	6,90	14,1	444,15
Crna	5.890,0	863,0	228,0	4,56	37,4	1.178,10
Strumica	1.520,0	638,0	75,1	18,03	4,2	132,30
Crn Drim	4.348,2	1.166,0	56,2	4,18	47,7	1.502,55

3 Definition of the Water Balance Characteristics

Water discharging in Macedonia is performed through three watercourses and they are: river Vardar at Gevgelija, Crn Drim at Debar and river Strumica at Novo Selo. The average annual runoff coefficient is highest in the western part (0,48) and lowest in the eastern part (0,18). The average annual evaporation is lowest for river Vardar basin (440mm), and highest for river Strumica basin (649 mm). Inflow waters in Republic of Macedonia are rivers: Lepenec, Pchinja and Eleshka. Output waters are rivers: Vardar, Crn Drim, Strumica and Cironska. Available water quantities from the surface input waters are 1.014 millions m^3/annum, 6.360 millions m^3/annum from output waters and 5.346 millions m^3/annum are domicile waters. It can be stated that in the territory of Republic of Macedonia, 84% of the available water quantities are domicile waters and only 16% are outside waters.
Three natural lakes have also great significance for the hydro-geography of the Republic of Macedonia and they are: Ohrid Lake with total area of 348,8km^2 (Macedonian part 229,9km^2)

and with maximum depth of 285m; Prespa Lake with total area of 274,0km^2 (Macedonian part 176,8km^2) and with maximum depth of 52,4m; Dojran Lake with total area of 43,0km^2 (Macedonian part 27,4km^2) and with maximum depth of 10m. In order to utilize the hydrological potential of the rivers, 19 large and over 100 small reservoirs/lakes have been constructed, with total volume of 1.854 millions m^3 of water.
In our country 4414 springs with total yield of 991.9 millions m^3 /annum have been registered. Three of these springs are in the central part, and all others are in the western region. More significant springs are: Izvor (yield over 3m^3/s), Studenchica (yield 0,4 to 4,3 m^3/s), Pitran, Peshnica and Belica (yield over 6 m^3/s), in the Treska basin, then St. Naum (yield over 10 m^3/s), Biljana, Duvlo, Vevchani (yield over 1,5 m^3/s), and Rosoki (yield over 2,5 m^3/s) in the Crn Drim basin with Ohrid Lake. In the Crna Reka basin, four springs exist and the largest one is Izvor (yield over 1 m^3/s). In the river Vardar basin without river Treska, 19 springs exist and Rashche is the largest one (yield over 6 m^3/s).
Ground waters have also been noticed, but insufficient and no appropriate data for their yields and quantities exist. Observation and examination of the ground waters have not been performed systematic and continuous, except for the local demands in certain regions. Static funds of ground waters are: for the region of Polog 193 millions m^3, for Skopje valley 925 millions m^3, for Kumanovo valley 675 millions m^3, for Ovche Pole 256 millions m^3, for Strumica valley 850 millions m^3, and for the region of Gevgelija and Valandovo 342 millions m^3.
Required water quantities in different water economy segments for the condiotion in 1996 and designed condition until 2010 and 2020 are shown in Table 2, with a remark that water demands for the energy/power produce are not completely developed in the existing and valid plan documents, as a result of the fact that most of the built and planned hydro-power systems release the installed discharges again in the water courses and are used as available water resources for other and/or same purposes.

Table 2 Water demands in (m^3/annum)·10^3

	1996	2010	2020
Water supply: -population -industry	 207.993,5 274.147,0	 285.602,2 280.580,5	 336.389,5 287.014,0
Tourism	7.254,0	9.295,4	11.871,8
Irrigation	899.335,0	907.376,0	1.806.711,0
Fishery	166.800,0	189.483,0	212.166,0
Total	1.555.529,5	1.672.337,1	2.654.152,3

Souce of data: Physical Plan of Republic of Macedonia, (1998)

3 Vulnerability Assessment

Water demands in Macedonia generally are increasing. Therefore, it is assumed that in the near future, socio-economical development will escalate in characteristic pressure for solving out water economy problems, especially because these problems cannot be solved irascible and incidentally, but organized and in long term bases. Climate change can only increase this pressure having into consideration that hydrological parameters are especially sensitive on the change of the climate. Change of the intensity and time distribution of the rainfalls, dictates the frequency and intensity of the floods and droughty seasons, while the change of temperature will cause changes in evaporation, infiltration and water capacity, and all those factors participate in the water balance. Water resources and hydrological components in one region are influenced by the human activities as well, such as woodcutting, urbanization and water utilization. Hydrological investigations have been directed lately, towards the effect of

global warmth. But, having into consideration that they are still in the development stage, final answers for causes and consequences do not exist. Regional differences can be determined by analysis of hydrological time series of water levels and flows. As the time period is longer, influence factors are more reliable. Selection of the hydrological stations has been made according to the following criteria: (ii) hydrologic station should be representative for the basin with certain climatic-meteorological conditions, (ii) hydrologic station should show results from several river basin areas different by exposition, (iii) measured data should not be influenced by structures and reservoirs with flow regulation or this influence on the watercourse regime downstream should be minimum, (iv) hydrological station should be with longer period of observation and with qualitative information, (v) hydrological station should be on the watercourse that belongs to the representative basins on the territory of Macedonia, (vi) near the hydrological station a meteorological station is required in order to register all climatic-meteorological parameters and (vii) meteorological station should be with continuous data series and should represent the climatic variability in the river basin.
In accordance with the stated criteria the following hydrologic stations have been selected for further analysis of the climate change impact on the hydrology and water resources: river Vardar in Skopje, river Crna in Skochivir, river Radika in Boshkov Most and river Strumica in Sushevo, and Prespa Lake in Stenje, Dojran Lake in Nov Dojran and Ohrid Lake in Ohrid, have been selected for the natural lakes. In this paper will be discussed the discharge and water level characteristics only for the stations: river Vardar in Skopje, river Strumica in Sushevo and Dojran Lake in Nov Dojran.
The analysis of available water resources and estimation of their change in the future have been performed with determination of flow trends for the surface watercourses hydrological stations and with trends of water levels for hydrological stations of the lakes, for the period 1951-2000. Real hydrographs Q=f(t) and real water level diagrams H=f(t) are shown on the Figures 1, 2 and 3. Reduction of the characteristic flows and water levels for all analyzed hydrological stations is obvious.
Average flow for river Vardar in Skopje, Figure 1, for the period 1971-1980 is 64,56 m^3/s, for the period 1981-1990 it is 53,61m^3/s, and for the period 1991-2000 it is 46,02m^3/s. Presented in percentage average ten years flows reduce for 14%-17%. Presented in percentage, maximum waters in river Vardar, within the period 1961-2000 have reduced for 79%.
For the river Strumica in Sushevo, Figure 2, a very bad hydrological condition can be stated. Minimum flows are within the range 0 to 0,5m^3/s. Average flows are Q_{av}=2,11m^3/s for the period 1961-1970, Q_{av}=1,84m^3/s for period 1971-1980, Q_{av} =1,42m^3/s for period 1981-1990 and Q_{av}=1,0m^3/s for the last decade 1991-2000. With this trend of descending, it is expected in the following period, river Strumica to be classified as a river that is drying up, that is, to lose the category of permanent watercourse. Considering the time and space distribution of hydrological and meteorologiocal parameters in this part of the country, similar situation can be assumed for the other watercourses in this region.
For Dojran Lake in Nov Dojran, Figure 3, are shown the characteristic water levels for the period 1951-2000. Small amplitudes of the water levels with continuous descending trend have been noticed for the period 1955-1985. Oscillation amplitude for the period 1956-1980 (24 years) is 219 cm, and for 1984-1995 (11 years) the amplitude increases to 411cm. As a conclusion it can be stated that the amplitude increases proportionally almost twice, with double reduction of the time period. Average and minimum water levels have same descending trend that continues in the last five years too. Alarming descending of the level of this natural lake causes ecological catastrophe with largest consequences on the rare species of flora and fauna. Life quality of the people in the region is significantly reduced, and society demages are also presented trough the impact of the tourism which is almost completely shut down.

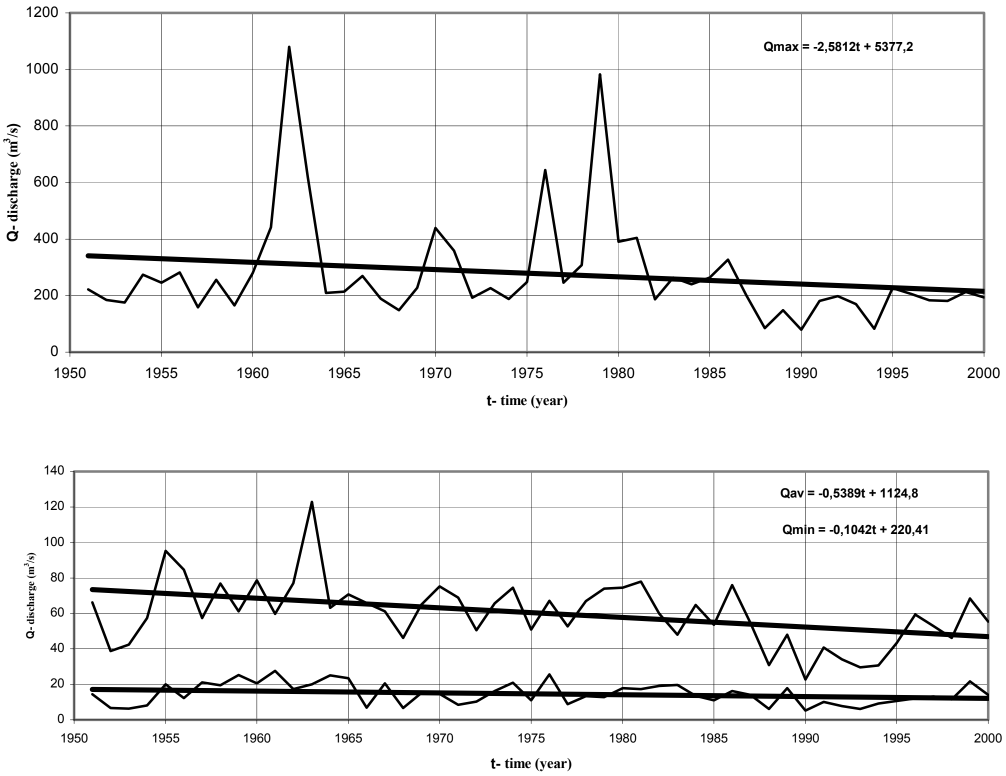

Fig. 1: Annual values of characteristic flows of the river Vardar in Skopje

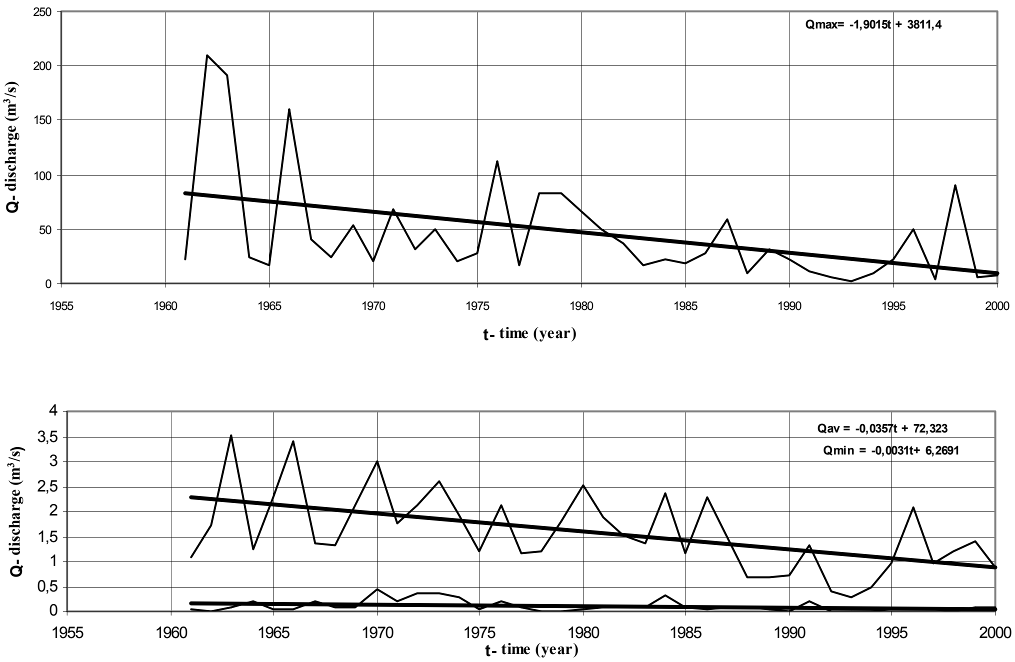

Fig. 2: Annual values of characteristic flows of the river Strumica in Sushevo

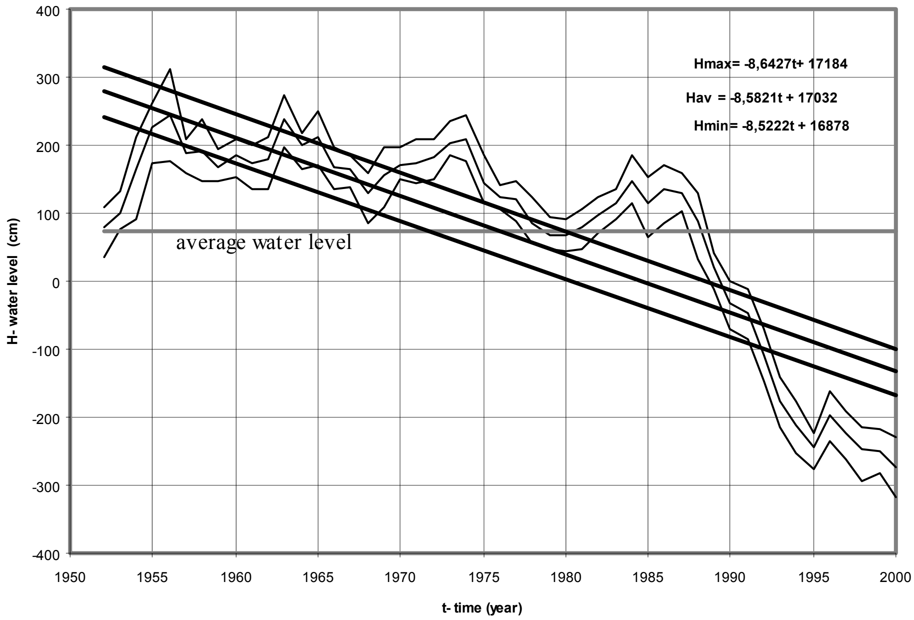

Fig. 3: Annual values of the characteristic water level elevations of Dojran Lake

4 Adaptation Measures

International community is more and more intensively engaged in the investigation and measures taking for prevailing of the climatic changes consequences. Climatic Changes Frame Convention carried by UN, represents member's obligation. Change of the climate and its influence on the hydrology and water resources in Macedonia has not been investigated so far.

Global warmth or so-called "conservatory" influence has greatest influence on the hydrology and water resources and on the life quality in all segments. Also it is common impression, not only in our country but also in global frames, that surface flows, oceans, lakes and rivers are treated as water "bins" with unlimited capacity that even more increases the negative impact of climate change, often with enormous consequences on the living environment.

Hydrological cycle is closed circle and it seems that we have realized too late that acid rains also come from the atmosphere, that poison gases come in the atmosphere and destroy forests, and with it significantly disturb the cycle components, that is the water balance. With ground warming, winds, floods and droughty seasons become more frequent and more severe and planet Earth seems more vulnerable than ever. Inheritance that we will leave will show are we going to learn something from our mistakes in the past and in the present.

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Carried out analyses, conclusions and opinions have no ambition and cannot give final answer on this complex question. Required data for the analyses are historical, hydrological and meteorological data series, and the existing ones are relatively short, often inhomogeneous,

and it is more than obvious that acceptable answers, solid forecasts and appropriate solutions for negative influence reduction will be still waited for a long time.
Inside the National Action Plan for Adaptation, great problem represents the fact that there is large time difference between the climatic changes influence identification moment and the moment when carried out adjustment measures show some results. The required forecasting period in hydrology is mostly 25 to 50 years. Therefore, first step in this direction is certainly adoption of the Action Plan for the impact **c**limate changes at national level. Only afterwards taking of the technical measures for protection of the endangered regions and adaptation of the water management would be possible so the main functions in that area/region will remain. This protection strategy can be impossible or unacceptable due to neighbouring economic and other influences. Therefore, different strategies should be developed and the best, that is the most adjustable one, should be chosen. The choice of most adjustable strategies is possible only if climate changes influences are elaborated in several main segments. Economic: production losses, cost of the carried out measures. Medical: safety, supplying, food quality. Social: unemployment, migrations. Administrative: legal problems, competency, administrative limits. While implementing these important principles in practice, it is very important to provide undisturbed information flow, stable and continuous information exchange, as well as public, professional and open discussion of all partners. The discussed problems in different sectors connected to water resources and hydrology and adaptation measures to climate changes as integral part of the National Action Plan, are systematized as follows.
In hydrometeorology the measures are: modernization of the network, data monitoring establishment of the meteorological, hydrological and water quality parameters, efficient processing of the measured data, implementation of the real time predictive models. In water supply sector the measures are: water losses reduction, placing of pressure and flow meters, implementation of dual supply systems for potable and non-potable water, recycling of water for non-potable uses, rain water collection for non-potable uses. In sewage systems the following activities are necessary: water efficient appliances, waste water purification and their re-use, street and car washing with recycled waters. In agriculture the most vulnerable sector is irrigation and the measures are: covering-lining of the open canals, introduction of the closed conduits, introduction of drip, micro-spray and other low-energy irrigation systems, improvement in measurements, night time irrigation, use of waste water effluent, control and management of the systems. In hydropower systems the adaptation measures are also necessary, such as: keeping the reservoirs at lower head, implementation of more efficient hydropower turbines, construction of additional reservoirs, taking plants off-line in low flow periods, construction of alternative systems, control and management of the systems.

5 Conclusions

Due to its orographic and climatic features, Macedonia falls within the group of countries highly vulnerable to climate change. The sphere expected to suffer most seriously is agriculture because the water demands are increasing rapidly and the same time higher frequency and intensity of dry periods in future is expected. Also it is expected agriculture cultivation to become more expensive due to increased concentrations of CO_2. Most forests habitats in Macedonia will be exposed to stress as a result of climate change. Biotic diversity will suffer as well. Numerous specific, geographically isolated ecosystems, which constitute shelter for endemic species, will not be able to adapt to the changed climate zones.
According to the performed analysis of the hydrological parameters, the following concluding remarks can be carried out: (i) observed serial of data for 40-50 years is insufficient for more

complete analyses and more reliable forecasts for the trends of the hydrological parameters in the future, that is 21 century, (ii) for estimation of the water resources state in present and in future, real data for ground waters are also required, (iii) for all analyzed rivers for the observed period the average discharges have decreasing of 10% to 20%, and for the maximum discharges the reduction is to 80%, (iv) special attention should be given to the measurement, processing and analysis of the real hydrological and meteorological parameters, (v) in order to achieve the last goal, it is necessary to moderate measurement techniques, methodologies of processing and analysis of the influencing parameters on the hydrological cycle.

Climate change will affect almost all domains in the Republic of Macedonia. They certanly foresee harmful influences on the agriculture, such as soil. Yield and livestock producation. The forest areas will decline because of droughts and some specific tree diseases, as well as from fires. The exceptionally rich biological diversiry in many mauntinous and river regions will be affected, so movements of some species can be expected, even their disappearances. The water resoureces will continue to be reduced because of more often and longer drought periods. Human life will be attacked seriously because of the stated vulnerability of the water resources. Therefore, within this project, adaptation measures are proposed. Main purpose of the proposed measures for water resources adaptation with the constructed and/or required infrastructure in Macedonia, is to reduce the water losses in conditions of climate changes, various in time and space. Studies for evaluation of the river basin's sensitivity and water balance components under different scenarios of unfavorable climatic impacts are necessary.

At the same time, Macedonia and its economy will face great pressure from the future commitments specified in UNFCCC. It is therefore important that people, including decision-makers and the population are acquainted with the problem of climate change and with the measures necessary for preventing or mitigating this change.

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The Influence of Landfill "Radosavci" on the Surrounding Soil and Water Quality

Vitomir Premur, M.Sc.[1)]; Prof. Ivan Gotić, B. Eng., Ph.D. [1)]; Emilijan Levačić, B. Eng., Ph.D[1)].

Summary:

In Croatia there are many municipal solid waste landfills that do not meet the requirements of sanitary waste disposal and can in great majority be characterized as waste dumps. Large part of them have been inherited and a little is known about their effects on the surrounding soil and water. The landfill "Radosavci" near Slatina is one of the most typical examples.

This paper presents a detailed description of the landfill "Radosavci", and the scope of investigation works, which tried to get insight into the influence of the landfill on the groundwater and surface water quality and the quality of the surrounding farming soil. The geotechnical structure was investigated, as well as the content of the remarkable pollutants in groundwater and surface soil layer.

The paper presents and discusses all measured data.

Key words:

municipal solid waste, landfill, groundwater, soil

[1)]University of Zagreb, Geotechnical Faculty in Varaždin, Hallerova 7, 42 000 Varaždin, vitomir.premur@vz.htnet.hr; phone: +385 (0) 42 212 228, fax: +385 (0) 42 313 587

Introduction

Solid waste presents imminent result of human civilization. Generally, everything produced by human activity is or will be waste. Our society is not immune to production of solid waste either. Therefore, we should carefully consider its treatment.

One of the developed countries features is an ecologically accepted treatment of waste, whereas non-developed countries are characterized by disposal of waste into dumps.

The basic characteristic of the current state of waste treatment in Croatia is its unsuitable treatment. The 2000 estimate showed there was 1,172,543 t of municipal solid waste produced in Croatia and the trend indicated a 20% growth for the period of five years [1].

In the Virovitica – Podravina County, with population of 93,952, 52% of the inhabitants have been involved in organized waste collection. The estimated 31,274 t of waste is produced in the County per year, which comprises 2.67% of the total municipality solid waste in the Republic of Croatia. The County has four registered landfills, of which none has obtained an operation permit or the necessary documentation. The city of Slatina, with the population of 15,844, has one landfill, i.e. "Radosavci".

The landfill site has been chosen at random. It is situated 2.2 km southeast of the city centre and 1.3 km from the edge of civil engineering zone, just along the northern edge of the national road Slatina – Daruvar (Figure 1). About 100 m further north flows the stream Čađavica, on the other side of the alluvial valley covered with crop fields and grasslands that separate the city and the stream. To the south, behind the mentioned national road there is a small hill covered with wood. In March 2002, the total landfill area was 21,576 m^2, comprising 47.288 m^3 in volume. The average thickness of the disposed waste layer is the lowest along the road edge but it reaches as much as 5m along the western part [2].

The collection of municipal solid waste is conducted by the local public utility company and waste is collected by garbage trucks. So far, industrial waste was also collected but records on types and quantities of the brought in waste do not exist. The landfill is partly fenced. From time to time the waste is spread and compacted by a bulldozer, and is partially covered with inert material, mostly construction waste. The landfill is not monitored and has a completely free access.

Considering the existing solid waste landfills, the greatest attention related to the environment is drawn to the possible pollution of groundwater caused by the landfill infiltrating waters. Even the smallest quantities of the infiltrating water can pollute large quantities of groundwater to the extendt which makes them unsuitable for consumption (drinking) or other purposes. Since infiltrating water from landfills can pollute the underground for many years ahead, potential necessities related to the use of groundwater should also be taken into consideration. This requires risk assessment of the existing depot and, subsequently, a suitable rehabilitation and improvement of the landfill, in compliance with professional and legal requirements.

Basic components to be used in risk assessment of the existing landfill are the following:

- Sources and ingredients of hazardous and dangerous substances in infiltrating waters,
- Transport ways of the infiltrating waters from the source, i.e. the landfill, to the groundwater and further on, possibly including a part of dissolved gases and potential pollutants from the landfill,
- Types and distances of the receptors and users of the groundwater, including people, animals or plants on which possible pollutants deriving from the landfill may have hazardous impact.

This crude solid waste landfill, where undefined types and quantities of different industrial and other hazardous waste can be found, contains various harmful and dangerous chemical substances and pathogenic organisms. Solid waste and infiltrating water of such a landfill can contain primary pollutants, i.e. heavy metals, evaporable organic compounds such as halogenized hydrocarbons and dissolving agents, poly-condensed aromatic compounds, etc. However, it can also contain potential pollutants whose large quantities may endanger human health or decrease possibilities of using such water. This is why concentrates of the defined substances are regularly measured in the eluat and infiltrating water, in order to define the level of pollution, the ways of pollution, necessities for recovery and landfill renovation planning. [3].

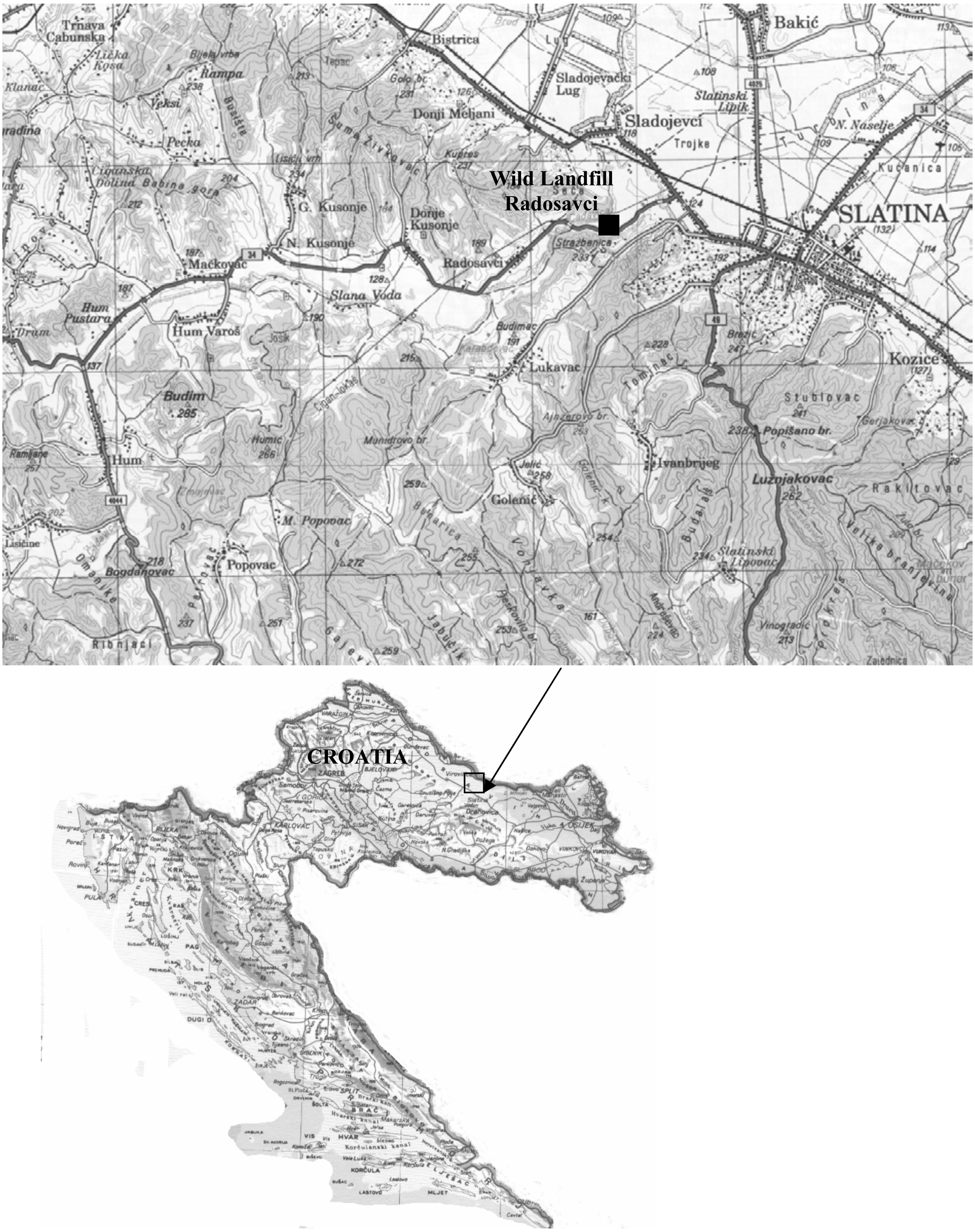

Figure 1- Location of landfill “Radosavci”

2. Investigation works

The goal of investigation works was to define the level of impact of the landfill to the soil and the ground and surface water quality. Type and scope of works were adjusted to the size of the landfill, its relief, estimated geological and geotechnical soil structure and hydrologic conditions.

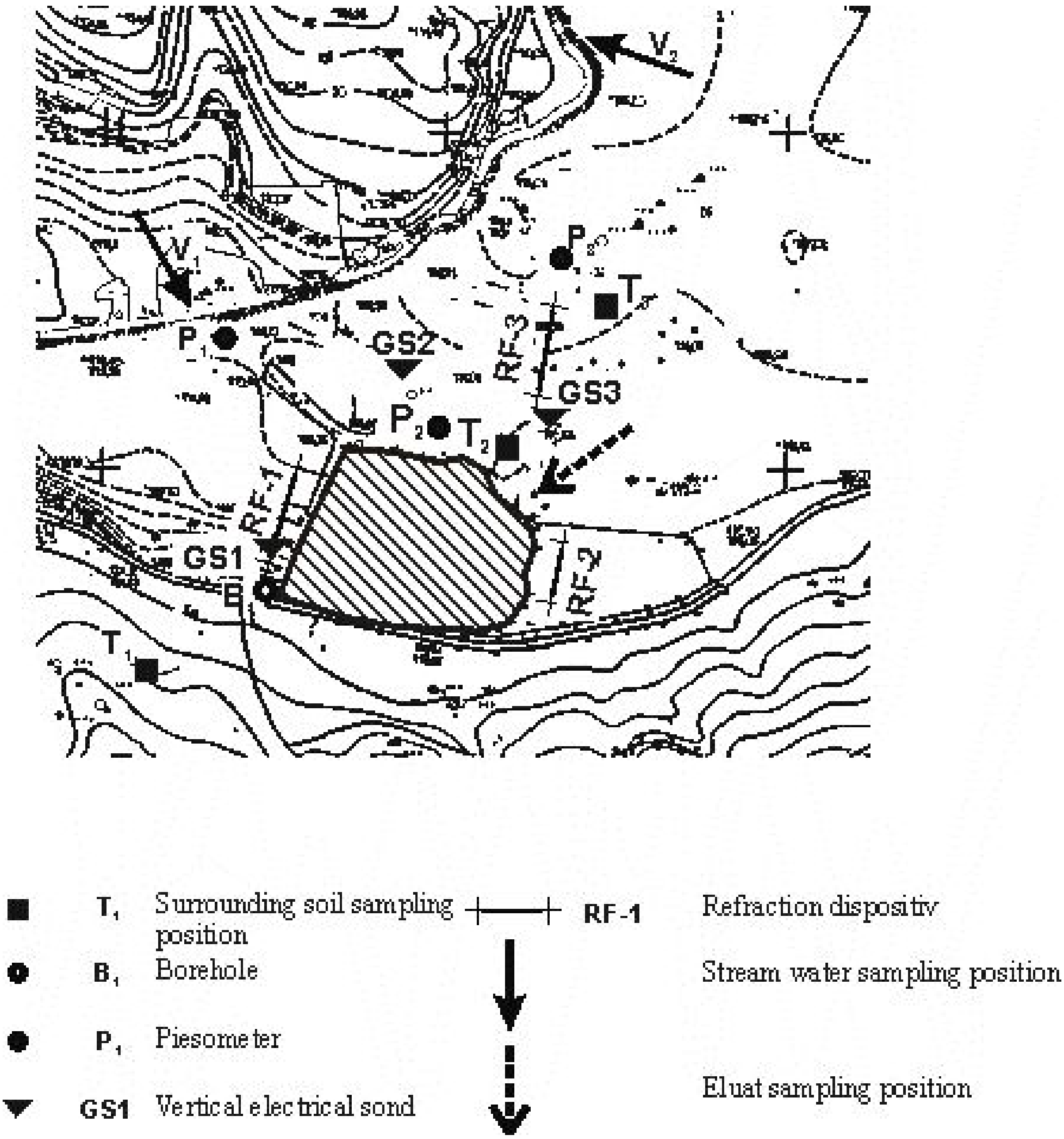

Figure 2- Position of research works on the landfill “Radosavci”

2.1. Surface Soil Layer Analysis

A lot of attention has been paid to the selection of soil sampling site. In order to assess the effect of the landfill on the surrounding soil, three samples have been analysed. One is located on lot T1, up the hill away from the landfill. The second is placed immediately near the landfill on the lot T2 (Figure 1) and the third is further north down the slope on the lot T3 [4].

The third sample has been taken from a slightly elevated place, in the direction of the lowest airflow frequency. In order to avoid the effects of chemicals used in field treatment, the sample was taken from a non-farming area.

2.2. Geological and Geotechnical Prospecting

Geological prospecting was carried out by prospecting of the whole area and was based on core interpretation from the borehole. In order to get insight into the soil structure, comprehensive in situ investigation works were performed. Four geotechnical investigation boreholes were drilled, eight metres in depth. Piesometers were installed in three of them. The location of the boreholes was chosen in order to define geological and geotechnical soil structure and get insight into hydro-geologic conditions. Besides, the piesometres were installed to indicate groundwater pollution and possible pollution grade.

In the course of drilling, the USC field soil classification was performed, disturbed and undisturbed soil samples were taken and tested in a geomechanical laboratory.

Apart from that, geophysical prospecting was conducted as well. The purpose of such prospecting was to define substantial changes in geologic soil structure underneath sampling depth, as a supplement to geotechnical sampling. It consisted of creating three geoelectrical vertical sonds AB/2 of up to 100 m and three shallow refractional seismic profiles, up to 100 m long.

2.3. Water Analyses

For the purposes of analysing the effects of the landfill, water samples were taken from the nearby stream and groundwater.

In order to define the possible effects of rainfall surface run-off and the groundwater from the landfill to the stream water structure, two samples were taken from the stream Cadzavica flowing north in the vicinity of the landfill. One sample was taken upstream - V1 sampling site, and the other was taken downstream – V2 sampling site (Figure 2).

Water coming from the infiltrating ditch in the immediate vicinity of the landfill was analysed and this sample was marked as eluat (Figure2).

Parallel to the analyses of soil and water, samples were taken from the piesometer and the stream Cadzavica, at a considerable distance from the existing landfill up the hill, in order to define the difference in the structure of surface water and groundwater. This could be the basis for defining the possible effect of the existing landfill on the surrounding environment.

3. Results of the conducted prospecting

As already mentioned, the samples taken on site were soil samples for geo-mechanical analyses, surface layer samples for chemical analyses, groundwater samples and stream water samples. Geo-mechanical prospecting was conducted in the geo-mechanical laboratory of the Faculty of Geotechnical Engineering in Varaždin. Chemical and microbiological analyses were conducted in the Institute of Public Health in Varaždin.

The measured values are shown in Tables 1 and 2.

SAMPLE					SOIL SAMPLE TESTS																
Nr.	Borehole	Depth	D	UD	Humidity	GRAIN SIZE DISTRIBUTION Gravel	Sand	Silt	Clay	CONSISTENCY LIMITS Liquid limit	Plastic lastičnos	Plast. indeks	Consist. indeks	Act. by Skempt.	DIRECT SHEAR Cohesion c	Frict. Ang φ	WATER PERMEABILITY koef. of perm. k20 [cm/s] Vertical stress sv [kN/m2] 50	100	200	400	Class. by USC
		[m]			w0 [%]	[%]	[%]	[%]	[%]	wL [%]	wP [%]	IP [%]	IC [1]	[1]	[kN/m2]	[O]					
1	P - 1	1,50 - 1,80			22,20		32,67	48,59	18,37	40,11	19,66	20,45	0,876	1,11			2,28*10-8		1,30*10-8	6,68*10-9	CL
2	P - 2	1,60 - 1,90			23,75		30,09	54,97	14,93	34,04	19,50	14,54	0,708	0,97	3,7	26,1	5,59*10-8		3,52*10-8	2,40*10-8	CL
3	P - 3	1,70-2,00			26,28		9,92	63,10	26,98	48,35	22,14	26,21	0,842	0,97	23,1	22,9		1,15*10-8	8,13*10-9	4,53*10-9	CL
4	B - 1	2,20-2,30			23,39		35,96	50,79	13,24	33,20	19,62	13,58	0,722	1,03	1,4	31,1	1,01*10-7	8,26*10-8		3,93*10-8	CL
5	P - 1	0,30 - 3,20			25,37		14,21	64,48	21,30	39,62	22,01	17,61	0,809	0,83							CL
6	P - 1	3,60 - 5,80			22,18		59,31	34,92	5,45	24,90	18,86	6,04	0,450	1,11							SC-CL
7	P - 1	5,80 - 8,00			24,45		29,69	60,32	10,10	29,25	20,25	9,00	0,533	0,89			1,25*10-7	1,17*10-7	1,0*10-7		CL-ML
8	P - 2	3,60 - 4,00			27,00		24,94	53,05	22,01	39,83	19,19	20,64	0,622	0,94							CL
9	P - 2	0,30 - 3,30			27,04		42,03	39,60	18,37	30,80	18,69	12,11	0,310	0,66							CL
10	P - 2	4,00 - 8,00			22,24		58,15	25,10	6,76	23,10	16,91	6,19	0,139	0,92			1,52*10-7	1,07*10-7	7,59*10-8		SC-CL
11	P - 3	0,30 - 4,40			26,45		10,95	72,18	16,86	37,74	21,80	15,94	0,708	0,95							CL
12	P - 3	4,40 - 5,40			25,07		39,96	51,39	8,64	27,77	19,28	8,49	0,318	0,98							CL-ML
13	P - 3	5,40 - 8,00			24,71		15,24	74,94	9,28	27,95	20,73	7,22	0,449	0,78			3,55*10-7	3,15*10-7	2,56*10-7		CL-ML
14	B -- 1	3,00 - 8,00			24,94		19,28	61,87	18,84	36,75	23,49	13,26	0,891	0,70							CL-ML

Disturbed soil sample (D)
Undisturbed soil sample (UD)

Table 1:Results of geo-mechanical soil sample tests

Table No 2. Results of Chemical and Microbiological Analyses of Water

Parameter	Unit	Samples of Surface Water		Samples of Piezometric Water			Samples of Topsoil		
		V1	V2	P1	P2	P3	T1	T2	T3
Temperature	^{o}C	7,0	7,0	4	10,0	8			
pH value		7,89	8,02	7,81	7,66	7,76			
Conductivityt	µS/cm	467,0	453,0	582,0	687,0	599,0			
Ammonium (as N)	mgN/L	0,165	0,150	291	343	298			
Nitrite (as N)	mgN/L	0,006	0,010	0,105	0,090	0,008			
Nitrate (as N)	mgN/L	0,330	0,325	0,676	0,400	0,324			
Chloride	mgN/L			8,350	46,930	16,420			
Sulphate $mgSO_4^{2-}/L$	mg/L	19,15	2,100	10,54	52,17	22,81			
Calcium	mg/L	73,67	72,07	88,09	104,10	84,88			
Magnesium	mg/L	19,44	18,47	24,30	28,19	23,33			
Iron	µg/L	120,000	181,00	2230,00	1280,00	404,00	10721,00 *	10812,00 *	12000,00 *
Manganese	µg/L	96,8	118,0	3402,00	1400,0	6070,0	820,700 *	530,700 *	628,500 *
Copper	µg/L	5,2	<1,0	<1,0	<1,0	<1,0	1,82 *	2,96 *	7,61 *
Nickel	mg/kg						5,650 *	10,650 *	15,56 *
Zinc	mg/kg						47,72 *	49,94 *	152,70 *
Chromium (Total)	µg/L	<10,0	<10,0	<10,0	<10,0	<10,0			
Chromium(VI)	µg/L						<0,1 *	<0,1 *	<0,1 *
Cobalt	µg/L	<1,0	<1,0	<1,0	89,4	19,0	5,83 *	5,32 *	8,74 *
Cadmiumj	µg/L			<1,0	<1,0	<1,0	<0,02 *	<0,02 *	<0,02 *
Fat/Oil (Total)	mg/L			0,104	0,088	0,108			
Phenols (Total)	mg/L			<0,001	<0,001	<0,001			
Lead	µg/L			163,8	178,5	136,7	130,900 *	126,600 *	628,500 *
TDO	mgO_2/L	7,10	6,36						
BOD_5	mgO_2/L	4,03	5,55						
COD	mgO_2/L	24,00	3,060						
$KMnO_4$	mgO_2/L	7,11	7,90						
TDS	mg/L	233	226						
Aerobic. Mezophyl. Bac.	in 1 ml	30000	20000						
Total Coliforms	in 100 ml	>24000	24000						
Fecal Coliforms	in 100 ml	8000	2000						
Fecal Streptococci.	in 100 ml	0	0						
Suphfitoreduc. Clostridium	in 100 ml	0	0						
Pseudomonas aeruginosa	in 100 ml	0	0						

* Concentrations in mg/kg

Geo-electrical sampling method defined that the soil of the sampling depth, which is approximately 30 m from the surface, consists mainly of materials similar to the surface layer. Soil layer resistance ranges between 8.82 and 49.22 Ωm.

The speed of longitudinal waves expansion, defined by shallow refractional seismic, in the layer up to approx. 4 m in depth was 250 to 450 m/s, and 600 and 800 m/s in a deeper layer.

In the course of drilling, the groundwater appeared at the depth between 1.8 and 3.2 m. In P-2 and P-3 piesometers, the groundwater level was stabilized at the depth of about 1.0 m, and in P-1 piesometer it remained at over 3.0 m. It should be mentioned that the first appearances of water were noticed at the depths between 2.8 m (in P-3 piesometer) and 1.8 m (in P-2 piesometer). This indicates that drilling through the soil was conducted more quickly than the water accumulation through mainly silt containing layers. However, after a longer rainfall season, the level of groundwater was noticed at the depths lower than 1.0 m, in almost all boreholes [5]. Permeability to water was defined in laboratory edometers using the soil samples from the P-1, P-2 and P-3 piesometers and amounted to $1.15 \cdot 10^{-8}$ cm/s and $5.47 \cdot 10^{-8}$ cm/s. In this part of the soil profile vertical transmission can be a little higher, but a horizontal one is extremely low.

4. Results and analysis

Based on the conducted geological and geotechnical prospecting it is possible to conclude that the major part of the investigated location is situated on the alluvial siaposals of the stream Čađavica. Only a minor part of the location, along the access road, is situated on the delluvial and prolluvial disposal of the terrain edge, between the slope built from pleistocene lake and swamp sediments and the sediment of the stream Čađavica.

Delluvial and prolluvial disposals were investigated by the B-1 borehole, located immediately next to access road, on a gentle slope. Under a humus layer which is about 0.3 m deep, and to the ultimate drill depth of 8.0 m, there are amber or brown silt-containing clays, clay and sand-containing silts. According to the results of the laboratory analysis conducted in compliance with the Unified Soil Classification System (USCS), they correspond locally to the CL group and with transfer to the ML group. Along the drilling interval there are slightly marked and graduate variations of grain size distribution structure. The soil is low plastic and low cohesive and of hardly pressable consistency. The plasticity index varies between 13.33 % and 13.58 %, and the consistency index varies between 0.732 and 0.920. In the course of drilling, no water was noticed in the created borehole.

Alluvial disposals of the stream Čađavica on the concentrated location area, were investigated by creating the P-1, P-2 and P-3 piesometers, and were characterized by graduated sequence from sand to clay containing silts, with total thickness ranging between 5.4 m and 5.8 m.

Pleistocene lake and swamp sediments (jb) are the foundation of alluvial disposals. They are presented in grey-blue silts of variable contents of fine-grained sand and a relatively low clay content (up to 10 %). These disposals, being non-coherent, are medium compacted with compaction index of $Dr < 0,7$. However, due to mineral characteristics of the clay in the grain size lower than 4 μm (montmorillonites and hydromicas) they can act as a coherent soil when non-cohesive to poorly cohesive, of hardly pressable consistency and low plasticity. According to Unified Soil Classification System (USCS) they correspond to the transfer type between CL and ML.

The results of physical-chemical and microbiological analysis of the samples taken from the stream Radosavci have led to the conclusion that there is no direct effect of landfill rinsing by rainfall run-offs on the stream water quality downstream the landfill. The sample results from the lots V1 and V2 expressed in physical-chemical and microbiological parametres almost do not differ. An increase of iron and manganese has been noticed, which can be the effect of landfill, but does not have any considerable impact on the stream water quality and does not change its classification in compliance with Articles 3 and 4 Water Qualification Regulation [6].

The effect of landfill on groundwater may be assessed by comparing the results of physical-chemical analysis (tables 1 and 2). Groundwater in the immediate vicinity of the landfill contain slightly more dissolved substances and corresponding higher electric conductivity, but the sample showed higher level of chlorides and sulfates. This was also noticed in the groundwater further away from the landfill, compared to the groundwater sample taken from the piesometer P1. The increased level of calcium and magnesium content under the landfill must be the result of the disposed construction waste. Further down the groundwater flow, the content decreases to its basic value given in Annex 3, probably caused by the disturbed carbonate balance and absorbance and exchange of ions in the soil.

The content of iron and magnesium in the groundwater samples under and away from the landfill has decreased in comparison to the basic content in the sample taken above the landfill. This can be explained by reduction conditions in deeper layers of the landfill and in infiltrating water, and is also indicated by decrease of the nitrite and nitrate part. However, there are no data on ammoniac. Ammoniac will have to be defined in landfill recovery planning. Nevertheless, the increase of manganese content in lower piesometer is surprising. Such increase cannot be explained only by differences in redox potential between iron and manganese and no other information exist to perform a more detailed analysis.

The content of other materials corresponds to those from the above piesometer, except for the surprisingly high level of cobalt concentration. Since there is no data on the structure of waste on the landfill, the source of this metal is doubtful. There is a layer of dust clay spreading underneath the landfill and cations, especially the polyvalent metal cations [7], are primarily adsorbed to clay. Therefore a conclusion can be made that cobalt existed in a form of some anion, which hardly connect to clay, or as the ingredient of some organic complex, either external or resulting from decomposition of a part of organic waste.

The content of lead in the groundwater from all three piesometers is by an order of magnitude lower then the lead content in the analysed soil samples. This should point to large sorption quality of the soil above the groundwater and can be used in explaining decrease in lead concentration on the lower piesometer compared to the increased lead content under the landfill.

The soil sample analyses may show the effect of pollutants expansion in the atmosphere and water. The content of metals has been defined and the results point to the fact that by possible rinsing, i.e. infiltrating of the surface rainfall and landfill water the content of all metals is increased, except of iron and manganese whose concentrations remained almost the same in all three samples. The higher increase of zinc concentration, especially on location T3 (Figure 2), and slight increase of lead on the same location show that eluats rinsing off the landfill may effect soil structure ever further from the landfill. General, although rather small, increase in metal concentrations behind the landfill may be the result of the chloride increase in the eluat and infiltrating water, which could create complexes with metal ions that absorb poorly to clay. The effect does not influence the structure of the stream Radosavci, but can be noticed further on in the sediment structure.

Due to the shown lead concentration in and a certain structure of the analysed soil, and pursuant to Article 3 Regulations for the Protection of Farming Soil from Pollution Caused by Hazardous Substances [8], the analysed soil is of borderline ecological quality.

The analysis of eluats taken from the landfill (Table 2) shows high content of dry substance, resulting from evaporation, and high electrical conductivity, which points to the fact that infiltrating water exclude rather high quantities of dissolvable substances from the landfill. Ammoniac values are considerably higher than nitrites, indicating reduction conditions, corresponding to the results of other analyses. The content share of zinc and copper in the eluat supports the results of the soil analysis on the location T3 (Figure 2), indicating that the increased metal content in soil derives from the landfill.

Pursuant to Article 13 Regulations for Conditions of Waste Treatment [9], the content of eluat is not defined in municipality solid waste but the landfill Radosavci also containes other types of waste. In compliance with Article 12 of the stated Regulations the landfill could be included into the first category of solid waste landfill.

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Detailed Class Parameters of Navigable Inland Waterways - the "Sava Basin Initiative"

Prof. Marko Pršić, PhD C.E.
Duška Kunštek, MS. C.E.

University of Zagreb - Faculty of Civil Engineering, Kačićeva 26, 10000 Zagreb, CROATIA
tel: +385::1: 4639222, fax +385::1:4639260, Email: kduska@grad.hr

Abstract
The «Sava basin initiative» is an international programme for the integral management of the Sava river basin. The member countries are Bosnia and Herzegovina, Croatia, Slovenia, Serbia and Montenegro. The new UN/ECE « Classification of European navigable inland waterways» covering 7 classes was created and accepted in1992. A bit earlier in 1992 the classification was adopted at the CEMT conference. It is based on the UN/ECE classification from 1960 and CEMT classification from 1961 that encompassed 6 classes. The UN/ECE classification from 1960 was created as a recommendation, and the UN/ECE classification from 1992 is a component of the European treaty -AGN from 1996 and therefore it obliges the signatory countries. Only the Republic of Croatia as a member of the Sava initiative signed the AGN. Other Sava initiative members announced to join soon. The UN/ECE '60 classification for the tugged convoy contained detailed parameters of the navigable way. The new UN/ECE '92 classification contains only the sizes of tugged convoys. The directions are given to calculate detailed parameters of a navigable way for the area of the «Sava basin initiative». A table of all detailed parameters for all the classes of a navigable way should be created on the basis of the given directions.

1 Introduction

The paper deals with standardization of detailed class parameters of inland waterways resulting from the main parameters of UN/ECE classification [2] from 1992 for the «Sava initiative» programme. The standardization should enable an easier application of the 1992 classification on the common waterways and their tributaries. The detailed parameters should contribute to the preparation of development plans.

The new UN/ECE classification [2] from 1992 does not contain detailed parameters of European navigable waterways. They should be handled separately due to respective differences in morphology and traffic. Detailed parameters are to be defined according to specific features. The paper focuses on the overall elaboration of detailed parameters within the Sava initiative. It encompasses two major waterways of different international classes: those of Sava and Danube, their tributaries of regional classes that are navigable on a short section around the mouth. The generalization is possible as there are only few navigable watercourses, and no considerable differences take place.

2 Navigation in the inland waterways

2.1 Standard vessels and convoys

Standard vessels and convoys that are going to be used are defined in the UN/ECE classification from 1992 [2,5]. The international class of a navigable waterway is secured by the vessels of the navigable waterway of the IV. class, with dimensions 85×9,5×2,5 [m], or of the larger size (Annex IIIa).

2.2 Determination of the minimal radiuses according to classes of navigable waterways

The minimal radiuses for this classification have been adopted on the base of data analyses on minimal radiuses of the navigable waterways in the bends of the existing waterways and in several recommendations [1, 2, 7, 9, 10]. They are given in the shape of comparative graph of the adopted minimal radiuses within the «Sava initiative» and UN/ECE classification, Fig. 1.

3 Detailed parameters of the inland waterway classes for the "Sava initiative" programme

3.1 The width of the waterway in the direction

The widths of the navigable riverbed for the two-way navigation in direction for all the navigable waterway classes are determined by the analyses of disposable theretical and practical methods. The width structure of the two-way navigable riverbed in the direction in the standard overtaking operation of the downstream navigation is the following :

$$B_{F,r}\,[m] = B_r + 2\times S_S = (2{,}5\times b + 2\times \tan 3^\circ \times l_{max}) + 2\times 0{,}5b = \\ = 3{,}5\times b + 2\times \tan 3^\circ \times l_{max} = 3{,}5\times b + 0{,}1 \times l_{max} \qquad (1)$$

and for the canal:

$$B_{F,k}\,[m] = B_k + 2\times S_S = (2\times b + 2\times \tan 1{,}5^\circ \times l_{max} + 2) + 2\times 1{,}5 = \\ = 2\times b + 2\times \tan 1{,}5^\circ \times l_{max} + 5 = 2\times b + 0{,}05 \times l_{max} + 5 \qquad (2)$$

For the horizontal reserve "S_S" between the navigable gabarit and watercourse profile the following is adopted in this classification:

$S_S=0{,}5\times b$ for rivers

$S_S\,[m]=1{,}5m$ for canals.

where:

- b [m] - is the width of the standard cargo vessel, or the stiff tugged convoy (when the convoy is larger than a vessel)
- l_{max} [m] - is the length of the standard vessel or stiff pushed convoy for the calculation of the navigable riverbed width
- S_B [m] - is the horizontal reserve between two navigable tracks
- S_S [m] - is the horizontal reserve between navigable gabarit and watercourse profile
- B_r [m] - is the width of the two-way navigable gabarit on the river in the direction in overtaking of the downstream navigation
- B_k [m] - is the width of the navigable gabarit on the canal in the direction at passing by
- $B_{F,r}$ [m] - is the width of the two-way navigable riverbed in the direction on the navigable river
- $B_{F,k}$ [m] - is the width of the two-way navigable riverbed in the direction on the navigable canal

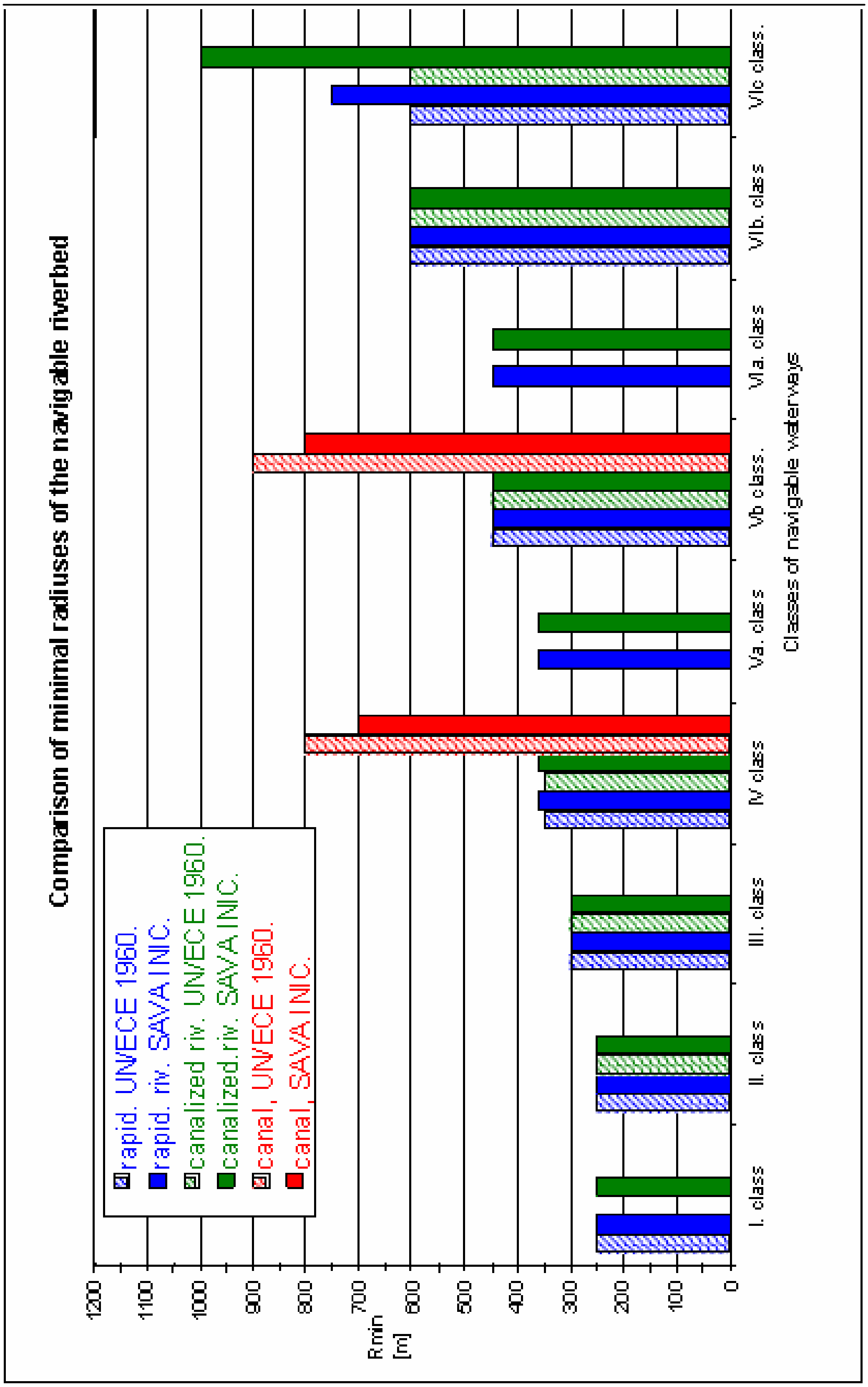
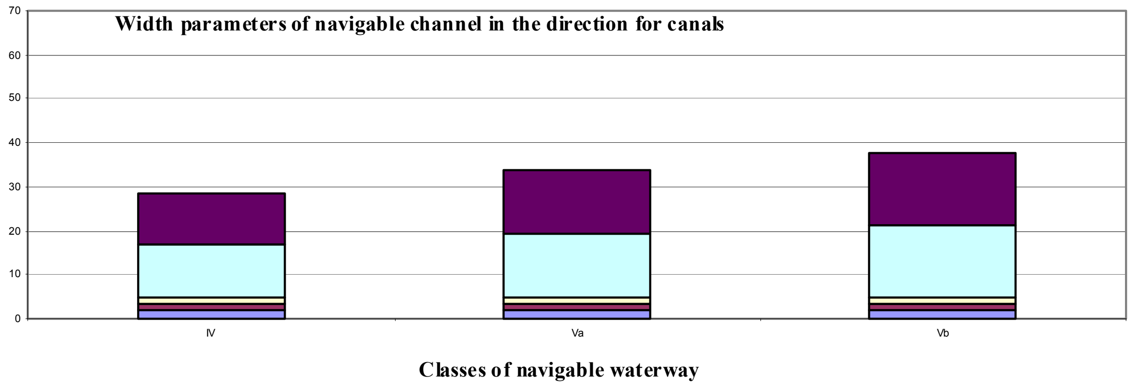


Fig. 1: The comparison of the minimal radiuses of the navigable riverbed in the bend according to the UN/ECE classification from 1960. and UN/ECE classification from 1992 ("the Sava initiative").



		IV	Va	Vb
	Distance between tracks (m)	2,0	2,0	2,0
	Distance to the brim (m)	1,5	1,5	1,5
	Distance to the brim (m)	1,5	1,5	1,5
	Upstream track (m)	11,7	14,3	16,2
	Downstream track (m)	11,7	14,3	16,2
	Total (m)	28,5	34,0	37,5

Fig. 2 Diagram of the final parameter recommendations for the canal navigable riverbed width in the direction, for the IV and V navigable waterway classes.

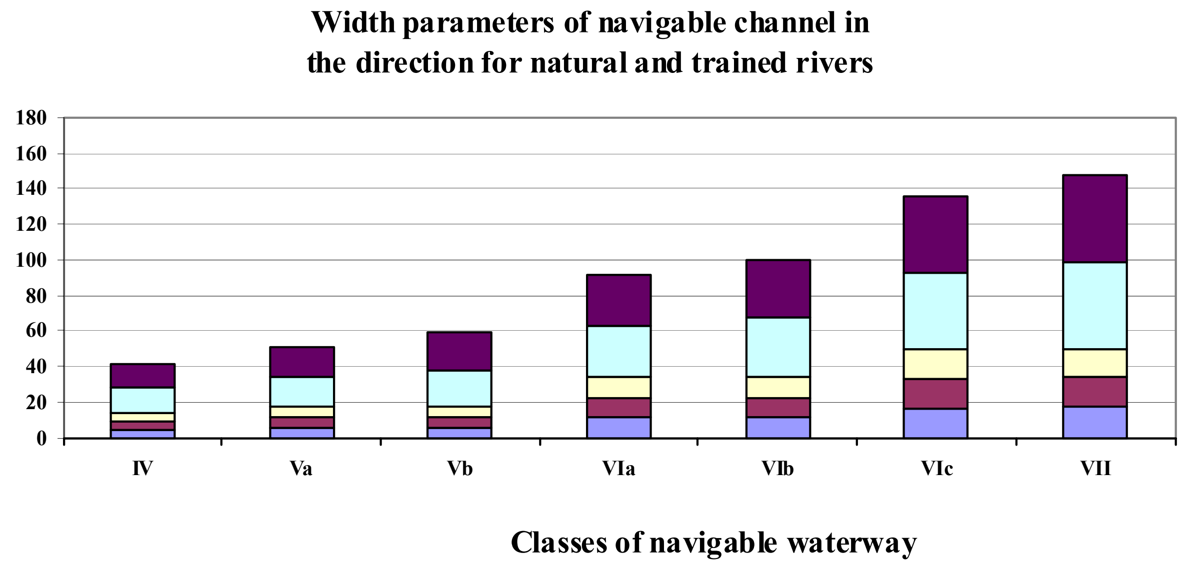
The only exception is the class VIa on the river where the size of the horizontal reserve between two navigable tracks in the direction will be calculated according to the expression: $S_S=0,5\times b$, but for "b" the width of the vessel, and not the convoy width will be taken.

Chyba! Chybné propojení.

The following general conditions were endorsed in this case:- The width of the standard stiff rigid convoy and the width of the standard vessel for the calculation of the navigable riverbed width are defined by the UN/ECE classification [5];- The hydrodynamics of the navigation considerably depends on the narrowness of the navigable riverbed defined by the ratio of the cross sections "n". With the newer and future canals it ranges between n=5,4 and n=6,3, and tends to the optimal value of 7;- The navigation dynamics in the horizontal plane is similar with that of canals and canalized rivers, and is different with rapid rivers, as the canals and canalized rivers contain "still water";- The standard operation: overtaking in the downstream navigation on the rivers and passing by on the canals;- Standard horizontal winding angle β that defines the navigable track width in the direction is : $\beta^1=1,5°$ for the case of upstream river navigation and passing by on the canal, and $\beta^2=3°$ for the case of downstream river navigation and overtaking on the canal;- The standard horizontal reserve size between two navigable tracks in the direction amounts: $S_B=0,5\times b$ for rivers and $S_B=2$ [m] for canals;- Standard horizontal reserve "S_S" between the navigable gabarit and watercourse profile in this classification are adopted as: $S_S=0,5\times b$ for rivers and S_S [m]=1,5 [m] for canals.

3.2 The navigable waterway width in the bend

For I-III on the river



	IV	Va	Vb	Via	VIb	VIc	VII
Distance between tracks (m)	4,75	5,7	5,7	11,4	11,4	16,5	17,1
Distance to the brim (m)	4,75	5,7	5,7	11,4	11,4	16,5	17,1
Distance to the brim (m)	4,75	5,7	5,7	11,4	11,4	16,5	17,1
Upstream track (m)	14,0	17,2	21,1	28,6	33,0	43,2	49,1
Downstream track (m)	14,0	17,2	21,1	28,6	33,0	43,2	49,1
Total (m)	45	55	65	80	105	140	155

Fig. 3 Diagram of the final parameter recommendations for the natural and trained rivers in the direction from IV to VII class

It refers to rapid and canalized rivers I. - III. class with predominant tugged and self-propelled technology. As there are no canals from I. to III. class, neither expected in future, the navigable riverbed width is determined only for rivers in the bend on the rapid and canalized river and is expressed by:

$$B_{F,r,z} = \left[2{,}5b + \frac{l_{\max}^{2}}{2R} \right] + (2 \times 0{,}5b) = 3{,}5b + \frac{l_{\max}^{2}}{2R} \tag{3}$$

where:

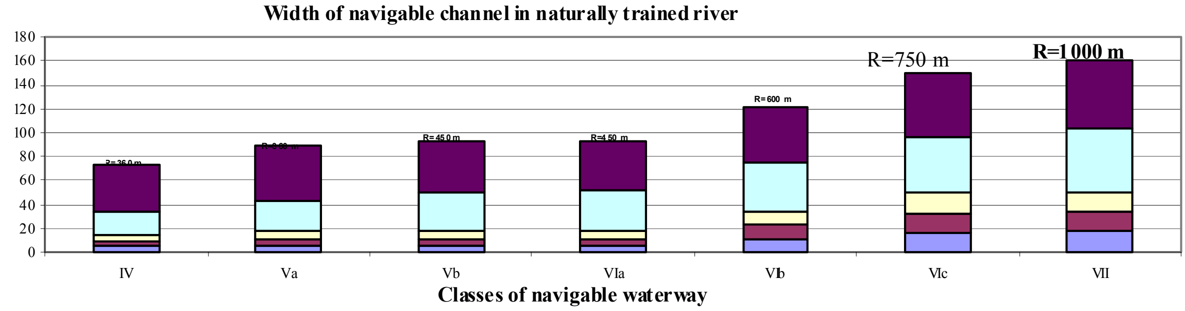
b [m] - is the width of the vessel, or the stiff convoy
l_{max} [m] - is the length of the tugged convoy or self-propelled vehicle
R [m] - is the radius of the navigable riverbed ax

For IV. - VII. class on the river
It refers to rapid and canalized rivers IV. - VII. class with predominant pushing technology. The expression for the final width of the navigable riverbed in the bend is :

$$B_{F,r,z} = \sqrt{\left(R_{i,2}+b\right)^{2} + \left[\frac{l_{\max}}{2} + \left(R_{i,2}+\frac{b}{2}\right) tg\beta_{2}\right]^{2}} - R_{i,2} + \sqrt{\left(R_{i,1}+b\right)^{2} + \left[\frac{l_{\max}}{2} + \left(R_{i,1}+\frac{b}{2}\right) tg\beta_{1}\right]^{2}} - R_{i,1} + 1{,}5 \cdot b \tag{4}$$

where: $R_{i,1}$[m] - is the radius of the inner brim (closer to the center of curvature) of the navigable track B_1 for upstream navigation on the rapid and canalized river on the concave (depressed) bank curve,

$R_{i,2}$[m]	- is the radius of the inner brim (closer to the center of curvature) of the navigable track B2 for downstream navigation on the rapid and canalized river on the convex (protruding) bank curve.
l_{max} [m]	- is the length of the stiff pushed convoy
β_2 [°]	- is the horizontal angle of the swerve of the vessel or the stiff pushed convoy, in the bend for the downstream navigation on the rapid river at the convex (sticking out) bank curve,
β_1 [°]	- is the horizontal angle of the swerve of the vessel or the stiff pushed convoy, in the bend for the upstream navigation on the rapid river at the concave (depressed) bank curve.
b [m]	- is the width of the vessel or stiff pushed convoy (when the convoy is larger than a vessel).



	IV	Va	Vb	Via	VIb	VIc	VII
Distance between tracks (m)	4,75	5,7	5,7	11,4	11,4	16,5	17,1
Distance to the brim (m)	4,75	5,7	5,7	11,4	11,4	16,5	17,1
Distance to the brim (m)	4,75	5,7	5,7	11,4	11,4	16,5	17,1
Upstream track (m)	20,0	26,0	33,0	34,0	41,0	48,0	54,0
Downstream track (m)	38,0	46,0	43,0	42,0	46,0	52,0	57,0
Total (m)	72	89	93	93	121	149	162

Fig. 4 *Diagram of the final parameters recommendations for the navigable waterway width in the bend on the trained rivers, for the IV, V, VI and VII navigable waterway class.*

For any navigable way class the procedure of width detremination of the navigable riverbed in the bend by means of the expression above is iterative.
In this case the following conditions are decisive: - the decisive operation: passing by on both the rivers and canals:- Decisive swerve angle β for the calculation of the navigable river track width in the bend: β_1 for the case of upstream river navigation and β_2 for the case of downstream river navigation, is to be read off in dependence of the radius R_i of the inner (closer to the curvature center) brim of the navigable track in the bend and central angle of the navigable riverbed in the bend α=30 [°], from the Graewe 's experimental diagrams - [8] that were worked out for the stiff pushed convoy 2xE_1+ pusher. For the case of passing by on the canal, the horizontal swerve angle β_1, decisive for the calculation of the navigable river track width in the bend, is expressed in the dependence on the navigable riverbed ax radius and central curve angle α=30 [°] from the Graewe' s experimental diagram [8]. The decisive horizontal reserve size between two navigable tracks in the bend is: S_B=0,5×b for rivers and S_B=2m for canals;- The decisive horizontal reserves "S_S" between the navigable gabarit and the watercourse profile in this classification are adopted as : S_S=0,5×b for rivers and S_S [m]=1,5m for canals.

3.4 The navigable channel depth

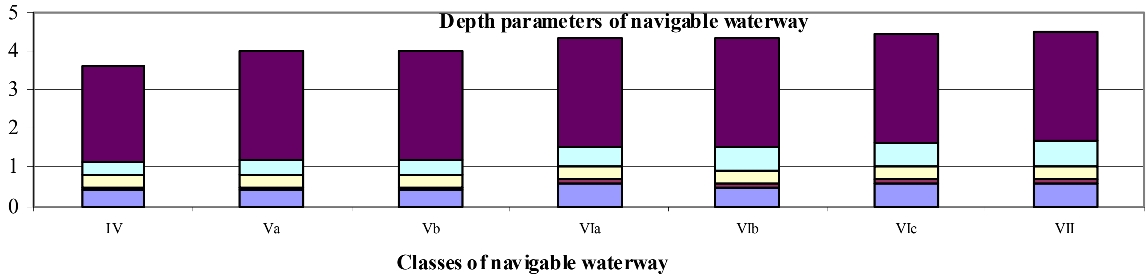
The needed navigable riverbed depth on both canals and natural rivers is determined by: draught of the vessel, longitudinal vessel trim, vessel speed sink, absolute reserves (water cushion between the vessel bed and the navigable way bed) and inexatness of measurements, or the cut of the transversal cross section of the navigable way. [3RICHTLINIEN]:

$$T = T_v + \Delta + \delta = t + \Delta t + S_z + \Delta + \delta = t + C \qquad (5)$$

where:
- T_v [m] - is a navigable gabarit depth
- t [m] - is a draught of a vessel
- Δt [m] - is a longitudinal trim of a vessel
- S_z [m] - is a speed sink
- Δ [m] - is an absolute reserve
- δ [m] - denotes an imprecision of a measurement or a cut of a navigable way cross section
- C [m] - is a water depth below a keel - clearance.

The following general conditions were decisive:- For the depth calculation of a navigable riverbed the regional navigable waterways should secure the draught lower than up to 2,5 [m], the existing international the least of 2,5 [m], and the future international at least 2,8 [m] [2 and 5];- The navigation dynamics in the vertical plane significantly depends on the tightness of the navigable way transversal cross section; consequently it is similar to rapid and canalised rivers, while different from the canals;- The following is decisive for determination of the navigable riverbed depth in two-way navigation: the central navigation in direction and full speed of the vessel or the convoy. The determined depth accounts for the whole width of the navigable riverbed;- The longitudinal trim of a vessel according to experience is adopted with s: $\Delta t = 0{,}1$ [m];- The values of the speed sink S_z for IV to VII class of the navigable way on the canals and rivers are read off or estimated from the Kuhn ` s experimental diagrams [4]. As there are no diagrams for all the classes, the next closest diagrams were used. With the rivers of I, II. and III. class the expression according to Wiegleb was applied [6]; - The absolute reserve for I. to IV. class is taken as: $\Delta = 0{,}3$ [m], for V. class. = 0,4 [m], for VI. a and b class $\Delta = 0{,}5$ [m], and for VI. c and VII. class $\Delta = 0{,}6$ [m]; - The arbitrary size $\delta = 0{,}3$ [m] is taken as a reserve due to the imprecision of a measurement.

The reserve due to the inexactness of the cut of the navigable way transversal profile in the subject area is taken as s $\delta = 0{,}3$ [m];- The recommended navigable way depth for the full draught is to be secured at $V_{65\%}$ AGN [5];- In this classification, it is recommended for shorter rapid rivers that reach the depth of the navigable bed for the full draught only at high water levels with the NVP (that matches the water level of 95% durability) to lower the navigable bed depth up to 60% of depth calculated for the full draught according to;- It is assumed that canalised rivers, built as artificial watercourses intended for navigation, should guarantee a safe navigation with a full draught with NVP, which corresponds to the water level of 95% of durability;- Here is adopted that canals as the artificial facilities serving to navigation should be built to guarantee safe navigation at the full draught "at each moment "; that means, with the NPV that suits the constant water level in the canal.



	IV	Va	Vb	Via	VIb	VIc	VII
Speed sink (m)	0,4	0,4	0,4	0,6	0,5	0,6	0,6
Longitudinal trim (m)	0,1	0,1	0,1	0,1	0,1	0,1	0,1
Inexactness (m)	0,3	0,3	0,3	0,3	0,3	0,3	0,3
Absolute reserve (m)	0,3	0,4	0,4	0,5	0,6	0,6	0,7
Draught (m)	2,5	2,8	2,8	2,8	2,8	2,8	4,5
Total (m)	3,6	4	4	4,3	4,3	4,4	6,2

Fig. 5 Diagram of the final parameter recommendations for the navigable riverbed depths, for the IV to VII navigable waterway classes.

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The Macedonian Regulatory System n Water Towards EU Acquis Communautaire

EU PROJECT - STRENGTHENING THE CAPACITY OF THE MINISTRY OF ENVIRONMENT AND PHYSICAL PLANNING

Bernhard Raninger, Prof. PhD, International Team Leader, GOPA worldwide Consultants, Bad Homburg / Germany; Mining University of Leoben / Austria'
cmepp@moepp.gov.mk

Peter Klein, GTZ – German Technical Cooperation,
International Water Resources Expert, gtzwater@mt.net.mk

Summary

On the basis of the Stabilisation and Association Agreement (SAA) signed in 2001 FYR Macedonia is moving closer to EU integration. One of the aspects of the SAA is the harmonisation of environmental legislation and the adoption of environmental standards. The quality of the environment in the country, particularly in the larger cities and in industrialised areas often does not meet EU requirements. Macedonian national environmental regulations and their enforcement are currently rather weak, and the responsibilities of the institutions are often overlapping and unclear. Unfavourable environmental conditions apply most of all to surface water, to air quality, to poor waste and particularly hazardous waste management practices, to the enforcement of nature protection, to the implementation of environmental management systems, to the prevention of industrial pollution or to the minimisation of environmental impact during planning and construction measures. In order to contribute to the development of the environmental sector in fYR Macedonia, the European Union (EU) is funding an ambient air monitoring network, water monitoring infrastructure, elimination of environmental 'hot spots' and management plans for transboundary waters such as River Varda and Dojran Lake. EU has further provided funds for the implementation of the project *"Strengthening the Capacity of the Ministry of Environment and Physical Planning",* the main tasks of which are to transpose the key environmental legislation and strengthen the performance and visibility of the Ministry of Environment and Physical Planning (MEPP), as the key beneficiary. The Water Regulative Systems plays in terms of approximation of legal instruments as well as from the intuitional complexity an important role.

1. Objectives of the project

The overall objective of the project is *"to support the country in the long process of solving its environmental problems"*: The immediate objectives of the project are as follows:

- Improve the quality of the current environmental legislation, and draft acts which will supplement the existing Act on Environment, thereby supporting the Ministry's efforts to adapt and complete its current legislation to the Acquis Communautaire;
- Propose a permit and enforcement structure which considers environmental protection in a sustainable way during planning, implementation and operation;
- Improve the level of functioning and efficiency and enhance the MEPP's performance;
- Strengthen the Ministry's position in relation to other Ministries;
- Improve communication between stakeholders in the field of environmental management;
- Reinforce institutions in charge of environmental awareness raising;
- Streamline the MEPP's tasks in the field of environmental monitoring and data management in order to manage water resources, solid waste disposal, soil protection, effects from noise emissions and the aspect of biodiversity.

2. Beneficiaries of the project

The Ministry of Environment and Physical Planning (MEPP) is the key beneficiary of the project. The creation of the MEPP out of the Ministry of Urban Planning, Construction and Environment in January 1999 is seen as a milestone in the process of unification of environmental institutional arrangements in Macedonia. However, there are still overlapping responsibilities with other Ministries, notably the Ministry of Agriculture, Forestry and Water Economy (MAFWE), the Ministry of Health (MOH) and the Ministry of Transport and Communications (MOTC), the Ministry of Economy over various environmental issues, and the relatively weak position of the MEPP, in comparison to these other longer established ministries, indicate the present inadequate implementation of the legislative framework and its low level of enforcement.

3. Project Targets

The main targets of the project are allocated to 9 working groups:

The adaptation and approximation of Macedonian Environmental Legislation to the Acquis Communautaire of EU is the scope of work of Component 1 and the working groups 1-4. WG I 'horizontal legislation' deals with 'Environmental Impact Assessment - EIA, Strategic Environmental Impact Assessment - SEA, Integrated Pollution and Prevention Control – IPPC and Access to Environmental Information'.
The project is further working on the sector environmental legislation regarding water management (in WG II), waste- and hazardous waste management (WG III) and nature protection (WG IV). The legal approximation and harmonisation process will be carried out in inter-ministerial working groups, composed by in total app. 180 members. These members are nominated by their institutions having the mandate to contribute to this consultative process of revising the existing MK legislation or to draft the newly required laws and regulations.

This process is based first of all on a comprehensive analysis of the MK and EU legislation, the so-called gap analysis, further on an analysis of the institutional responsibilities in regard to the relevant sectors and includes finally the drafting of new laws and regulations.

WG I/2 deals with the preparation of a 'Master Plan to phase out leaded petrol' in line with the Pan European Strategy and in order to speed up the process at national level.

WG 5/1 is to develop strategies in order to improve environmental communication between the environmental stakeholders on horizontal and vertical, institutional, local, national and international level.

Environmental awareness raising and the preparation of short- and medium term strategies to strengthen inter-ministerial communication and the relation with relevant stakeholders and to the general public is the subject of WG 5/2. The project will prepare and implement short-term interventions to raise the public environmental awareness including special subject campaigns. An Eco-bus will be established as a tool to carry out these environmental campaigns. The so far identified main topics will promote the general new visibility of the MEPP, the use of unleaded petrol and the phasing out process of leaded fuel, the use of bicycles in combination with a concept for a bicycle plan in Skopje as one subtask of the project.
The development of an environmental monitoring- and an environmental data management strategy to streamline and coordinate data collection and management and to disseminate environmental information to decision makers as well as to public, is the subject of the WGs 6/1 and 6/2.

4. Water Management Legislation, subject of Working Group 2

The water management sector is usually one of the most relevant and complex ones and in regard to the subject of this conference some findings and targets of the Working Group 2 on water management may be presented in more detail:
Apart from water pollution, the fYR Macedonia is struggling with many problems related to water resources, including those from floods (just recently in 2002), general water shortage and droughts (depletion of Dojran Lake and Prespa Lake), inadequate irrigation systems, people suffering from insufficient access to clean water supply and sanitation. All of these problems, - in addition to the recent period of years with decreasing rainfall - have increased the need for formulating a clear water policy and a related strategy that could lead to a rational and conservative management of the water resources. So far there has not been a clear picture of the key water-related policies, or a formulation of the water resources strategy of the country.
Besides the upgrading of the National Environmental Action Plan (NEAP), the formulation of a water resources strategy is an urgent need, the latter as a set of medium to long-term action programmes to support the achievement of the development goals of the country and to implement the water-related legislation. This should be effectuated within the framework of a National Water Management Base to be established under the stipulation of Article 20 of the MK "Law on Waters".

4.1 Sources of the Macedonian and the European Legislation

The Macedonian legislation in force on water related issues was established over the last decade in line with the transformation process of the country. Apart from regulations dealing directly with water management, there are important links with environmental or other sector or horizontal legislation that affects the institutions and procedures in the water sector. Overall, a number of 21 laws and draft legal documents that deal with water issues were identified. Additionally, 13 ordinances (books of regulations) and a number of decrees and decisions constitute the national regulatory framework on water.

The main legal documents to be regarded in respect of the irrigation sector are:

- Law on Waters (enacted 01/1998)
- Law on Agricultural Land (enacted 06/1998, amended 03/1999)
- Law on Water Communities (Proposal 05/2002)
- Law on Water Management Organisations (Draft under discussion)

Apart from national laws, some International Conventions or Agreements are sources of the water legislation as well. There is an agreement on transboundary waters with Greece on downstream water courses and jointly shared lakes, which originates from the former Yugoslavia.

The European water policy is implemented since the early 1970´s through two different policy implementation instruments: Environmental Action Programmes and EU Water Legislation. In parallel to six action programmes, the legislation was developed in three phases. Hence water is one of the most comprehensively regulated areas of EU environmental legislation.

Early legislation began in a "first wave" during the 1970s, including

- Surface Water Directive (75/440/EEC) and its daughter directive (79/869/EEC)
- Bathing Water Directive (76/160/EEC)
- Dangerous Substances Directive (76/464/EEC) and its daughter directives
- Information Exchange Directive (77/795/EEC), as amended
- Fish Water Directive (78/659/EEC), amended by (91/692/EEC)
- Shellfish Water Directive (79/923/EEC), amended by (91/692/EEC)
- Groundwater Directive (80/68/EEC), amended by (91/692/EEC)
- Drinking Water Directive (80/778/EEC) amended by (98/83EC)

The second wave of legislation followed a review and amendment of the existing ones and identification of necessary improvements and gaps to be filled. This phase of water legislation addressed protection of the waters related to various sources of pollution, including emission of dangerous substances, urban and industrial waste water as well as nitrate and other products polluting the waters through agricultural activities:
Nitrate Directive (91/676/EEC).

Urban Waste Water Treatment Directive (91/414/EEC)
The second wave of legislation meant that many stakeholders and interest groups found themselves digging in water-related legislative proposals. Eventually, the number of directives had grown to fifteen and more proposals were laid on the table. Consequently, the EU Commission reacted to the need for a more global approach, so as to integrate the

fragmented pieces of legislation in a restructured and comprehensive manner. After a wide-ranging and long consultation and negotiation process of over 5 years, a new Water Framework Directive has been put into force by 22. December 2000.
One significant advantage of the framework approach is that it rationalises the EU water legislation, replacing seven of the old directives, those on surface water and its related ones for monitoring, the fish water, the shellfish water and the groundwater directive, furthermore the directives on dangerous substances discharge and that on exchange of information. The operative provisions of these directives are taken over in the framework directive, allowing them to be repealed in a phased approach over many years. All transposition and implementation considerations concerning these old directives shall therefore be taken into account in the approximation process.
Thus, after this streamlining of the EU water legislation, the concept for approximation as concerns the water resources would deal with the following directives as a base line:

- Water Framework Directive (2000/60/EC)
- Bathing Water Directive (76/160/EEC)
- Drinking Water Directive (80/778/EEC) and its revision (98/83/EC)
- Urban Waste Water Treatment Directive (91/271/EEC), amended by (98/15/EC)
- Nitrate Directive (91/676/EEC)

Apart from the inter-links within the above water related directives, a large number of other EU legal documents are linked to the water sector.

4.2 The Gap Analysis

EU Directives are to be understood as instructions to the Member States to amend their national legislation in such a way that the provisions of the respective directive are realized. As such, they represent binding legal instruments concerning objectives and provisions to be achieved, but leaves it to the national authorities how to incorporate them into their legal system. In some of the above directives, the objectives and measures are formulated in set terms of technical standards. The so-called compliance or gap analysis between the European and the Macedonian legislation shall reflect in how far both the provisions and the defined technical standards of the EU directives can be found in the various Macedonian legal documents and regulations.

The European Water Framework Directive, being the core legal document, represents a policy instrument which seeks to achieve (a) good status for all waters within a set deadline, (b) to apply an integrated approach based on river basin management and (c) to protect all waters, addressing both quantity and quality issues. It brings together elements from a range of individual measures based on a combined approach of different strategic essentials. The overall legislative setting represents an integrated and comprehensive approach towards a water policy on clean water over the whole EU territory. It includes the following objectives:

- prevent further deterioration, protect and enhance aquatic ecosystems and wetlands;
- promote sustainable water use based on long term protection of available water resources;
- protection of aquatic environment from discharge and accidental losses of pollutants;
- reduce groundwater pollution;
- contribute to mitigating the effects of floods and droughts;
- achieve the objectives of international agreements.

Among the many Macedonian legislative instruments dealing with water-related matters, the Law on Waters is the central one. This core legal document was enacted in January 1998, replacing the Law on Waters of 1981. The need to establish a new Water Act is derived from the provisions in the Constitution: all natural resources are goods of common interest and enjoy special protection. All waters are defined as national assets and are property of the State.

The Law on Waters is comprehensively conceived; it regulates the water management along plausible principles and convincing precepts. From the point of view of contents, it covers essential features of a modern water management policy as it is developed in the EU Member States and internationally. This refers to the principle of "sustainable development" and other strategic elements which respond conceptually to the EU water policy and which would serve as a good staring point to approximate the EU legislation, including a general response on:

- water management based on river basins;
- protection of the waters;
- flood control and provisions on droughts;
- water quality objectives;
- getting the prices right;
- getting the people involved.

However, water quality is not addressed in particular definitions or standards, except for drinking water. There is no indication what programmes or measures to be taken in order to safeguard aquatic ecosystems and to meet water quality standards. Apart from this, large parts of the regulations are merely of a descriptive type without regulations of the legal consequences, or they are very general of non-normative nature or in the type of framework regulations.

Thus notwithstanding the appropriate basic concept, the Macedonian Law on Waters reflects in large parts merely general compliance with the provisions of the EU legislation. The relevant stipulations are little precise and effective. Also technical aspects and standards as defined in Annexes of the EU-Directives on water do not appear for the most part in the Macedonian regulatory system. The gap analysis revealed that a major part of the Macedonian water legislation would have to be modified in order to approximate it to the EU legislation. Transposing of the EU provisions into the Macedonian legislation will probably require the processing of a new law rather than a modification/amendment of the existing water-related laws.

4.3 The Institutional Setting

The EU legislation contains little provisions on the shaping of the water management institutions, the organizational structuring of the water sector and their procedural arrangements. The EU provisions to be transposed refer mainly to responsibilities between the Member States and the EU-Commission.

In contrast, the Macedonian laws regulate also the set of organisational and administrative arrangements as to formulate, implement and enforce water policies, laws, strategies and programmes. The various water-related legal documents reflect a criss-cross pattern of overlapping as concerns competencies and responsibilities, including authorisations, licensing, monitoring, reporting, inspection and enforcement:

- according to the Law on the Organisation and Operation of the State Administrative Bodies and the Law on Waters, the overall competence and responsibility for water resources development and management is vested in the Ministry of Agriculture, Forestry and Water Economy (MAFWE). Relevant administrative bodies within the MAFWE are the Water Management Administration and the Administration for Hydro-Meteorological Affairs;
- according to the Law on the Organisation and Operation of the State Administrative Bodies and Law on Environment and Nature Protection and Promotion, the competence for monitoring the conditions of the environment and protection of the water against pollution is vested in the Ministry of Environment and Physical Planning (MEPP). A relevant administrative body within the MEEP is the Inspectorate for Environment;
- the competence for health protection, water pollution, and protection of the population against water pollution as well as affairs of food products (drinking water) is vested in the Ministry of Health (MOH);
- the Ministry of Transport and Communication conducts affairs related to internal navigation and municipal water supply and waste water collection and treatment;
- the Ministry of Economy (MOE) conducts affairs as concern the water use for hydro-power generation and tourism;
- the Ministry of Local Self-Government conducts affairs of the development of hydraulic infrastructure in under-developed regions.
- according to the new Law on Self-government, the competence on water supply, sanitation and waste water treatment of the municipalities is vested in the relevant municipality. Also the measures for protection and pollution control fall under the responsibility of the municipalities;
- the Ministry of Foreign Affairs conducts activities concerning establishment, development and coordination of relations of the country in international affairs.

As concerns more specifically the irrigation sector, the institutional setting and the organisational arrangements of the national legislation failed to function appropriately:

- there is no clear distinction between the regulative administration (i.e. policy formulation and issuing of subordinate regulations, data collection and resources inventory, national development planning as well as permitting/licensing, monitoring and supervision) and the operative service administration (developing the water resources, operate and maintain the hydraulic infrastructure and make the water available to the users);
- the established central Public Water Management Enterprise (PWME) does not function as an effective mechanism of water supply services; this is because the overall financial consolidation of the regional predecessor-organisations and their restructuring got stuck by legal implications;

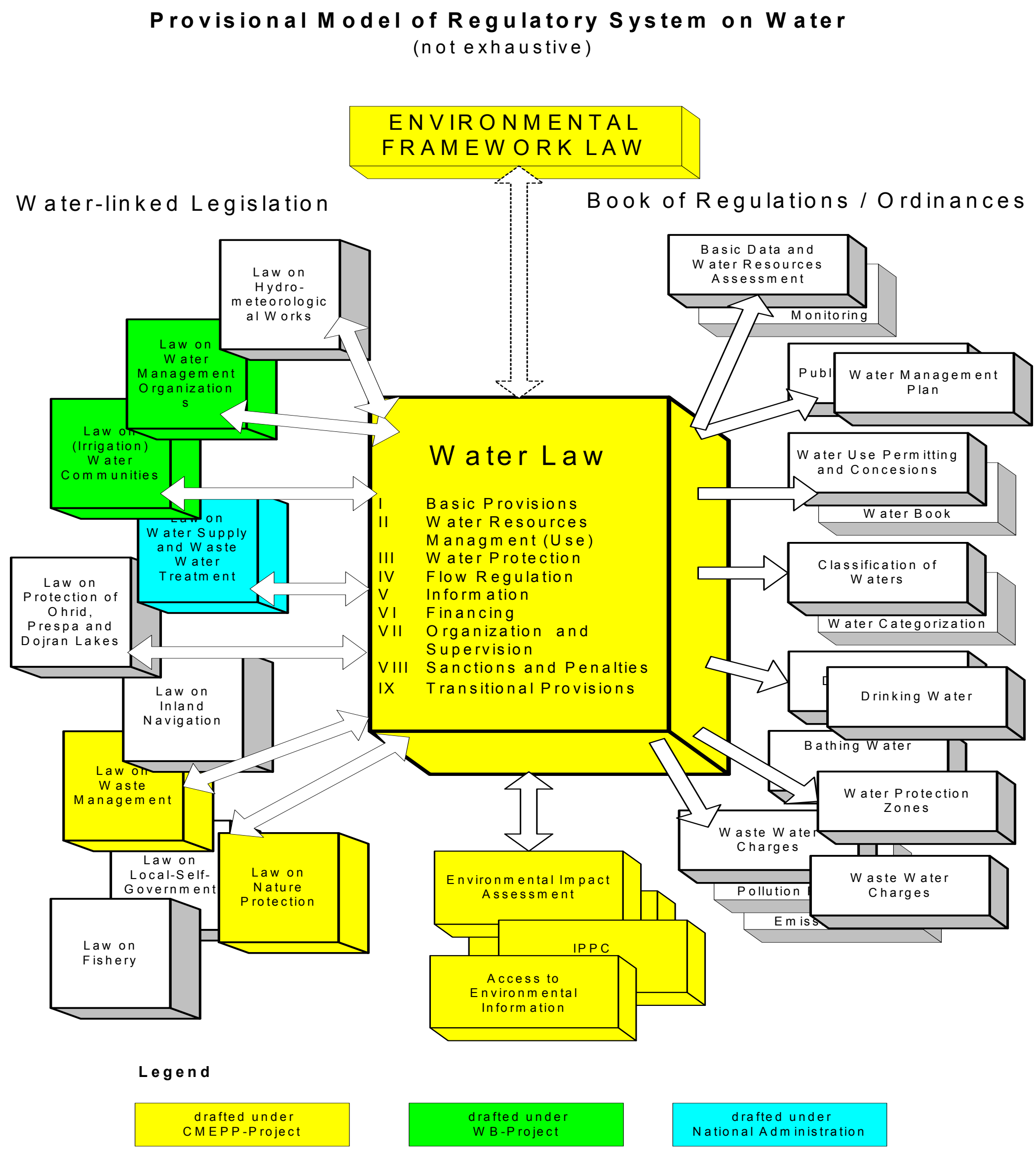

Figure 1: Water Regime – Regulatory Framework on Water

- the original concept of the water management organisations to concentrate on their water-service mandate, to divest side-business, to reduce operation cost and personnel was not implemented;
- the setting of powers, responsibilities and accountability is not adequate, unclear or overlapping;
- the outcome from funds-raising of the Water Fund is not sufficient to serve as efficient instrument for cost recovery, including environmental and resource cost (not debating of course the basic concept behind this economic instrument to promote rational water use.)
- the mandatory obligations of the state administration are deficiently implemented and enforced.

Seeing the inadequate functioning of the irrigation sector, there has been since some time a discussion on the stakeholder level concerning the restructuring of the water management institutions. A new lex specialis has been drafted and brought into the parliamentary process concerning Water User Associations; another lex specialis is proposed for Water Management Organisations. These reorganisation arrangements are mainly driven by the World Bank under the "Irrigation Rehabilitation and Restructuring Project" (the actions being oriented more to agricultural use of waters rather than to a comprehensive approach).

The restructuring of the irrigation sector in line with an overall reform of the water sector will entail the need for amendments in the Law on Waters; at the same the law needs *per se* streamlining and modification.

4.4 The Need for a Comprehensive Approach

Although the EU-Legislation focuses on water quality aspects, it incorporates rational use and quantity objectives as well as other issues of water management. What does water management mean? In a comprehensive sense, water management is to be understood as "appropriate allocation of the natural water resources in time and space, with regard to quantity and quality and according to the needs of the society." In this sense there are three main action fields for legislative regulation:
the water use (e.g. drinking, irrigation, industry, hydro-power, biological demand);
water flow control and flood protection;
protection of the waters.

Three dimensions: water quantity, water quality as well as water morphology would fall under the conceptual goal to establish a comprehensive regulatory mechanism. Similarly, the three action fields should not be dealt with in separate legal acts but in an integrated manner, in order to serve in type of an overall road map. It makes sense to incorporate the framework of the institutional structures which would deal with the different action fields.

Due consideration should be given to a systematic approach when modifying the legislative instruments on water. In order not to burden the political decision-making and the legislative process with many details of a bulky regulatory scheme, it appears to be advisable to provide in the first step a framework type of law. This should take account of streamlining the current legislative instruments enacted or being drafted, including such provisions to approximate the EU legislation. It would deal with substantive principles and rules, which should set forth in secondary legislation. Complementing the basic water law step by step with subsidiary acts, by-laws, ordinances and technical standards would later constitute a comprehensive regulatory entity. This approach would also allow a step-by-step alignment instead of a full implementation of the EU standards in the target-degree of approximation.

5. Conclusions

The project is building the ground for further development of the environmental sector in fYR MK and will contribute to the implementation of environmental infrastructure such as waste water treatment plants, engineered landfills, waste recycling and other cleaner production and pollution control technologies, as well as an appropriate environmental monitoring infrastructure including the communication of the information in the decision-making process and the provision of this information to the general public.

The project so far showed that, notably in the legal approximation component of the project, additional tasks indispensable for the accomplishment of the project objectives are recommended to be undertaken. This relates notably to the drafting of additional primary legislation, without which secondary legislation harmonisation cannot be addressed. The development of a framework law for waste management under which approximation to EU hazardous waste legislation can be addressed is one such example. It also entails the addition of legal approximation work on spatial planning without which approximation of other horizontal legislation for example on Strategic Environmental Assessment remains incomplete and unenforceable. The transposition of the SEA Directive can reasonably be done only through incorporating the relevant rules to consider the environmental impact of building activities and planning in the spatial planning law.

A by-product of the project is to identify new relevant projects, to support the ministry in identifying new project tasks to be prioritised in order to bring forward and accomplish the environmental managements and controlling tools in the country. Therefore, a new project to establish new waste management feasibility study and develop a national waste management plan is already under implementation. Subject of future projects will be the harmonisation of the legislation not touched so far by this and other projects, such as the chemical law, EMAs and Eco-labelling.

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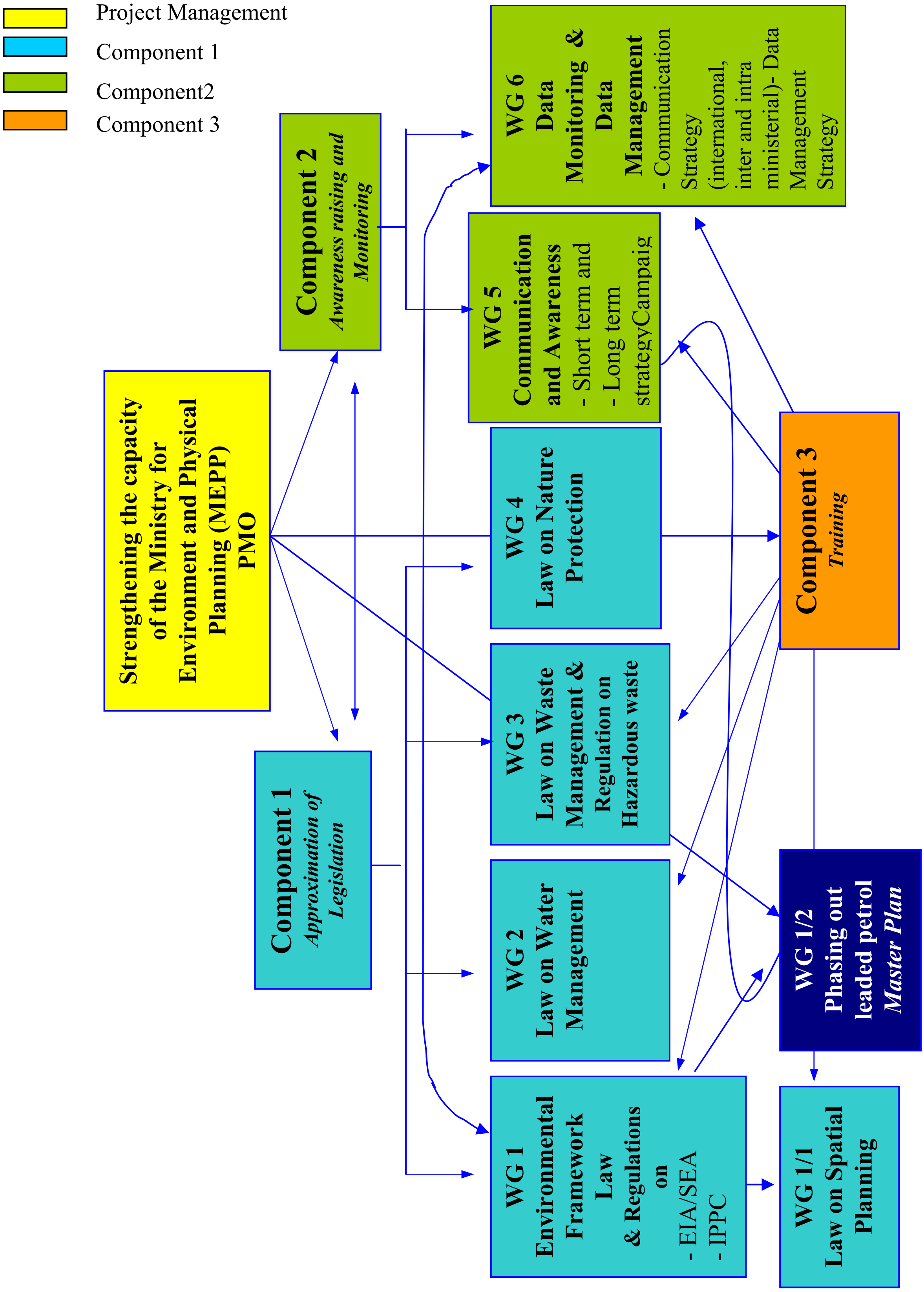
Figure 2: CMEPP Project Organization and tasks



The Discussion on the Uncertainty in the Dam Breach Peak Discharge Estimate

Jaromir Riha[1]

Earth dams are the most common type of dam constructed around the globe as well as in the Czech Republic. Recently, across the world, there have been instances of dam failure resulting in loss of life and property. As a result of the severe floods in the years of 1997 and 2002, the dam safety problem has become topical, not least in the Czech Republic, where more than 100 small dams and levees failed and about 10 large dams were seriously endangered due to overtopping. Overtopping of the earth dams is one of the most common modes of their collapse. The knowledge about the potential trend of breaching in terms of failure duration and peak breach discharge are of principle importance in the case of evacuation activities and hazard planning of potentially endangered areas.

For the maximum breach discharge and the outflow hydrograph estimate, several techniques are used. Some of them issue from the empirical formulae. Others use the numerical modelling of dam breaching due to overtopping. In the paper the variability of modelling results and their analysis are discussed. At the same time, the results of the case study of the potential failure of the Slusovice dam are shown.

1. Introduction

In the case of dam failure, property damage is certain, but loss of life can vary dramatically with the inundation area morphology and its extent, with the number of the floodplain population at risk, and the amount of warning time available. Some authors [1] report that the average number of fatalities per dam failure is 19 times greater when there is inadequate or no warning. In order to assemble emergency flood warning plans, detailed information about the warning time, inundation levels and velocities at downstream locations is necessary.

Part of such emergency action plan are the dam flood inundation maps, which have to be compiled for the area located downstream of the dams. These maps are based on results of comprehensive dam break flood propagation modelling in the area attacked by the dam breach wave. The basic input boundary condition for the unsteady flow calculation is the breach discharge hydrograph at the dam profile. The breach parameters are estimated by various methods like both empirical formulas and sophisticated numerical models based on sediment transport capacity of the flow. However the peak discharges and failure duration obtained can vary considerably and one must deal with the uncertainty of the results. The final statement about dam breach hydrograph should assure reasonable safety of the area covered by the corresponding inundation map. On the other hand it must not provide unreasonable "overdesign" of the potentially endangered area and increase the population at risk (necessary to be evacuated) and the final corresponding expenses. These considerations are also an essential part of the risk analysis of the dam failure and associated floodplain.

[1] Water Structures Institute, Faculty of Civil Engineering, Brno University of Technology, Žižkova 17, 662 37 Brno, Czech Republic, tel. 420 5 4114 7753, fax. 420 5 4114 7752, e-mail: riha.j@fce.vutbr.cz

2. Dam breach modelling techniques

There are various methods available today for analyzing dam failures and their resulting outflow hydrographs, eventual breach parameters like peak breach outflow and failure duration. The techniques can be divided into four groups [6], [4]:

- Comparative analysis can be used if the dam under consideration is very similar in size, reservoir area and volume parameters to a dam that failed. The failure of the similar dam has to be well documented. In that case appropriate breach parameters or peak outflows may be determined by comparison.
- Empirical equations are regression formulas based on case study data from failed dams.
- Physically based methods predict the development of a breach and the resulting breach outflow using an erosion model based on principles of hydraulics, sediment transport, and soil mechanics.
- Parametric models use case study information to estimate ultimate breach geometry. The breach growth and breach outflows are then simulated using principles of hydraulics.

We generally recommend to predict the breach outflow parameters using all possible methods and to critically compare the results obtained. The studies performed showed that results of calculations differ significantly (Fig. 3). The uncertainty analysis can be a reasonable approach when concluding the design breach outflow hydrograph for the further flood propagation analysis.

3. The uncertainty of dam breach parametres

The uncertainty of calculated dam breach parameters (breach outflow hydrograph, peak flood discharge and failure duration) is caused by numerous reasons. Sources of the uncertainty were grouped into four categories:

- Variability of initial conditions such as initial reservoir water level, initial manipulation with bottom outlets, etc.
- Variable and uncertain properties of the dam (mechanical properties, variable shape, etc.) and reservoir (reservoir volume and area).
- Human factor in manipulation with dam equipment (bottom outlets, spillway gates, etc.).
- Uncertainty due to the anticipated presumptions about breach shape and method used.

The sensitivity analysis of variable initial conditions and anticipated breach size and shape was done in more detail in [5]. This paper deals with the discussion on uncertainty due to the method used and aims to give some guidance on how to deal with it.

4. Case study – Slusovice dam

4.1 Basic data about the Slusovice dam

A comprehensive analysis of the dam breach discharge was carried out for the Slusovice dam. The first category dam Slusovice was built between 1972 - 1976 and has been fully operational since 1978. The dam site is located on the Drevnice River , km 28.80 upstream from the city of Zlin (85 000 inhabitants). The main purposes of the dam are the accumulation of water for the district water supply, the improvement of the minimum discharge under the dam (Q_{min} = 0.039 m^3/s), effective storage during flood periods and power generation in a small hydropower plant. Basic hydrological data are as follows:

- Catchment area: 42.4 km^2;
- Average annual rainfall: 821 mm;
- Average discharge: 0.5 m^3/s.

The dam sub-base is created by original alluvial loam with a layer thickness of about 4 m. Below this layer there is a 4 to 6 m thick permeable gravel layer. At the lowest position of the sub-base, the impermeable claystones and sandstones are located (Fig. 1).

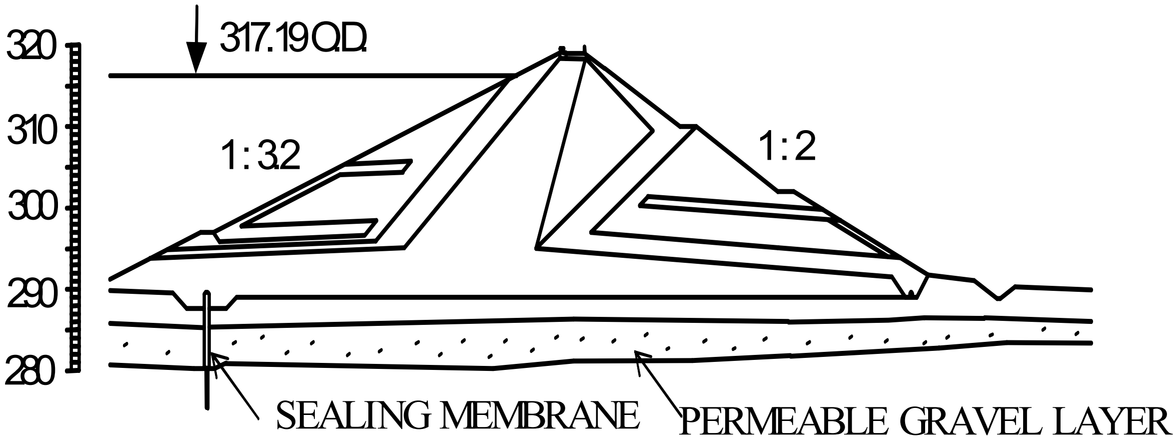


Fig.1: Cross section of the dam

The dam was designed and constructed as an earth-fill zoned non-homogeneous type with sealing element - clayey core (Fig.1). The earth-dam body is composed of clayey core (CL, CH) materials and upstream and downstream gravel shoulders (GW, GP, GC).
At the right-bank section of the valley, the permeable dam sub-base (gravel layer) is sealed by the clay-cement membrane seated to the sub-base claystone and sandstone (Fig.1 and Fig.2). At the membrane contact with the impermeable rock the impermeability is ensured by the grout curtain (see Fig.2). The grout curtain follows along the gallery of bottom outlets. It is inclined to the left bank where the grouting from the grout cap was performed. A free emergency spillway is located at the left bank. Its crest is located at 316.40 m O.D., the length of the weir is 26 m. Sufficient spillway overflow capacity was designed for both Q_{100} = 80 m^3/s and Q_{1000} = 153 m^3/s. When checking the spillway capacity for Q_{10000}= 247 m^3/s, it has also been proved that the dam crest will not be overtopped.
The bottom outlets are composed of the inlet tower, two steel pipes of 1000 mm diameter located at the gallery and the machine hall. Bottom outlets are equipped with three gates (two slide gates and the operation cone valves) and water supply piping branches.

4.2 The location of possible breach formation

According to older studies [2], piping (internal erosion) seems to be the most probable failure of the Slusovice dam. The critical places where the piping channel may occur are at the right bank end of the sealing membrane or at the lowest place in the valley, where the bottom outlets are located. The breach profile at the bottom outlet section (see Fig. 2) represents bigger water depth and therefore bigger peak breach outflow discharges. The analysis focuses on this alternative only.
In the case of piping failure, the initial channel is extending until the portion of the dam above the pipe collapses. From this moment the failure proceeds like a case of overtopping. In the case of the Slusovice dam, the second phase of the failure – the process of overtopping – is the most rapid and crucial for the peak outflow development. Therefore, overflow failure

models were also used for the simulation of the failure which was originally initiated by piping.

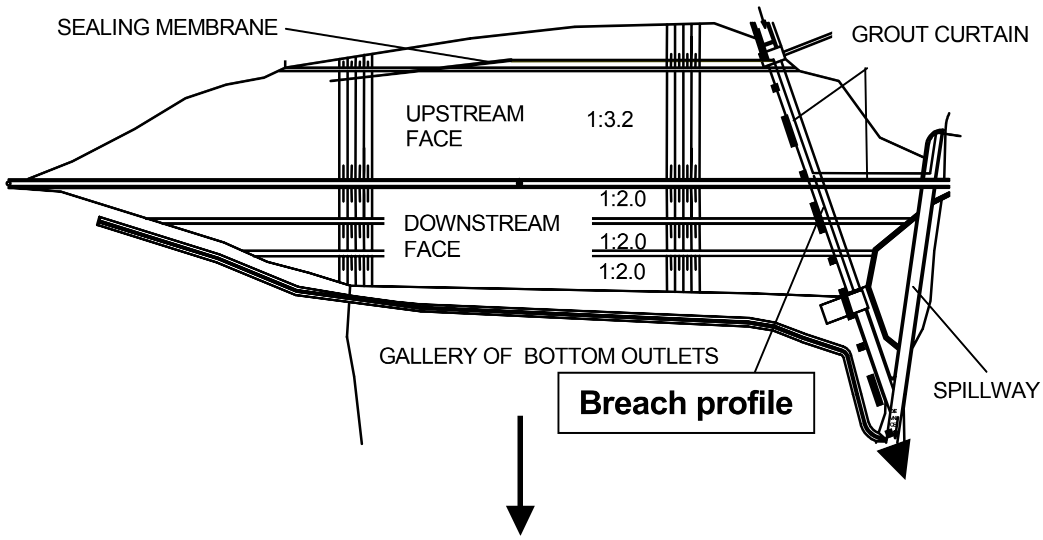


Fig.2: Ground plan of the dam with anticipated piping channel (breach profile)

4.3 Results of peak discharge estimates

The estimate of peak outflow during the Slusovice dam failure was carried by methods mentioned above. The more detailed description of the bibliography available and the methods used shows [4]. The table 1 shows the results of peak discharge assessment.

Tab. 1: Results of peak discharge estimate

Method / author used	**Peak discharge** Q_{max} [m^3/s]
Comparative analysis	
1. Apishapa (USA)	6850
2. Sales Oliviera (Brasil)	7200
Empirical formulae	
3. Categorisation of Water Structures in Czechoslovakia (1973)	8285
4. U.S. Army Corps of Engineers (1980)	15580
5. Statistical study ICOLD (1974)	9550
6. Statistical study elaborated by Czech Dam Safety Division (1987/88)	7820
7. Simek (1988)	8680
8. Vischer, Hager (1998)	3530
9. Dambrk (USA)	5700
10. Costa (1985)	3560
11. Molinaro (1990)	6570
12. Froehlich (1995)	4190
13. Webby (1996)	5180
14. Lemperiere (1998)	6915
15. Chinese dams (1998)	3251
Physically based methods - mathematical models	
16. Breach – numerical model	6803
17. Riha, Danicek (1999)	5989
Prametric model	
18. Holomek, Riha (2000)	3610

It can be seen, that estimates carried out by individual methods vary from 3251 m^3/s to 15580 m^3/s. Further text gives some guidance on how to deal with data obtained in terms of uncertainty and the respective probability of given discharge exceedance.

4.4 Data analysis

The basic assumption in the analysis was that the reliability of individual results (methods used) is the same and resulting peak discharges are the result of "random sampling". The results obtained were analysed using empirical percentiles of the array of results from table 1. Previous studies showed, that in almost all cases, results obtained from U.S. Army Corps of Engineers (1980) recommendations provided considerable higher peak discharges than other methods (see item 4 in Fig. 3). For that reason, this peak discharge was excluded from the analysis in the second step. The percentiles of both sets (with and without USBR results) were arranged into a graph shown in Fig. 4. The graph in Fig. 4 enables easy assessment of the uncertainty in peak breach outflow discharge estimate in terms of the probability of discharge exceedance.

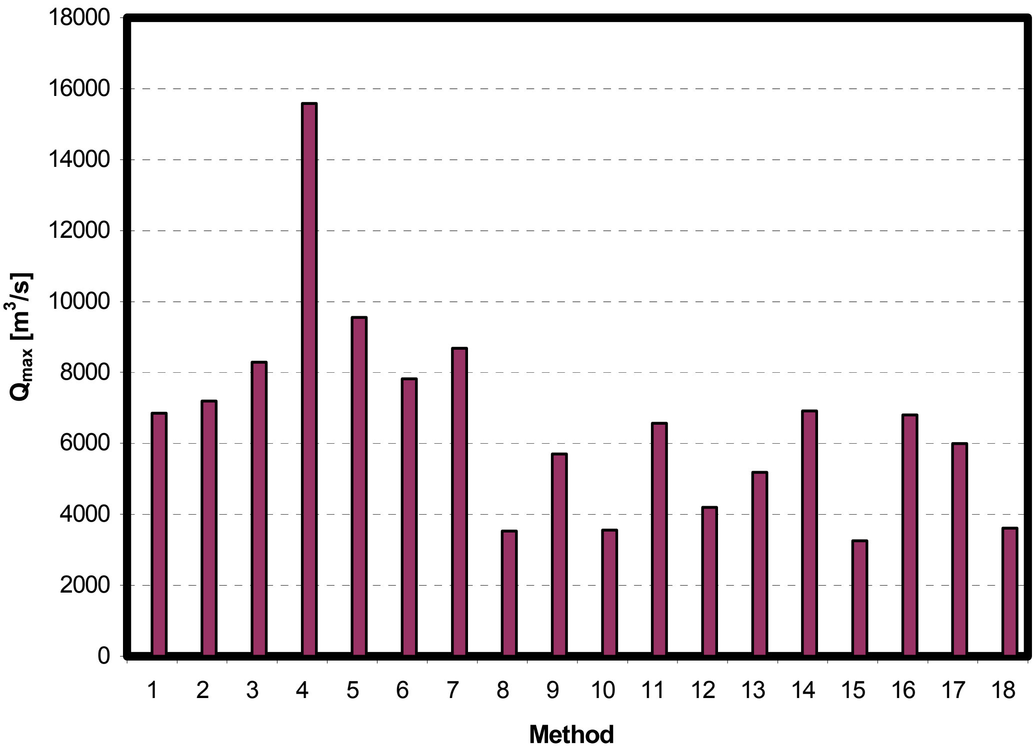


Fig.3: Comparison of estimated peak breach outflow discharges

The conclusion of the analysis is that the maximum peak discharge will probably not be bigger than 10000 m^3/s (when excluding USBR method). A discharge corresponding to 5% exceedance is about 9000 m^3/s. This knowledge is necessary when dealing with risk analysis of the dam and especially with the potential floodplain downstream of the dam.

5. Conclusions

In the paper, the variability of the resulting peak breach discharges is shown (Fig. 3). The aim of the analysis is to express the uncertainty due to the method of estimate. The uncertainty is expressed in terms of probability of peak discharge exceedance. For the analysis, sample percentiles of the array of resulting peak discharges were used. The probability of a given maximum peak discharge can be estimated using the percentile function (Fig. 4). The probability approach is necessary when estimating the risk at the downstream area of the dam.

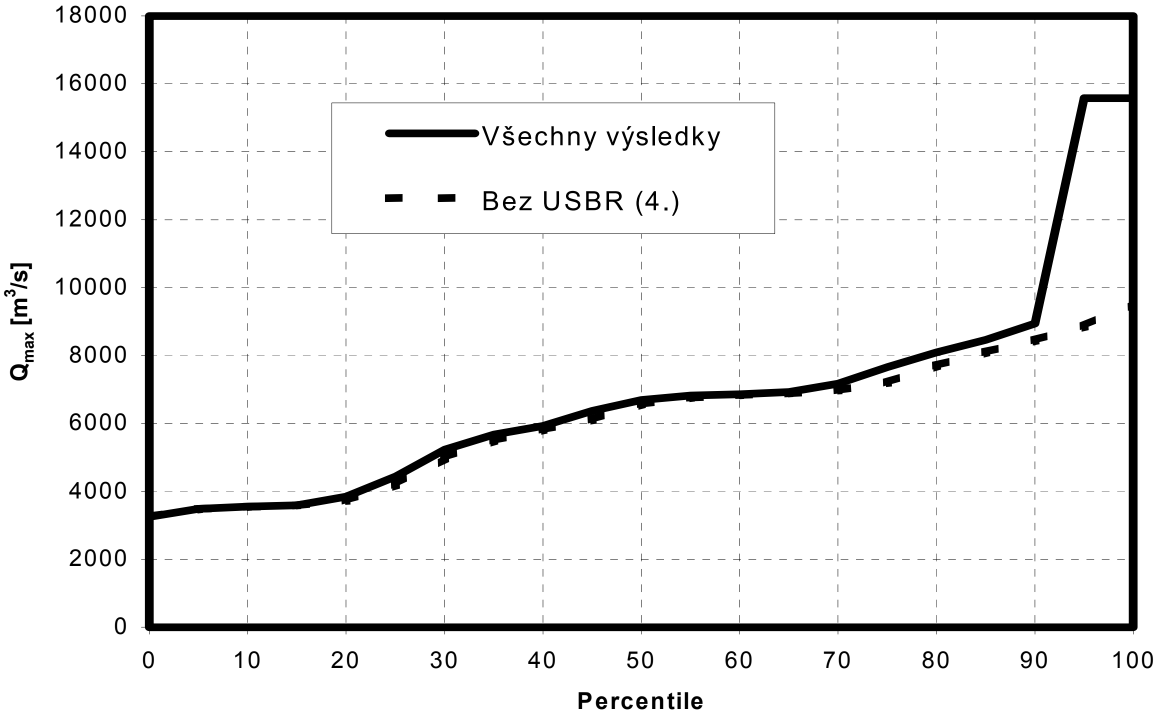


Fig. 4: The percentile function

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Dam Sance – Assessment of the Dam Body Resistance in Case of Its Overtopping

Jaromír Riha[1]

During the catastrophic floods in July 1997 and in August 2002, which affected the Czech Republic, the safety of several large dams was seriously endangered. Recently, a reassessment and rehabilitation of selected dams according to new safety standards has been processed. One of the dams endangered during the 1997 flood was the 60 m high rockfill dam Sance in the North of Moravia. As a result, the Odra river agency initiated the activities for the dam safety re-evaluation and for the design of appropriate measures. The feasibility study of possible improvement scenarios was elaborated by Aquatis consulting engineers from Brno in co-operation with Prague and Brno Technical Universities. Within the framework of the study, the permissible amount of water overtopping the dam crest was analysed. The paper shows some results of the dam overtopping study and presents the final recommendations for the dam safety improvement in case of its overtopping.

1. Introduction

During the catastrophic floods in July 1997 and August 2002, the 60 m high rockfill Sance dam was severely endangered by the flood discharge. On the morning of July, 9 1997 the spillway outflow was estimated at 110 m^3/s, which approximately corresponds to a 5year flood discharge according to older hydrological data (see Fig. 1). Together with the flood danger, the dam body was endangered by the potential wave originated by the landslide of a volume of approximately 1 million m^3. During the rainy spells the landslide crawls slowly at the reservoir area in the bay of the Recice river, which is a right bank tributary of the Sance reservoir. According to the older studies it can originate a wave that reaches a height of 2.2 m at the dam profile.

Within the framework of the feasibility study of possible improvement scenarios, Aquatis consulting engineers, together with Odra River Basin Agency, came up with scenarios of remediation measures against the possible dam overtopping. One of the problems was to assess the consequences of a potential dam overtopping and to quantify limits for safe overflow over the dam crest. The following issues were analysed during the research:

- assessment of present dam resistance against its overtopping from the view of:
  - decrease of downstream face stability due to increase of pore pressures in the case of overtopping;
  - downstream face material stability against its scouring during dam overtopping;
- selection and proposals of appropriate technical arrangements increasing the downstream face stability.

2. Basic information about the dam

The Sance dam is located in the North-East of the Czech Republic on the Ostravice river. It was built in the years 1964 to 1969. The dam was built of rockfill with an inner clayey core. Basic parameters can be seen from Fig. 2. The reservoir serves as the potable water source for Ostrava water supply distribution system. At that time Sance was the highest rockfill dam in the Czech Republic.

[1] Water Structures Institute, Faculty of Civil Engineering, Brno University of Technology, Žižkova 17, 662 37 Brno, Czech Republic, tel. 420 5 4114 7753, fax. 420 5 4114 7752, e-mail: riha.j@fce.vutbr.cz

Fig. 1 Spillway outflow about 110 m^3/s on July 9, 1997 (Odra River Board Agency archive)

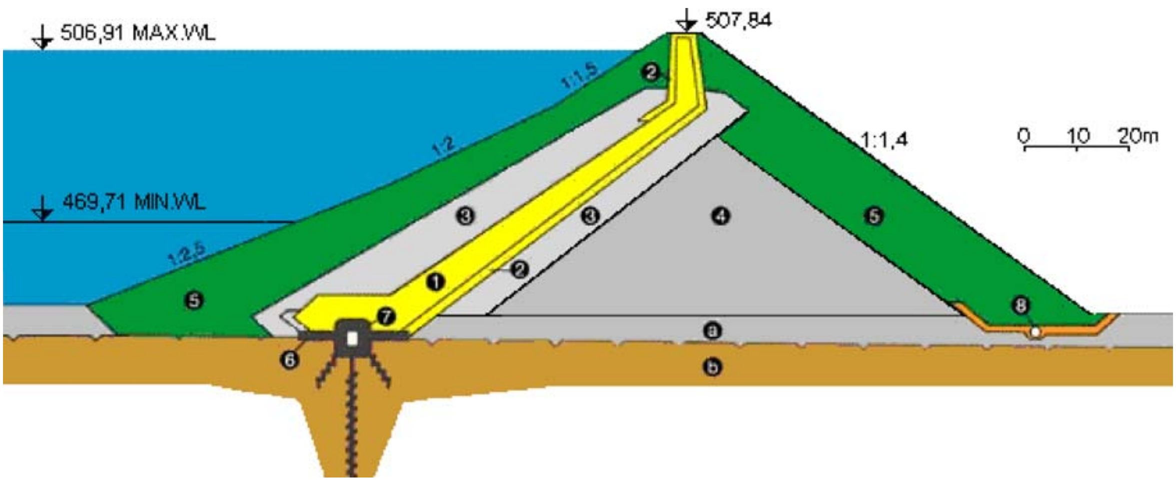

Fig. 2 Cross section of the dam - 1 clayey core, 2 filter, 3 gravel, 4 gravel and unsorted quarry stone, 5 crushed quarry sandstone, 6 concrete, 7 grouting gallery, 8 drainage, a alluvial gravel, b sandstone with clayey slate (according [3])

3. Assessment of the present state

Present dam safety against overtopping is influenced essentially by two factors:

- Extreme floods caused by heavy precipitation with a long duration [4];
- The landslide in the bay of Recice brook.

The most dangerous is the anticipated simultaneous action of both factors, because the landslide movement is initiated by the decrease of both effective stress and internal friction during the rainfall. The assessment of the present state consists of two parts. The part first is dealing with increased seepage though the dam body in the case of extremely high water stages (overtopping of clayey core) and consequent decrease of downstream shoulder stability. The second part deals with surface erosion of the downstream face due to dam overtopping.

3.1 Seepage through the dam body

The assessment deals with two phenomena associated with increased saturation of the downstream shoulder at high water stages in the reservoir. These are overtopping of the clayey core and dam crest overtopping.
In the case of clayey core overtopping, the filtration deformations can occur at the less cohesive and non-cohesive materials located above the dam core. The upper portion of the dam body above the clayey core consists of silty-gravel debris, which has proved to be quite resistant against filtration deformations, namely inner suffosion. The more critical seems to be the contact between the debris and the sandy gravel, which is susceptible to the origin of the "privileged path", which terminates at the downstream face of the dam. Hydraulic gradient J = 0.15 to 0.20 at the zones with different grain size contact can be critical, especially in case of long duration of high water stages. Moreover, inner core overtopping will increase the seepage through the downstream shoulder up to 30 l/s. After 5 to 7 hours it will probably saturate the toe of the downstream slope of the dam due to unsufficient capacity of the drainage system.
At the initial period of the dam crest overtopping (wavebreaker), intensive infiltration of overtopped water to the downstream rockfill shoulder is anticipated. Hydraulic calculations show a maximum specific infiltration rate of 0.1 to 0.2 m^2/s, which corresponds to the weir height 0.15 m to 0.25 m over the wavebreaker crest. Along the entire dam crest it total amount infiltration rate of up to 50 m^3/s is anticipated. The downstream shoulder saturation would be reached in approximately 40 to 50 minutes.
The downstream shoulder has the angle of internal friction $\varphi = 43^O$. To ascertain understand the influence of downstream shoulder saturation, the safety factor SF_S of the dry shoulder was defined as the ratio between the tangent tgφ of angle φ of internal friction and the tangent tgα = 0.714 of the downstream face. For the dry shoulder it holds that SF_S = 1.306. According to the Czech standards, the minimum recommended value is SF_S = 1.5, exceptionally for rockfill faces SF_S = 1.2.
In the case of downstream shoulder saturation, the formula for critical dam slopes was derived for the conditions at the toe of the slope. The formula was expressed graphically in Fig. 3 for variable specific weight of the downstream toe material. For the angle of internal friction $\varphi = 43^O$ and anticipated saturated specific mass γ_{PV} = 9.3 kN/m^3, the critical slope 1 : 2.7 at the shoulder toe can be found at safety factor SF = 1.0. Knowledge obtained shows that from the stability view, the saturation of the existing downstream slope and uncontrolled water seepage is not acceptable due to the decrease of safety factor below the prescribed limit. In the case of dam overtopping, downstream shoulder saturation cannot be avoided.

3.2 Dam crest erosion

In the analysis, the wavebreaker has been assumed as sufficiently rigid and resistant. The most exposed place at the dam crest is the transition zone between the dam crest pavement and downstream slope. Here water flowing over the crest has a relatively high velocity and generates a low pressure zone (see Fig. 4). These conditions accelerate the erosion of the downstream crest edge (Fig. 5).

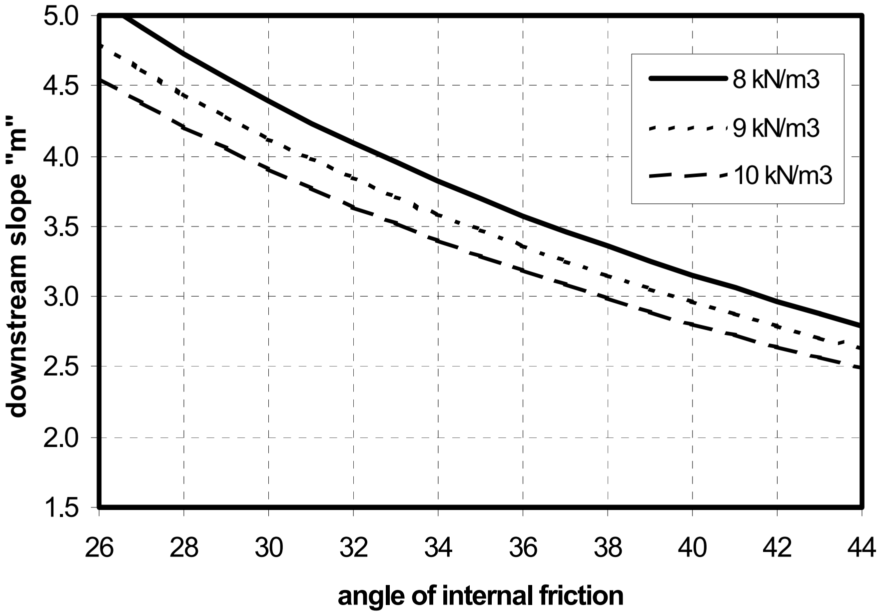

Fig. 3 Marginal slopes when downstream shoulder saturated

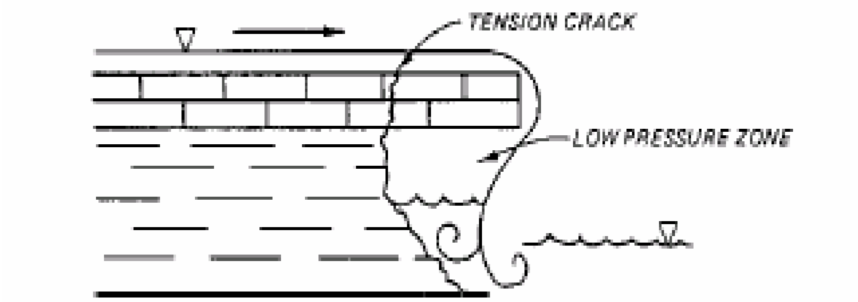

Fig. 4 Conditions at the downstream edge of the dam crest [5]

At the Sance dam, the downstream edge of the dam crest is covered by grass with sparse vegetation (bushes, low trees). When assessed according UK Floods and Reservoir Safety Guidance [7], the permissible flow velocity for grass cover at the crest edge is about 2 m/s for a period of 22 hours. It corresponds to a maximum specific discharge of q = 0,25 m^2/s. Lessons learned show, that in the case of singularities such as bushes or trees, the threshold velocities should be decreased 1,5 to 2 times. The corresponding specific discharge is q = 0,15 to 0,2 m^2/s.
In the case of the anticipated landslide at the Recice valley, overtopping duration was estimated for about 70 s. For such a short time, the permissible critical velocity is 3 m/s. Due to anticipated sparse vegetation uprooting close to the dam crest, local scours of 1,2 m depth will occur at the downstream edge of the dam crest. The asphalt road at the dam crest will probably sustain the short overtopping.

Fig. 5 Damage of asphalt dam crest of Lany dam after its overtopping for 8 hours

3.3 Resistance of downstream slope against surface erosion

Basic data for the assessment were results of the hydraulic calculation of the flow at the dam crest during the overtopping. The resistance of the downstream face and shoulder was analysed by four methods, namely:

- method of critical shear stress (according various authors);
- method of critical flow velocity (according various authors);
- method for the design of boulder spillways;
- method of critical specific discharges [6].

The results obtained by individual methods were compared and critically assessed. Previous studies specified maximum reservoir water level corresponding to:

- overflow height of 0.62 m for 22 hours at Q_{10000} flow discharge;
- overflow height bigger than 2.2 m for 70 seconds in case of landslide in Recice brook bay.

The basic data for the assessment were grain size curves of the downstream face material represented by effective grain size d_e = 0.125 m;

3.3.1 Method of critical shear stress

In this method the shear stress τ at the beginning of the erosion process was compared with the threshold (critical) shear stress calculated according various authors. The critical shear stresses obtained are shown in table 1.

The hydraulic calculation shows that water at specific discharge q = 0.1 to 0.2 m^2/s seeps into the dam body at the downstream face. Therefore the hydraulic parameters and corresponding shear stress were determined for a specific discharge bigger than q = 0.2 m^2/s. The dependence between flow velocity, shear stress and specific discharge is shown in Fig. 6. Shear stress was compared with its critical value mentioned in table 1.

Our previous research proved results of some authors [5], who showed that critical shear stress calculated by classical "open channel formulas" gives, when extrapolated 5 to 20 times, bigger values than those obtained from experiments made for greater slopes and bigger grain size of the rockfill. Based on this knowledge we incline to a permissible shear stress close to 1 kPa, which corresponds to a specific discharge of about 0.35 m^2/s at overflow height 0.40 m over the wavebreaker.

Tab. 1 Critical shear stress according various authors

Author	Critical shear stress in [Pa]
Schoklitsch	402
Shields	121
Kalis	138
Wittler	14
US Waterway	234
Krey	89

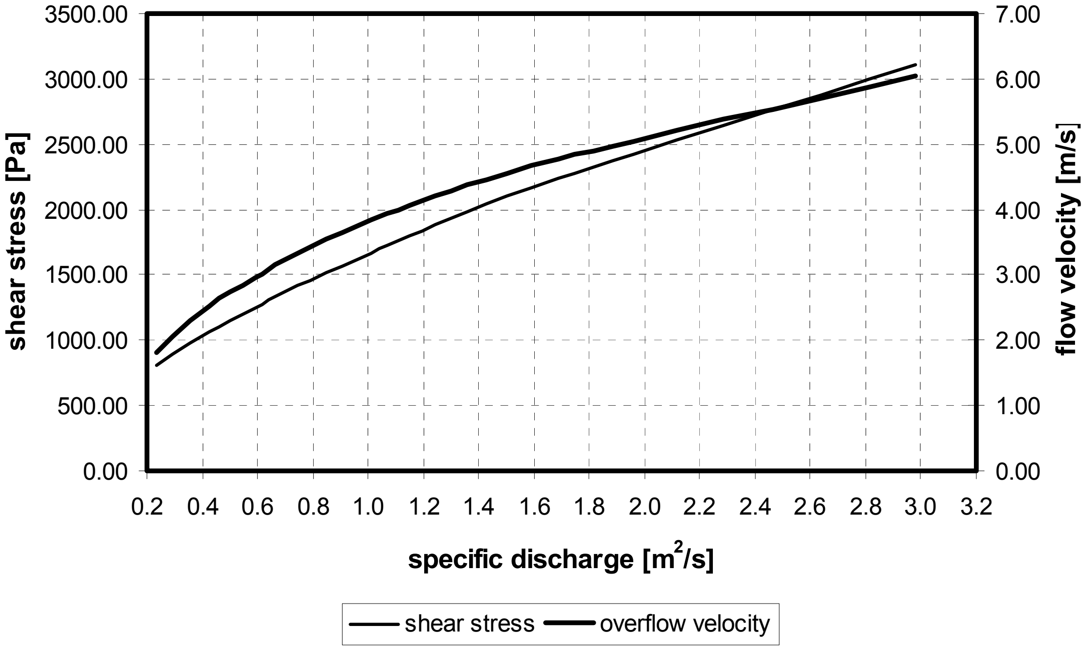

Fig. 6 Shear stress and flow velocity as a function of specific discharge

3.3.2 Method of critical flow velocity

The critical (non-scouring) velocity was calculated according authors working in the field in open channel morphology. The average flow velocity was used for the assessment. Resulting critical shear velocities obtained according various authors are shown in table 2. Finally we accepted a permissible shear velocity up to 2.0 m/s, which corresponds to a specific discharge 0.3 m^2/s.

Tab. 2 Critical shear (non-scouring) velocities according various authors

Author	Critical shear velocity [m/s]
Levi	2.10
Samov	2.17
Neil	3.05
Goncarov	2.20
Meyer-Peter	3.21

3.3.3 Method based on experimental research of boulder spillways

The hydraulic research on boulder chutes was carried out in the sixties and seventies in the hydraulic laboratory of The Technical University in Brno. It was performed for chutes of longitudinal slope of up to 1 : 6 ($J = 0.167$), while the Sance dam has a downstream slope 1 : 1.4 ($J = 0.714$). Therefore the use of the technique needs the results obtained during the research to be extrapolated. The results of our calculations show the critical flow velocity as 1.2 m/s and a permissible specific discharge of about 0.15 m^2/s.

3.3.4 Method of critical specific discharges

For the rockfill dams the extensive research was done by [6]. From the graphs published (see Fig. 7) when extrapolated for the slope $J = 0.714$ and effective grain diameter $d_e = 0.125$ m, the permissible specific discharge for random fill is 0.2 to 0.3 m^2/s.

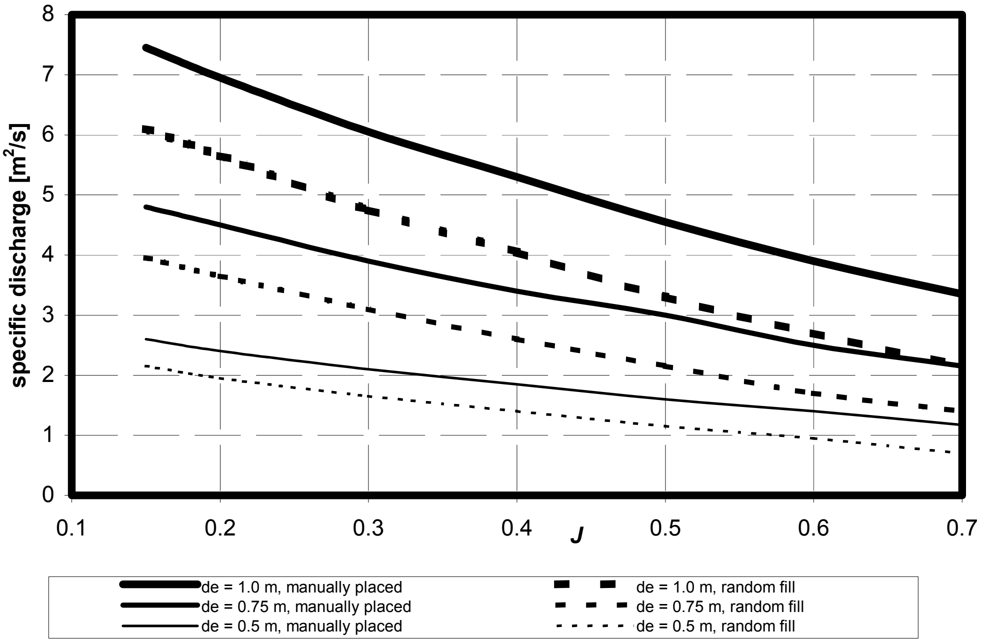

Fig. 7 Critical specific discharges for rockfill face (Knauss 1979)

3.3.5 Comparison of results

Calculations mentioned above were carried out for the most unfavourable grain size represented by effective diameter $d_e = 125$ mm. Comparison of the results obtained gives a permissible specific discharge during dam overtopping in the range of 0.2 to 0.3 m^2/s. It corresponds with an overflow height of 0.3 m and total overtopping discharge of up to 60 m^3/s over the dam crest. Of course, this can be only assumed in the case of ideal horizontal crest of the wavebreaker. A slightly bigger overtopping discharge (up to 80 m^3/s and the overtopping height 0.35 m) could be accepted in the case of single short overtopping due to a landslide in the Recice brook bay. This will produce the movement of individual stones on the downstream face.

4. Conclusions

From the view of global downstream shoulder stability, long term flood overtopping cannot be accepted due to the increase of pore pressures and loss of the stability of the relatively steep downstream face of the dam. It concerns the especially long duration of the overtopping in the case of a hydrological flood. The final recommendations are:

- to ensure perfect function of drainage system;
- to avoid clayey core overtopping by arrangements at the dam crest (underground wall connecting the dam crest with the top of inner core).

Dam overtopping could be permitted only in the case of increased downstream face stability by the additional fill in of the slope 1 : 2.7 at the downstream toe of the dam. Further analysis should be focused on

- flow analysis at the downstream shoulder with detailed assessment of pore pressures;
- more detailed assessment of the downstream face stability;
- the reassessment of slope stability for the reconstructed downstream face.

Our conclusion is that the dam, in its present state, could be overtopped for a short time by a flow corresponding to a specific discharge of up to 0.3 m^2/s. The potential wave from the landslide has its biggest height at the left bank abutment so the overflow along the dam crest would be non-uniform. The seepage through the downstream shoulder and flow erosion can cause some local deformation at the dam toe and at the dam crest edge. Considering the very low probability of such a scenario, we recommended to permit a small and time limited overtopping as a supplementary arrangement in context with other improvements to the dam safety. Of course, to permit dam overtopping is an unusual act in dam engineering and in a context to engineering ethics.

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Tracer Investigations of Dynamic Properties of Fluid-Flow Reactors

Jerzy M. Sawicki[1], Magdalena Kinga Skuza[1], Sławomira Bering[2]

[1] Gdañsk University of Technology, Department of Hydraulics and Hydrology, ul. G. Narutowicza 11/12, 80-952 Gdańsk, Poland, jsaw@pg.gda.pl

[2] Technical University of Szczecin, Department of Sanitary Engineering, Aleja Piastów 50, 70-311 Szczecin, Poland

Abstract

Designing new reactors or analysing already existing objects, one should have at his disposal credible models of the processes, which run in the considered devices. Especially valuable method, which gives possibility to solve practical problems, is a tracer analysis. The paper contains four examples of the laboratory tracer investigations of four different fluid-flow systems: rectangular reactor, abdominal cavity, oil separator and trickling filter. In each case the individual features of the object were discussed, but also some general conclusions are presented.

1 Introduction

Two kinds of problems can be met in the technical practice: new objects design and investigation or modernization of existing objects.

Solution of each problem is conditioned by the model applied by the investigator. Such a model should describe the considered system with an acceptable accuracy and should be dissolvable. Existing possibilities could be classified as follows:

- *general models* (based on differential equations of mass, momentum and energy conservation);

- *simplified models* (based on algebraic relations, which combine the averaged parameters of the object).

As far as fluid-flow reactors are considered, the following *equation of dissolved mass conservation* [5]

$$\frac{\partial c}{\partial t} + div(\boldsymbol{u}c) = div((D_e grad \quad c) + Z \tag{1}$$

(c - concentration of dissolved matter, **u** - velocity vector, D_E - effective coefficient of relative transport, Z - effective source function) can be recognized for the main mathematical expression of the family of general models.

In the category of simplified tools in turn, there are two basic possibilities: the *plug flow model* and the *ideal mixing model* [1].

The credibility of each model (both general and simplified) is determined by the results of its practical verification. The empirical data for such verification must be collected during the measurements of the parameters of the considered process. Especially convenient are the *tracer methods*, when the initial $c_P(t)$ and terminal $c_T(t)$ concentrations of the dissolved indicator, introduced into the system, are measured. As the most convenient form of the tracer injection one has to recognize the *impulse* (when a small amount of a highly concentrated solution of the indicator is instantaneously injected into the initial cross-section of the reactor). In this case one easily compare the operation of different objects and it is possible to discuss the applicability of different theoretical models (Fig.1)

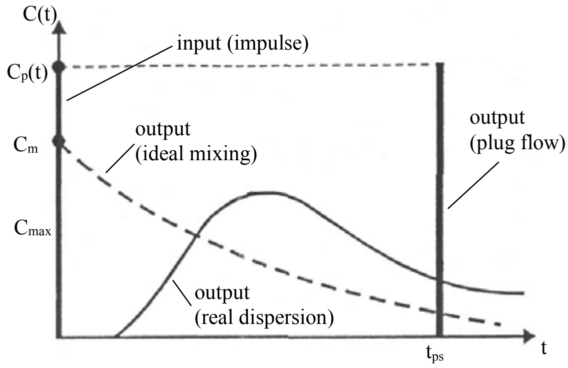

Fig. 1: Main theoretical models of fluid-flow reactors

Besides, the tracer methods enables [4]: construction of the theoretical models of the reactors; identification of its parameters; immediate appreciation of existing reactors and also theoretical reflection.

This paper contains the presentation of four examples of laboratory investigations of the fluid-flow reactors, with the use of the tracer method. A concentrated solution of the sodium chlorine *NaCl* was applied as an indicator. Its concentration was measured by a conductometer.

2 Rectangular reservoir

The shape and dimensions of the investigated object are shown in Fig. 2.

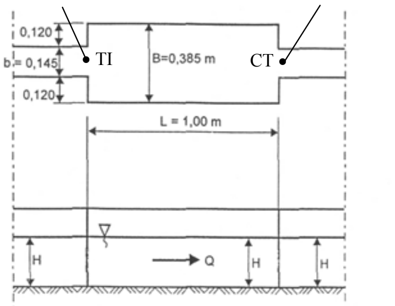

Fig. 2: Scheme of the rectangular reactor

The tracer (200 g/dm^3 solution of *NaCl*) was introduced (three times in each test) in the initial cross-section of the reservoir, into the uniformly flowing water. The volume of this impulse was equal to 200 ml. The terminal concentration of *NaCl* was measured in the outflowing stream (point CT in Fig. 2). Indications of the conductometer were recorded by means of a video-camera. An example of the obtained results is shown in Fig. 3 [3].

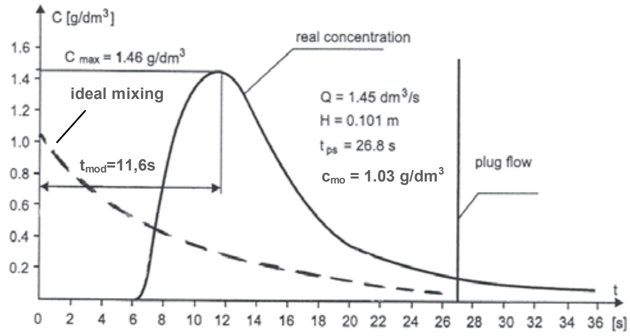

Fig. 3: Dynamic characteristics of the rectangular reactor

The analysis of these results leads to some interesting and useful conclusions. Referring these results to simplified models one can state that the transformation of the tracer concentration in the reactor does not suit the theoretical plug flow model (which all the same is commonly used by the sanitary engineers) nor the ideal mixing model (what was to be expected).

The results of this experiment can also serve as an empirical basis for the verification or identification of each differential model. Here we will confine our considerations to the 1D dispersive model, which is quite often (although sometimes without any deeper reflection) used in similar cases by engineers (e.g. [2]). The real curve $c_K(t)$ (see Fig. 3) in this 1D-case should correspond to the classical analytical solution [5]

$$c_K(t) = c(x = L, t) = \frac{M}{2BH\sqrt{K_L \pi t}} \exp\left[-\frac{(L - vt)^2}{4K_L t}\right] \tag{2}$$

(M - total mass of tracer, in this case M = 40 g; K_L - coefficient of longitudinal dispersion; v - mean velocity; t - time).

According to Eq. 2, the maximal concentration of the tracer appears in the terminal cross section after a lapse of a modal time

$$t_{mod,calc} = L / v = 26.8 \text{ s} , \tag{3}$$

which is about two times longer than the observed modal time

$$t_{mod,obs} = 11.6 \text{ s} . \tag{4}$$

The value $c_{max,obs}$ = 1.46 g/dm^3 means, that the coefficient of dispersion, obtained from Eq. 2 for the calculated time of flow, equals $K_{L,ident,calc}$ = 0.0015 m^2/s, whereas for the measured value of time we have $K_{L,ident,obs}$ = 0.0034 m^2/s.

Using in turn the well-known Elder formula [5] (for the Manning coefficient n = 0.016) we obtain $K_{L,th,calc}$ = 0.0018 m^2/s and $K_{L,th,obs}$ = 0.0041 m^2/s respectively. Comparing these values we can state, that this theoretical model of the relative mixing (longitudinal dispersion) gives quite good conformity of the measured and calculated concentration, but from the other hand - mean velocity v very poorly describes the real advection. This conclusion means that the 1D model of this kind of problem should not be applied for the fluid-flow reactors.

3 Intraperitoneal perfusion

This term denotes the forced flow of a fluid (e.g. dissolved medicament) through the cavity of a human body. The aim of the case described above was the laboratory simulation of the perfusion through the human abdominal cavity. This process is an element of a relatively new, but poorly investigated, method of a medical treatment in some tumours. The sketch of the laboratory installation is shown in Fig. 4.

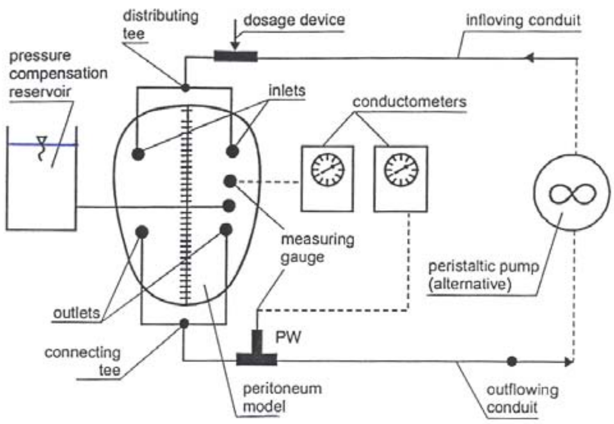

Fig. 4: Model of the perfusional flow

The main goal of the experiment was to describe the influence of the inlets and the outlets positions on the mixing intensity. In the ideal case the system should work as a full mixing reactor.

As it shown in Fig. 5, the level of miscibility in the investigated installation is rather far being perfect. The analysis of all possible positions of drains led to the conclusion that the most convenient solution would be to change the direction of flow during the course of the considered process. In order to apply this possibility, a special switch should be installed in the system.

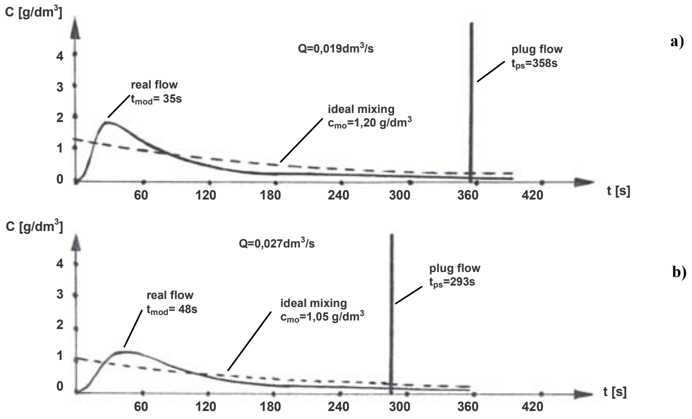

Fig. 5: Dynamic characteristics of the intraperitoneal perfusion

4 Model of the oil separator

Oil separators are the indispensable elements of the storm sewage systems. There are different individual solutions of the inner equipment of such a device. In this particular case a laboratory model of so-called *coalescence-assisted oil separator* was investigated (Fig. 6a).

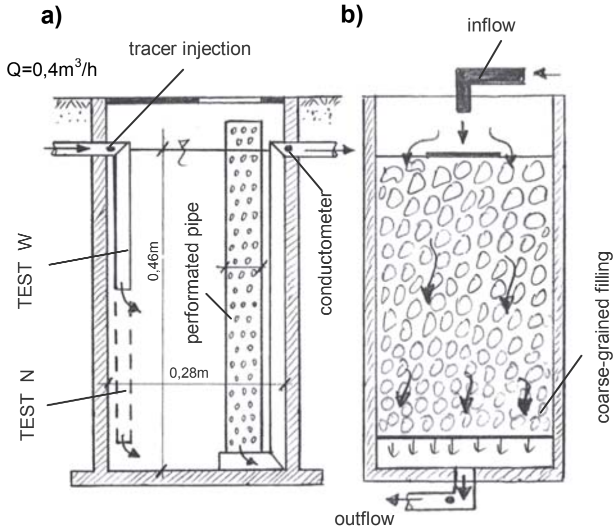

Fig. 6: Laboratory model of oil separator (a) and trickling filter (b)

Two variants of the inlet positions (test W - in the middle of the separator height; test N - 5.0 cm above the reservoir bottom) and three variants of the outlet configuration (M - perforated pipe, 2.5 mm hole diameter; D - perforated pipe, 5.0 mm hole diameter; B - simple outlet in the reservoir bottom) were investigated (for different discharges of water).

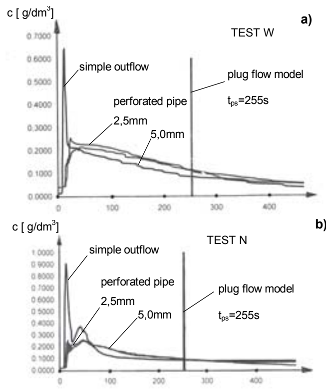

Fig. 7: Dynamic characteristics of the oil separator

An example of the obtained results (test W, for Q = 0.11 dm^3/s) are shown in Fig. 7. As it is seen, the plug flow model (so commonly used for dimensioning of water treatment and waste disposal reactors) is completely useless in this case. The *mean detention time*

$$t_{ps} = V / Q , \tag{5}$$

which is the main parameter of this model, equals here t_{ps} = 28/0.11 = 255 s. This value is at least five times longer than the modal detention time t_{mod}.

These results show also, how important is the shape of the outlet. Perforated pipes radically improve the conditions of penetration of the reactor by the dissolved matter.

5 Model of the trickling filter

The hydraulic characteristics of the reactors packed with the coarse-grained filling are especially poorly recognized, even in comparison with other reactors. For this reason the investigations of the model of such a device (i.e. trickling filter - Fig. 6b) had rather recognitive character.

Making use of the tracer method, the terminal distribution of the indicator was measured in the outflowing conduit (situated in the reservoir bottom).

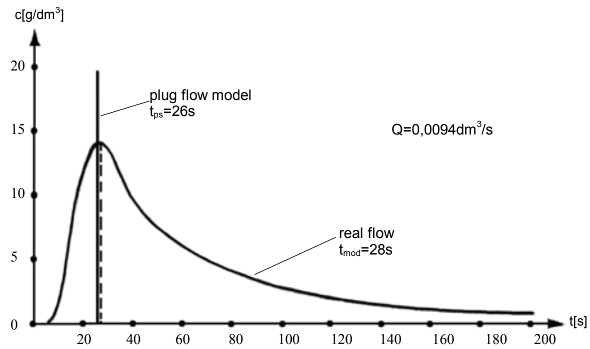

Fig. 8: Dynamic characteristics of the trickling filter

In order to refer the results (Fig. 8) to other parameters of the considered device and to compare them with practical instructions and guidelines, one should be able to determine the total volume of water V_f, which stays in the reactor in each moment of time. Having regard to the fact that the theory of this problem practically does not exist, the value V_f was determined experimentally (Fig. 9).

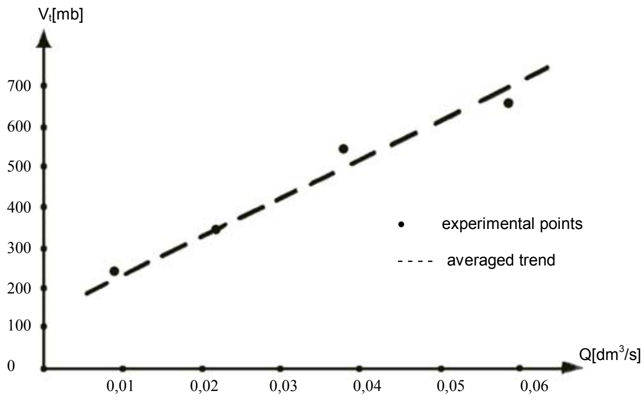

Fig. 9: Volume retention of the liquid in the trickling filter

According to the plug flow model (Eq. 5), for discharge Q = 0.0094 dm^3/s we have in this case t_{ps} = 0.245/0.0094 = 26 s, what is surprisingly (see Figs. 3, 5 and 7) close to the measured modal time for the function $c_K(t)$. It can be a very essential information to somebody, who intends to develop a theoretical model of a trickling filter or a similar device. This problem will be the subject of the further investigations.

5 Conclusions

The results of the laboratory investigations of four reactors, described above, in the first order show the usability of the tracer method, when the dynamic properties of the fluid-flow reactor must be determined.

These results can serve as an empirical basis for the verification of different mathematical models of the considered objects, but they also make possible to adjust a proper simplified model for such device. Especially interesting is the statement that the plug flow model has a very differentiated applicability. Namely, the mean detention time for the trickling filter is very close to the real detention time. For a simple rectangular reactor the value t_{ps} is about two times longer than the modal time t_{mod}. And for reactors with a complex velocity field (e.g. abdominal cavity, oil separator) the value t_{ps} is several times greater than the modal time, what practically disqualifies the plug flow model for such objects.

A very important result is the indication of the influence of the position of inlets and outlets from the reactor. Each correction of this factor causes the change of the real detention time (whereas the computational mean detention time keeps the same value - see Eq. 5).

The third advantage of the tracer investigations is a startpoint for the analysis and possible correction of the existing objects, what must be done individually.

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Impact of Climate Change on Mean Monthly Flows of the Orava and Ipel Rivers

Ján Szolgay, Kamila Hlavčová, Róbert Danihlík

Department of Land and Water Resources Management, Slovak University of Technology, Radlinského 11, 813 68 Bratislava, Slovak Republic, szolgay@svf.stuba.sk, hlavcova@svf.stuba.sk, danihlik@svf.stuba.sk

Abstract
The potential impact of a changed climate on monthly river runoff in two basins in Slovakia was evaluated using a monthly water balance model. The selected basins represent typical patterns of seasonal runoff distribution along a transect from the north to the south across Slovakia. The Watbal monthly water balance model, which was used in previous studies, calibrated using genetic algorithms with data from the baseline period of 1951–1980, which is one of the standard periods usually considered representative of the runoff distribution for stationary climate conditions in the framework of the Slovak National Climate Program. Regionally downscaled outputs of two coupled ocean and atmosphere global circulation models (GCM), the Canadian CCCM97 model and US GISS98 model were selected to test the sensitivity of the selected basins to climate change. Based on these scenarios the possible changes in the distribution of mean monthly runoff for the time horizons 2010, 2030 and 2075 were evaluated.

Key words
Monthly water balance model, impact of climate change, seasonal distribution of runoff

1 Introduction

It is widely recognized that greenhouse gases of an anthropogenic origin that are released into the atmosphere have the potential to change the present climate both on global and local levels. It is also expected that changes in the temperature and distribution of precipitation can alter generating conditions of runoff and, subsequently, river flow regimes. The importance of water to both society and the environment underscores the necessity to study how the impact of climate change may change spatial and temporal distribution of runoff in different regions. In this study the potential impact of a changed climate on monthly river runoff in two regions in Slovakia was evaluated.
Several climate change impact studies have been conducted in recent years on the territory of Slovakia. To assess changes in river flow, several watersheds and/or regions of a characteristic location and size were selected. Observed runoff series from the periods 1931 to 1960 and/or 1931 to 1980 were usually considered as baseline scenarios in the impact studies. Several water balance models were used with annual, monthly and daily time steps to simulate the impact of the climate change scenarios. No comprehensive review will be attempted here of the vast number of papers published on the subject. Only references that are relevant to the methodology and the region will be cited. This study builds on methodological approaches developed in Szolgay, et al. (1997), Hlavčová and Čunderlík (1998), Čunderlík,

Hlavčová and Szolgay (1998), and Hlavčová, Kalaš and Szolgay (2002). For a comprehensive contemporary review of climate change impact studies conducted in Slovakia see Szolgay, Hlavčová and Kalaš (2002), where basic adaptation strategies were also discussed. Details on climate change impact on hydrology in Slovakia can be found e.g. in the following case studies: Fendeková (1995), Lapin, et al. (1995), Lapin, et al. (1997), Lapin and Melo (1999), Marečková, et al. (1997), Pekárová, et al. (1996), Petrovič (1998), Fendeková (1999), Faško et al. (2000), Halmová (2000), Kostka and Holko (2000), Majerčáková (2000), Pekárová (2000), Szolgay and Hlavčová (2000), Fendeková and Némethy (2001), Faško and Štastný (2001), Kostka and Holko (2001), Pekárová et al. (2001), Pekárová and Miklánek (2001). For reviews on a global scale, we refer to the latest reports of the Intergovernmental Panel Council on Climate Change, especially to McCarthy, et al. (2001).

2 Study sites and data

For the evaluation of seasonal runoff changes two river basins were selected: the Biela Orava River in Lokca, and the Ipeľ River in Holiša. The catchment areas and mean altitudes of the basins are summarised in Table 1. They are located along a transect from north to south across Slovakia. The Biela Orava River is representative of the hydrological regime of mountainous regions of northern Slovakia and the west flysh belt. In that area several reservoirs are in use for hydropower generation, flood protection and drinking water supply. The basin of the Ipeľ River represents the hydrological regime of the southern slopes areas of the Slovenské Rudohorie mountains. Several multipurpose and drinking water reservoirs are operating in the area, agricultural water use is also of importance in water resources management.
For water–balance modelling in these basins, climate characteristics from the standard period of 1951-1980 and a suitable number of climate stations were chosen. For calibrating the runoff model in a monthly time step, the following data were used: time series of mean monthly runoff, precipitation, air temperature, duration of sunshine and relative air humidity.

Table 1: The basic characteristics of the basins selected for the study

Basin	**Station**	**Area [km^2]**	**Mean altitude [m a.s.l.]**
Biela Orava	Lokca	359,96	865
Ipeľ	Holiša	685,27	375

3 The climate change scenarios

Climate change scenarios should represent the future climate of selected localities to the largest possible extent but on the other side they should also show alternatives for future development of the climate. In this study therefore the regionally downscaled outputs of the CCCM97 and GISS98 coupled ocean and atmosphere general circulation models have been applied to test the sensitivity of the selected basins to climate change. Downscaling was performed in all the climatic stations selected for the study by the methodology recommended by the Slovak National Climate Program. Detailed description of the scenarios is given by Lapin, et al. (2000).

4 Scenarios of changes of seasonal distribution of monthly runoff

In Slovakia, a simple, conceptual spatially-lumped hydrological rainfall-runoff model, WatBal, which is based on the works of Yates (1994), was extensively used for modeling river runoff in a monthly time step in climate change impact studies (see, e.g., Hlavčová, Čunderlík (1998), Čunderlík, Hlavčová and Szolgay (1998)).
WatBal is a conceptual lumped rainfall-runoff model, which simplifies a river basin into a single nonlinear reservoir. The model simulates water accumulation in the catchment, snowmelt, evapotranspiration, runoff from impermeable areas in the basin, surface and subsurface runoff and baseflow. The inputs required for water balance modeling when using a monthly time step are: the mean monthly precipitation for the basin, the mean monthly river discharges in the closing profile of the basin and the mean monthly potential evapotranspiration (PET). If the PET data is not available, the model uses either the Thornthwaite or the Priestly-Taylor method. The Priestly-Taylor method was used in this study. It requires additional data: the mean long-term monthly hours of sunshine, the mean long-term monthly values of relative air humidity, and the mean monthly air temperature values.
In the present study a genetic algorithms (GA) was applied to calibrate the model to at site data. GA are stochastic search methods that simulate the process of the natural selection and the mechanism of population genetics. In recent years they have been widely applied in a number of fields. In this study the so-called Generational GA (GGA) was used. The GGA is a type of genetic algorithm in which the entire population is replaced in each iteration. This method of progression for a genetic algorithm has proven to work well for a wide variety of problems. It tends to be a little slower than some other of its modifications, but it also tends avoid local minima. For the optimalisation the Nash-Sutcliffe criterion, which is widely used in modeling studies, was used.
The hydrological scenarios of changes in seasonal runoff distribution were constructed in a standard way as follows:

a) calibration of the Watbal model in the selected basins,
b) the generation of the reference (baseline) model data using input data from the standard period of 1951-1980,
c) modification of the model input data from the baseline period (precipitation and air temperature) according to the climate change scenarios for the time horizons of 2010, 2030 and 2075,
d) simulation of the monthly runoff series using the WatBal model based on the changed input data and parameters of the model from the calibration,
e) comparison of the differences between the seasonal runoff distribution for the individual scenarios and the time horizons considered.

Results of the typical behavior of the long-term mean monthly runoff under climate change are given in Table 2. In the seasonal distribution of river flows in the northern part of Slovakia (as represented by the Orava River), it can be concluded that for all the time horizons, a similar redistribution of river runoff within the year can be observed. From October till April an increase in runoff can be expected; on the other hand, during the months of May to September a decrease in discharge may take place.
In central and southern Slovakia (as represented by the Ipel River), similar changes in the river runoff redistribution within a year can be expected; however, the amplitude of the changes will be greater. From November to February, an increase in river runoff can be assumed. On the other hand, the period from March to October represents a continual period of low flows.

Table 2: Changes in mean monthly discharges in the selected catchments

Basin	Scenario	Horizon	I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII
								[m^3/s]						
Biela Orava		2010	0,50	0,97	1,03	-1,49	-1,49	-0,60	-0,93	0,04	-0,22	0,43	0,92	0,62
Lokca	**CCCM**	2030	0,95	1,91	1,74	-2,26	-1,38	-0,48	-1,15	-0,15	-0,35	0,46	1,13	0,96
		2075	2,96	4,89	2,40	-4,77	-2,36	-1,52	-2,13	-0,99	-1,11	-0,18	1,79	2,44
		2010	0,33	0,33	0,42	-0,24	-0,57	-0,07	-0,02	0,46	0,90	0,78	0,64	0,61
	GISS	2030	0,93	1,07	0,18	-1,61	-0,75	0,20	0,20	0,41	0,83	1,03	1,09	1,09
		2075	2,97	3,38	1,57	-3,70	-1,28	-0,10	-0,31	-0,24	0,03	0,59	1,69	2,88
		2010	1,45	2,00	0,68	-2,95	-1,77	-0,67	-1,01	-0,64	-0,59	-0,26	0,53	1,18
Ipel’		2010	0,27	0,37	0,25	-1,57	-0,99	-1,64	-1,24	-1,21	-0,29	-0,05	0,22	0,27
Holiša	**CCCM**	2030	0,62	0,97	0,43	-1,70	-0,71	-1,45	-1,25	-1,30	-0,39	-0,07	0,38	0,62
		2075	2,08	2,76	-0,34	-2,66	-1,55	-2,64	-2,24	-1,85	-0,93	-0,72	0,30	2,08
		2010	0,14	-0,06	-0,29	-0,74	-0,22	-0,15	-0,13	0,20	0,39	0,31	0,18	0,14
	GISS	2030	0,76	0,36	-0,82	-1,35	-0,44	-0,05	0,08	0,35	0,47	0,53	0,45	0,76
		2075	2,32	1,93	-0,52	-1,78	-0,48	-0,43	-0,39	-0,42	-0,32	-0,09	0,35	2,32
		2010	0,27	0,37	0,25	-1,57	-0,99	-1,64	-1,24	-1,21	-0,29	-0,05	0,22	0,27

5 Conclusions

According to the results of previous impact studies, a decrease in the mean annual runoff seems to be far more probable than no change in the present conditions or an increase in runoff. The effects of an increase in air temperature are expected to be decisive for such behavior, even if the mean annual precipitation moderately increases. The changes could exhibit a north-south gradient with the northern territories and higher altitudes being less affected or without any significant change. The aridity of the lowlands in the south could reach a severe degree (especially according to the analogue scenarios). As for the seasonal distribution of flows, it was concluded that discharges in general should increase in the winter low-flow period and that flows could (substantially) decrease in the spring. The flow regime in the summer and autumn could show stable behavior with a moderate decrease in runoff.
Results form this study are in good agreement with these results. According to the anticipated changes in the seasonal distribution of the mean monthly runoff, almost the whole territory of Slovakia could become vulnerable to drought in the summer and autumn. In the months with an increased water demand for irrigation, domestic and industrial use and tourism, monthly flows could exhibit a decrease under climate change conditions. The intensity of the changes could increase towards the time horizon of 2075. The continuous general decrease in the utilizable potential of the surface and subsurface water resources is likely to occur. This will have to be reflected in the planning and management of water resources in the near future.

Acknowledgement

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Discharge and Water Level Regime in Bio-Corridor of the Žilina Water Structure

Andrej Šoltész, Dana Baroková

Slovak University of Technology, Department of Hydraulic Engineering, Radlinského 11, 813 68 Bratislava, Slovakia, soltesza@svf.stuba.sk, barokova@svf.stuba.sk

Abstract
The Žilina water structure is the first structure in Slovakia whose influence on the environment has been assessed comprehensively by means of EIA method (Environmental Impact Assessment). The design of the amendment by the NC SR 127/1994 on the assessment of the environmental issues has been confirmed by the water structure (WS) Žilina. The proposals to abate the influence of the structure on the environment have been included into object composition of the structure (1994 – 1998) and the co-operation with the environment experts was also maintained at the realisation and operation of the structure.
One of the measures taken when building the water structure was setting up a bio-corridor, ensuring the migration of fish through the water structure. With respect to the fact that the bio-corridor constituted the substantial requirement for solving the water level regime of the ground water in the village of Mojš, it became crucial to complete the missing data about the discharge and water level regime in the bio-corridor by means of additional measurements carried out by the collective in situ.

1 Introduction

Our primary reason for dealing with the discharge and water level regime in WS Žilina bio-corridor was damping the house cellars in the village of Mojš, located in the surroundings of the Žilina WS. The target area, located in the alluvium on the right bank side of the Váh river, is marked in the east by its affluent - Varínka and in the north by the foot of the Kysuce mountain line. This area is marked by another peculiarity: there is an alternate corridor spreading alongside the dam on the right side of the water structure (filled from the reservoir at the confluence of Váh and Varínka. The alternate corridor due to its level and discharge regime actively affects the ground water level regime in the given area (Kadlec - Kvál - Bukvová, 2001). Due to the construction of the Žilina WS, a complex geological and hydrogeological research was carried out in this area. The research was done in at various times and by various organisations. We focused on the geological boreholes drilled, assessed and monitored on the right side of the Váh river with detailed focus on the Mojš village.

2 Problem description

The necessity to build an alternate bio-corridor has emerged due to cutting off the natural bio-corridor – the Váh river – by assembling the hydraulic structure. The requirements of the alternate bio-corridor were processed by the Zoological Institute of Comenius University in the research project „Study of the function of the alternate bio-corridor for determination of the positive development of biotop“ (Hensel et al., 1993). This study was a basis for an elaboration of the project of the alternate bio-corridor (HYCO Bratislava, 1996), with consequent outlay of the inlet object into the bio-corridor (Hodák et al., 1996).
In April 2000 the water status of the bio-corridor started to be monitored, although in a rather superficial way. The water level regime of the bio-corridor was monitored in B1 profile under

the confluence with the Gbely stream, in B2 profile at the footbridge in the centre of the village of Mojš and in B3 profile at the end of Mojš. The only additional hydrological data, enabling us to determine the water level regime was measuring the discharge into the bio-corridor beyond the inlet object (see fig. 1), and the consumption curve of the alternate bio-corridor of the Žilina WS in the profile of limnigraph below the inlet object, located in Varín. This data, however, say nothing about the current water level status in the bio-corridor. Fig. 1 shows the measurement performed in the period since 1.10 2001 until 31.7. 2002, fig. 3 shows the discharge measured on 12.6. and 15.7. 2002. The reason why we are stating this is the fact that in the time period mentioned we carried out the measurement of the discharge as well as the water level in the bio-corridor in eight profiles as a boundary condition for model calculation of the ground water. At the same time, we measured the discharges in two tributaries into the bio-corridor – an unnamed water stream (PF4) and Gbely stream (PF8). A sample measurement of the discharge in profile B2 in Mojš below the footbridge is shown in the Fig. 4 with simultaneous assessment of the discharge in profile and the map of isohyets in the given profile. Tab. 1 shows the calculation of discharge depending on the point velocity in perpendiculars. Fig. 2 indicates the measured transverse profiles of the bio-corridor. The discharge in the bio-corridor oscillated between 0,95 and 1,3 m. s. The differences in the water level were measured geodetically and connected to the monitoring system of the aiming points placed on the dam on the right side of the Žilina WS. Because we selected the profiles in order to monitor the longest possible stage of the bio-corridor, we obtained certain information about the level and discharge regime in the bio-corridor in the section from the inlet object up to the village of Mojš.

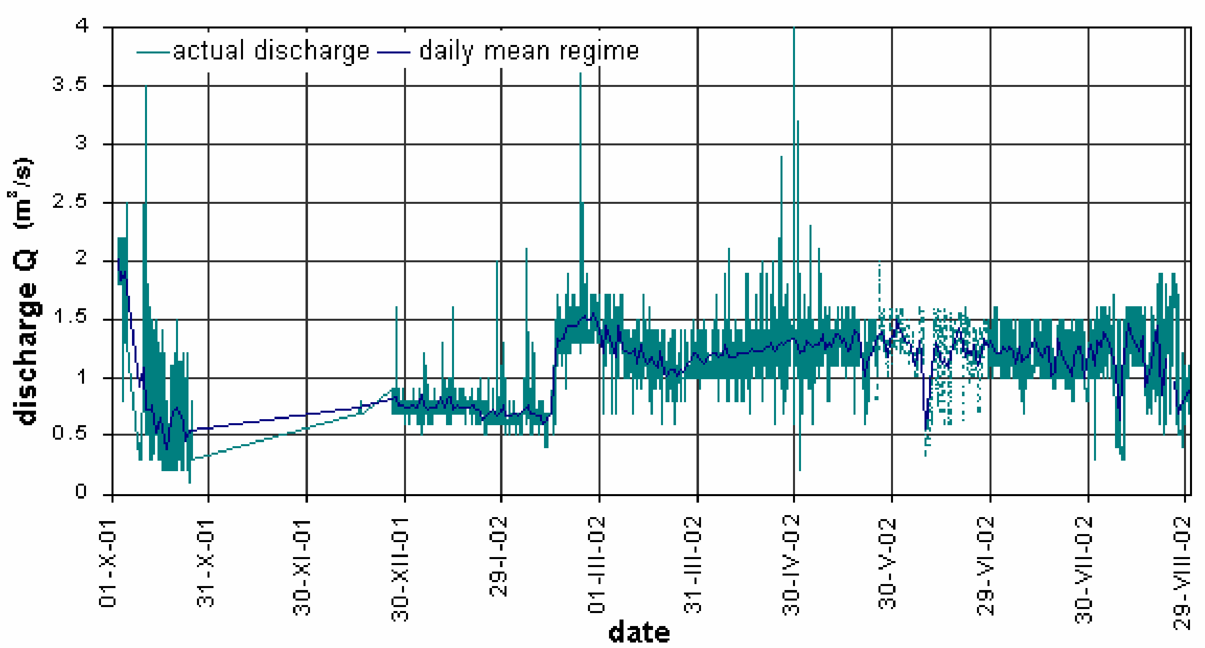


Fig.1 Measured discharge course in the bio-corridor below the inlet object (measured by VV)

Tab. 1 Calculated discharges and water levels in bio-corridor.

chainage of the		Profile	Date, time	depth H	water table	width B	discharge Q
dam	bio-corridor						
(rkm)				(m)	(m a. s. l.)	(m)	(m^3s^{-1})
			12.6.2002				
6.97	10.07	PF 1, (KVB24)	12:05	0.68	349.81	6.30	1.054
6.58	9.68	PF 2, (KVB23)	12:35	1.07	348.95	7.50	0.953
5.96	9.06	PF 3, (KVB22)	13:00	0.47	347.65	6.00	1.125
		PF 4, (unnamed	13:25	0.42		2.30	0.024
5.55	8.65	PF 5, (KVB21)	13:40	0.45	346.96	6.00	1.161
5.03	8.43	PF 6, (KVB20)	14:00	0.57	346.43	5.20	1.131
4.52	7.62	PF 7, (KVB19)	14:30	0.52	344.72	6.00	1.130
		PF 8, (Gbely stream)	14:45	0.50		2.60	0.136
4.16	7.26	PF 9, (KVB18)	15:00	0.95	344.01	7.50	1.237
			15.7.2002				
6.97	10.07	PF 1, (KVB24)	10:10	0.67	349.79	5.90	1.025
6.58	9.68	PF 2, (KVB23)	10:35	1.08	348.99	7.65	0.991
5.96	9.06	PF 3, (KVB22)	11:10	0.67	347.70	6.80	1.184
		PF 4, (unnamed	11:35	0.61		2.90	0.013
5.55	8.65	PF 5, (KVB21)	11:55	0.49	346.86	6.65	1.220
5.03	8.43	PF 6, (KVB20)	12:30	0.61	346.39	7.70	1.212
4.52	7.62	PF 7, (KVB19)	13:00	0.64	344.82	6.80	1.124
		PF 8, (Gbely stream)	13:55	0.56		2.30	0.040
4.16	7.26	PF 9, (KVB18)	13:30	1.07	344.19	7.50	1.295
4.15	7.25	PF 10, (B2-Mojš)	14:40	0.77	343.55	8.80	1.316

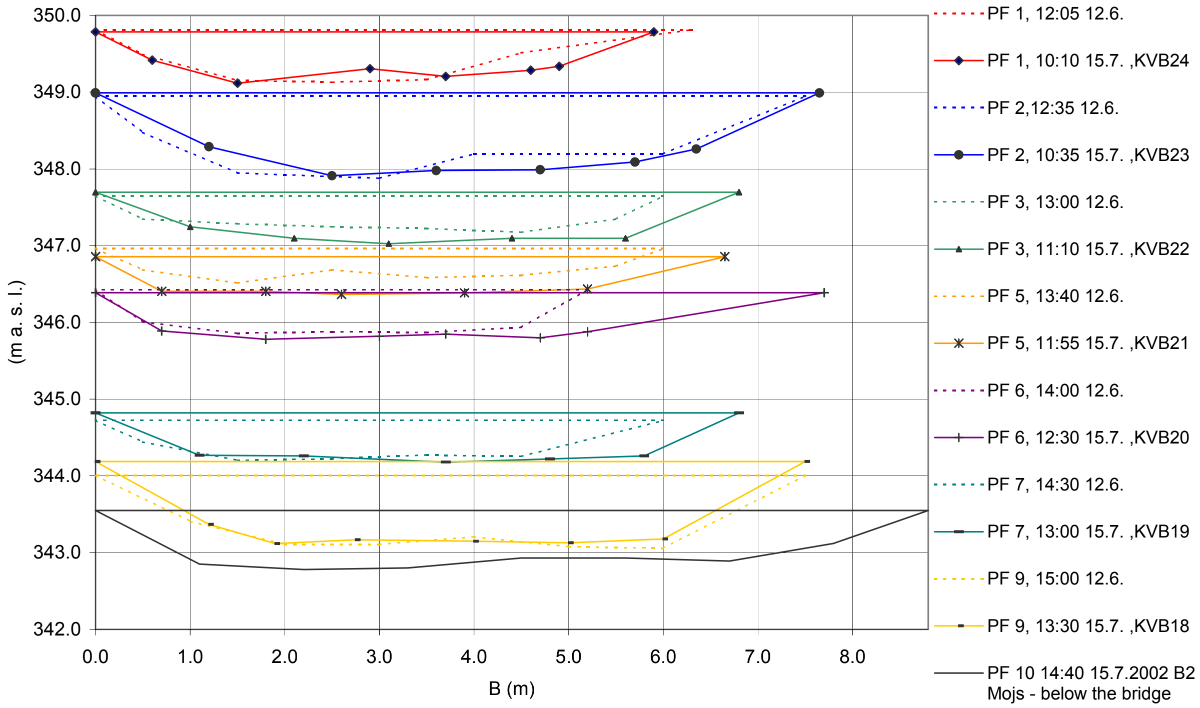
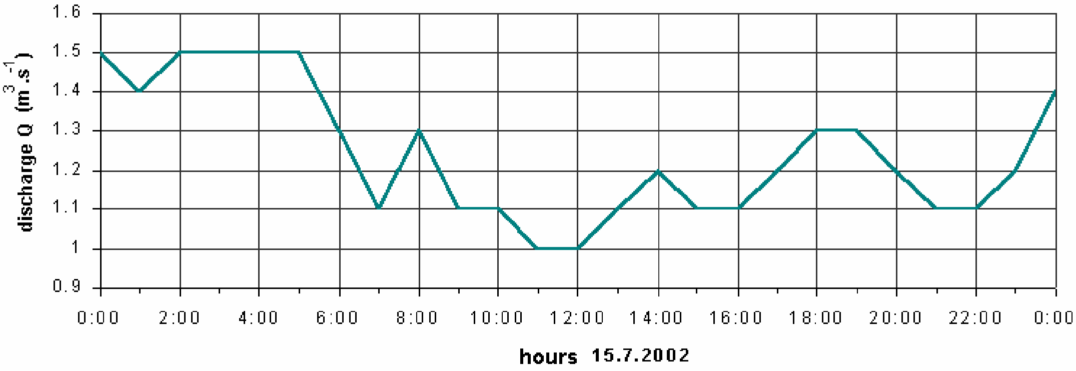


Fig. 2 Measured cross-section profiles of the bio-corridor on Žilina water structure.



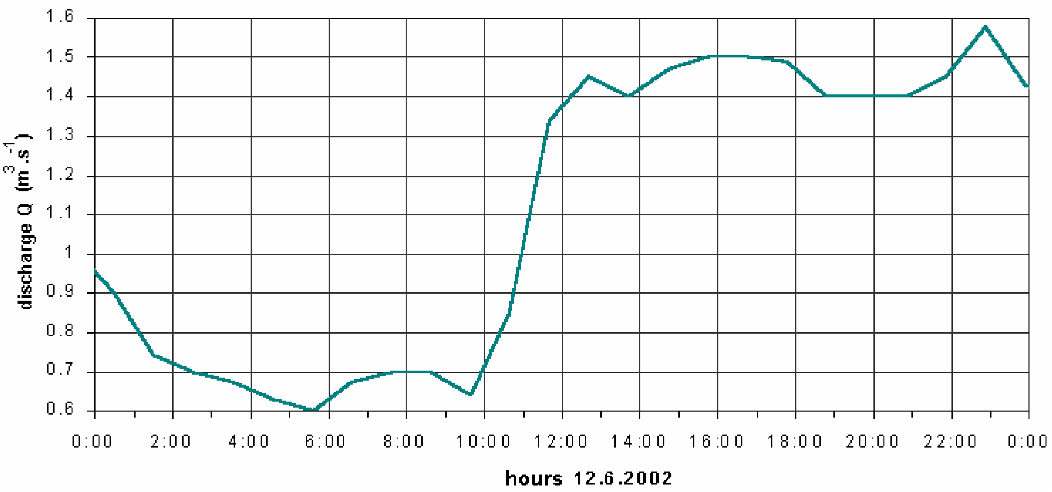


Fig. 3 Course of discharges below the inlet object into the alternate bio-corridor of the WS Žilina in days 12.6. and 15.7. 2002 (measured by VV)

3 Conclusions

Although we have focused our measurements on the section of the alternate bio-corridor starting at the inlet object and reaching up to the bridge below Mojš, we have determined the water level regime in the bio-corridor as applied to the whole of the measured section as a boundary condition of solving the water level mode in the alluvium on the right side of the Váh river at the Žilina WS. The result of the mathematical extrapolation suggests the minimum and maximum water level in the bio-corridor for the calibration of mathematical model at the 7 kilometer long section of the bio-corridor. This is shown in the Fig. 5.

PF	10	15.7.2002 14:40		Mojš – below the bridge - B2			
Perpendicular:	No.	$h_{above\ bottom}$	v	Perpendicular:	No.	$h_{above\ bottom}$	v
		(m)	(m/s)			(m)	(m/s)
I.	1	0.05	0.39	**V.**	1	0.05	0.07
l =	2	0.2	0.16	l =	2	0.2	0.19
1.10	3	0.35	0.21	5.60	3	0.35	0.43
H =	4	0.5	0.17	H =	4	0.5	0.43
0.70	5	0.6	0.05	0.62	5	0.6	0.49
II.	1	0.05	0.25	**VI.**	1	0.05	0.17
	2	0.2	0.27		2	0.2	0
l =	3	0.35	0.63	l =	3	0.35	0.27
2.20	4	0.5	0.52	6.70	4	0.5	0.29
H =	5	0.65	0.45	H =	5	0.6	0.3
0.77	6	0.7	0.56	0.66	6		
III.	1	0.05	0.08	**VII.**	1	0.1	0.01
	2	0.2	0.27		2	0.25	0.02
l =	3	0.35	0.29	l =	3	0.4	0.15
3.30	4	0.5	0.41	7.80	4		
H =	5	0.65	0.6	H =	5		
0.75	6	0.74	0.52	0.43	6		
IV.	1	0.05	0.16			L_{tot}=	H_{max}=
l =	2	0.2	0.37			8.80	0.77
4.50	3	0.35	0.46				
H =	4	0.5	0.41				
0.62	5	0.6	0.56				

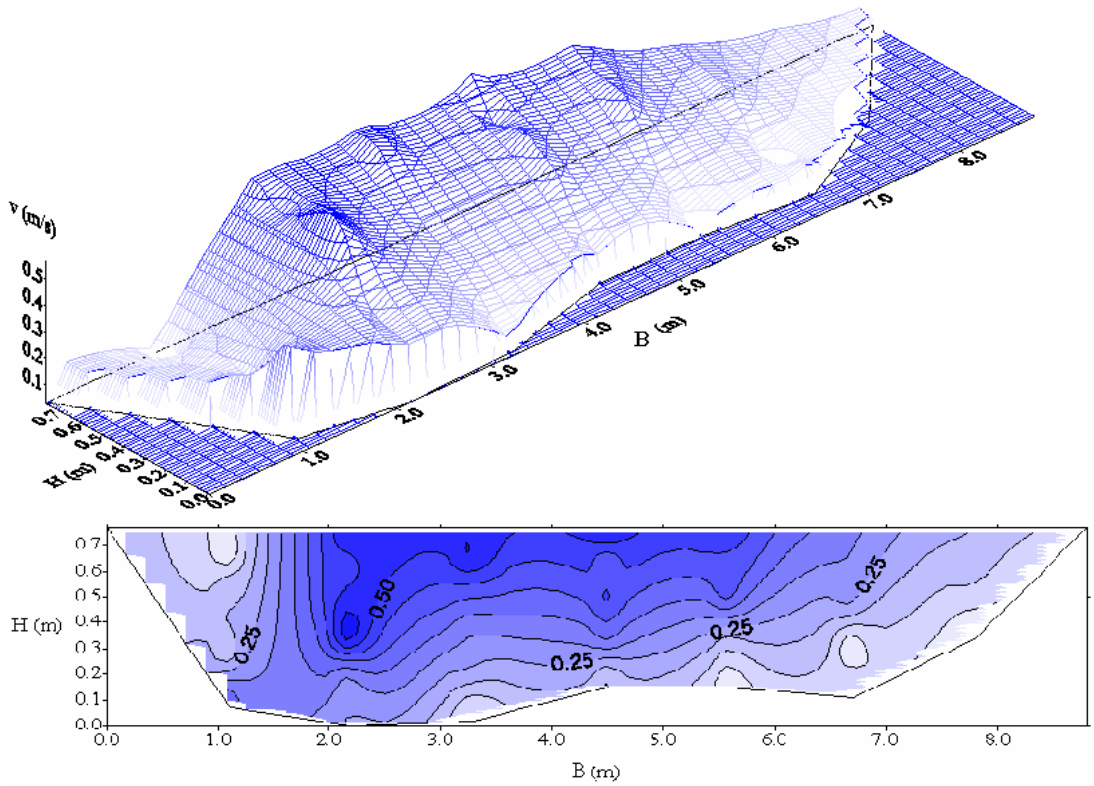


Fig.4 Illustration of the discharge determination in the B2 profile – Mojš below the bridge (PF10).

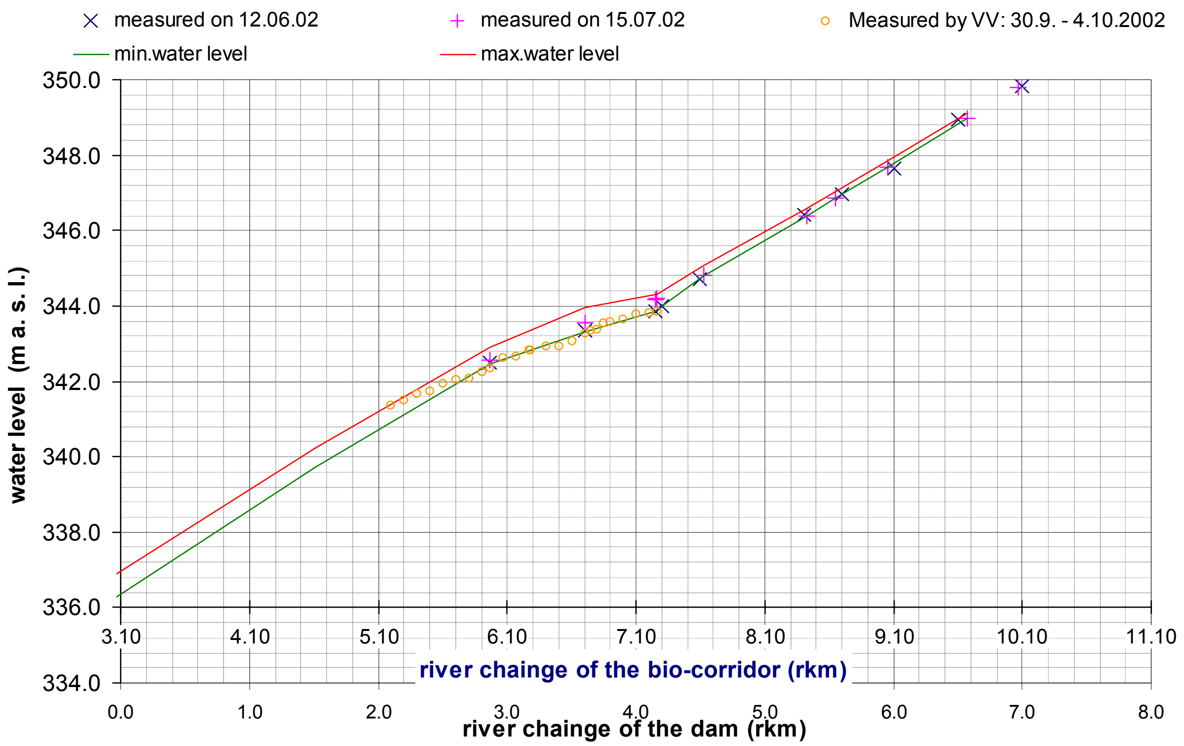


Fig. 5 Determined minimum and maximum water level in bio-corridor for the modelling calibration.

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Urbanization Impact on Hydrology

Marija Šperac

Faculty of Civil Engineering Osijek, Drinska 16a, 31000 Osijek, Croatia; msperac@most.gfos.hr

Abstract

The hydrologic cycle consists of inflows, outflows, and storage. Inflows add water to the different parts of hydrologic system, while the outflows remove water. Storage is the retention of water by parts of the system. Because water movement is cyclical, an inflow for one part of the system is an outflow for another.

Urbanization has an irreversible impact on natural drainage patterns and flows in the receiving water bodies impacted by urban development. Uncontrolled development or past development in the watershed that did not considered the impact on hydrology, watershed encroachment, morphology and ecology of the receiving water body system have had detrimental effect on the receiving water body, watershed development and downstream uses of the water body. Urbanization has had significant impacts on the hydrology of the environment by controlling: nature off runoff, rates of soil erosion and delivery of pollutants to rivers, streams, lakes and oceans.

In cities and suburbs, where much of the land is paved or covered rainwater runs off as much as ten times fasted than unpaved land. Since this water cannot be absorbed into the soil, it flows rapidly down storm drains or trough sewer systems, contributing to floods and often carrying debris and other pollutants to streams. Urbanization has caused the surface runoff to increase and replenishment of underground water to decrease.

Large cities and urban sprawl particularly affect local climate and hydrology. This alters the rates of infiltration, evaporation and transpiration that would otherwise occur in a natural settings. Because various effects determine the amount of water in the system and can result in extremely negative consequences for river watersheds, lake levels, aquifers, and the environment, it is vital to learn about and protect water resources.

Key words: hydrologic cycle, urbanization, uncontrolled development, watershed, protect water resources.

1 Introduction

In 1985, over 43 % of the world's population resided in urban areas. This number is expected to grow dramatically in the next few decades. Researchers predict that by 2025 over 60 % of the Earth's people will live in urban areas. Most of this growth will occur in developing countries. Urban areas in less developed countries (LDCs) are currently growing at 3.5 % per year as migration economic opportunities occur in the city. Growth is less than 1 % per year in developed countries as they areas underwent urbanization almost a century ago. In fact, in 1995, over 70 % of the population of North America and Europe live in urban areas.

One of the striking features of the distribution of the world's population is the tendency for large human concentrations to occur near vast expanses of water. Since the beginning of the industrial revolution, urban development have influenced the flow and storage of water, as well as the quality of available fresh water.

Urbanization has had significant impacts on the hydrology of the environment by controlling:

- Nature of *runoff* (water from precipitation or irrigation that does not evaporate or seep into the soil but flows into rivers, streams, or lakes, and may carry sediment);
- Rates of soil erosion; and
- Delivery of pollutants to rivers, streams, lakes and ocean.

Large cities and urban sprawl particulary affect local climate and hydrology. Urbanization is accompanied by accelerated drainage of water through road drains and city sewer systems, which even increases the magnitude of urban flood events. This alters the rates of infiltration, evaporation, and transpiration that would otherwise occur in a natural setting. The replenishing of ground water aquifers does not occur or occurs at a slower rate.

2 The hydrologic cycle

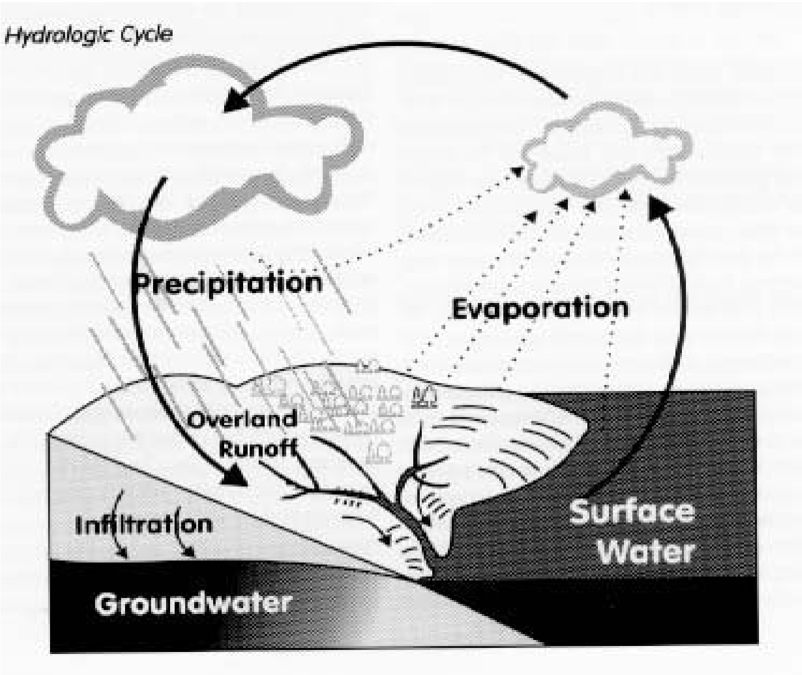

Fig. 1: Hydrologic cycle

Water continually changes from one form to another in what is called the hydrologic cycle. Water falls to earth in the form of rain, snow or sleet. Some rainfall flows into rivers, lakes, or oceans and is surface water. Other precipitation seeps through the soil and becomes groundwater. Some of the water in the soil evaporates; some is used by plants and passes into the air through the plant's leaves by a process called transpiration. Both evaporation and transpiration return water to the air where it forms clouds and falls back onto the ground as rain, completing the hydrologic cycle. The hydrologic cycle is illustrated in Figure 1.
The hydrologic cycle is a natural machine, a constantly running distillation and pumping system. The sun supplies heat energy, and this together with the force of gravity keeps the water moving; from the earth to the atmosphere as evaporation and transpiration, from the atmosphere to the earth as a streamflow and ground-water movement. As a cycle, this water system has neither beginning nor end, but from man's point of view the oceans are the major

source, the atmosphere is the deliverer, and the land is the user. In this system, no water is lost or gained, but the amount of water available to the user may fluctuate because of variations at the source, or more usually, in the delivering agent. In the geologic past, large alterations in the cyclic roles of the atmosphere and the oceans have produced deserts and ice ages across entire continents. Even now, small alterations of the local patterns of the hydrologic cycle produce floods and droughts.

3.Urbanization impact on the water cycle

The earth's water supply remains constant, but man is capable of altering the cycle of that fixed supply. Population increases , rising living standard, and industrial economic growth have place greater demands on our natural environment. Our activities can create an imbalancein the hydrologic equation and can affect the quantity and quality of natural water resources available to current and future generations.

Water use by households, industries, and farms have increased. People demand clean water at reasonable costs, yet the amount of fresh water is limited and easily accessible sources have been developed. As the population increases, so will our need to withdraw more water from rivers, lakes and aquifers, threatening local resources and future water supplies. A large population will not only use more water butt will discharge more wastewater. Domestic, agricurtural, and industrial wastes, including the intensive use of pesticides, herbicides and fertilizers, ofter overload water supplies with hazardous chemicals and bacterias. Poor irrigation practices raise soil salinity and evaporation rates. These factors contribute to a reduction in the availability of potable water, putting even greater pressure on exissting water resuurces.

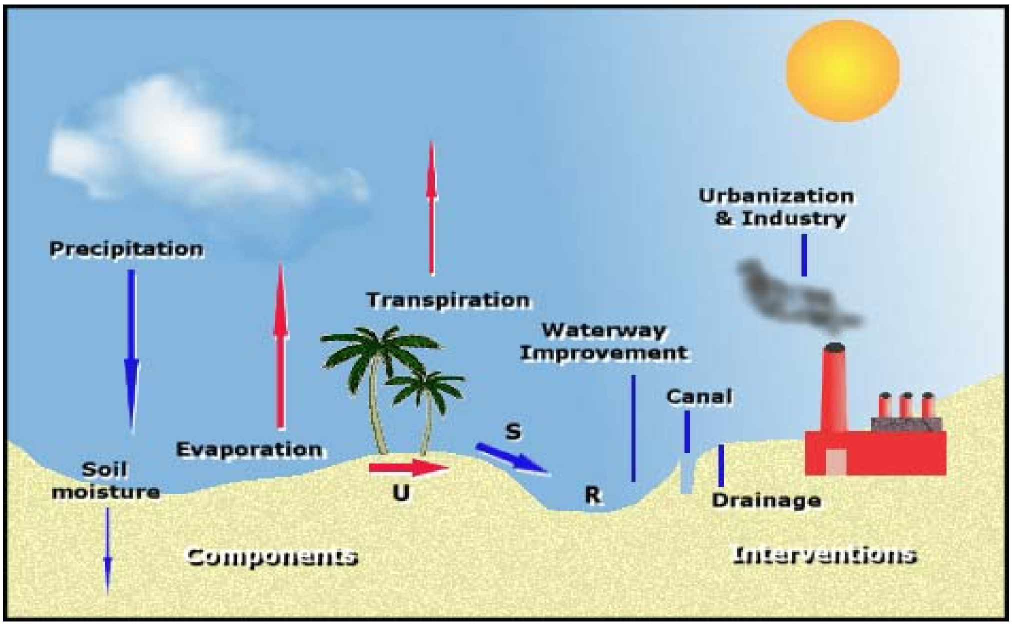

Fig. 2: Major components and human interventions in the hydrologic cycle

In cities and suburbs, where much of the land is paved or covered -- streets, buildings, shopping centers, airport runways-- rainwater runs off as much as ten times faster than on unpaved land. Since this water cannot be absorbed into the soil, it flows rapidly down storm

drains or through sewer systems, contributing to floods and often carrying debris and other pollutants to streams.
Figure 2 illustrates the components of water balance in the hydrological cycle. [Precipitation is either absorbed by the ground (A), or it becomes surface runoff (S). The absorbed water is divided between baseflow (B) (underground runoff to rivers) and evapotranspiration (E) , i.e., A=B+E). The total runoff (T) is equal to the surface runoff plus the baseflow.] Urbanization has caused the surface runoff to increase, and the replenishment of underground water to decrease.

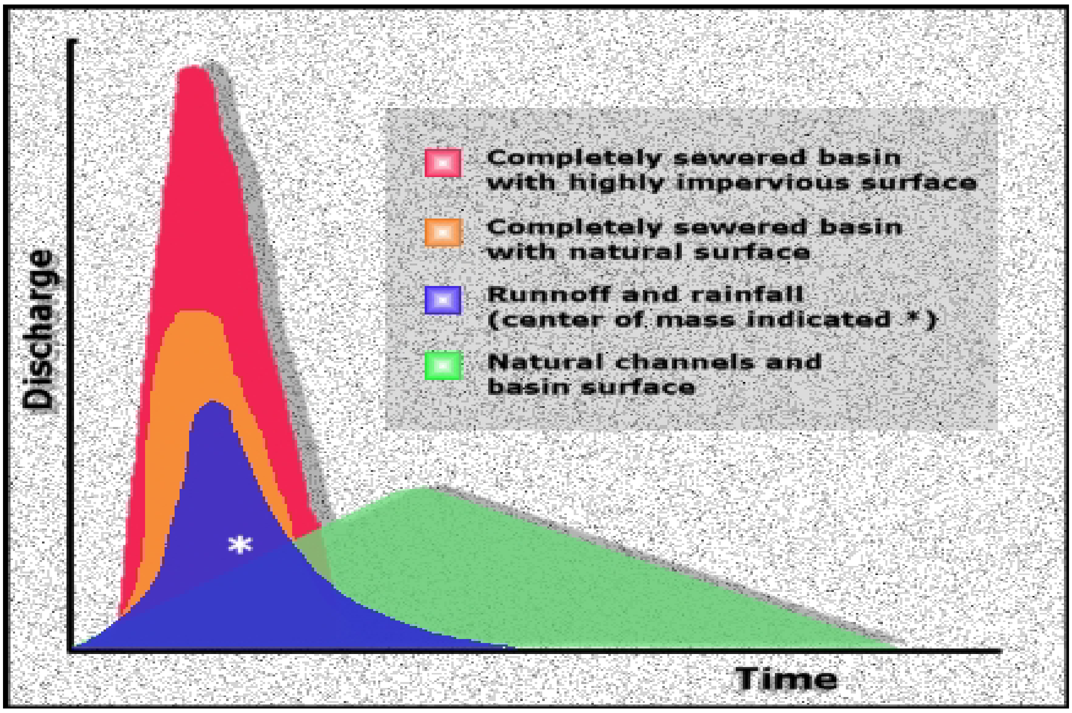

Fig. 3: The effect of urbanization

Impervious surfaces and sewers cause runoff after a rainstorm to occur more rapidly and with a greater peak flow than under nonurban conditions (see Figure 3). In turn, larger peak flows increase the frequency of floods. In general we can say that urbanization increases flood volume, frequency and peak value . The net result of this flushing effect may be to increase turbidity, pollutant loads, and bank erosion.
It has been observed that the size of small floods is increased by urbanization, while during large, infrequent floods there are not significant differences between the way rural and urban areas behave.

Urban development causes changes in hydrology, depletion of supplies and aquatic pollution.

1. Hydrology. Urbanization completely alters the landscape of an area, impacting local hydrology. Roads, parking lots and buildings do not absorb precipitation as vegetation does which increases runoff. In addition the water quickly drains from streets into storm sewers. Often wetlands, which efficiently act to absorb excess water, are filled in. The development of floodplains for homes and industry, remove land where excess runoff could go. With all the excess runoff and lack of adequate disposal places, there is an increase in both the number and

magnitude of floods. The removal of vegetation from stream banks for development, leaves the water exposed to the elements so that it is warmer in the summer and colder in the winter.

2. Consumption. Urban areas obtain their water from either surface sources or from ground water supplies. Not only does an increasing population put more demand on water resources but urbanized areas generally have a greater water demand per capita than rural areas. In most urban areas the price of water is relatively low (due to government subsidies) so there is no incentive for conservation. In addition, a lot of water is lost due to old, leaky distribution systems. Those cities without easy access to surface water supplies often use ground water. This can lead to land subsidence and salinization if withdrawal rates exceed the rate of recharge.

3. Water Pollution. There are many ways that urbanization contributes to the pollution of both surface and ground water. The dumping of wastes into waterways is common as they dilute and disperse pollutants. The current volume of wastes added is far beyond the capacity of these waters for effective dilution and dispersal. Over 220 million urban residents do not have access to clean drinking water. Some sources of aquatic pollution include industry, sewage, runoff and sediment.

Urban runoff pollutes waterways as it contains pollutants from hundreds of sources. As water travels over the surface it picks up pollutants from cars, fertilizers, pesticides, etc. Most of the time, runoff often enters waterways without treatment. In an urban area, sewage can be a major source of water pollution. Sewage is a source of nutrients that can lead to eutrophication of lakes. Sewage also contains many pathogens such as cholera bacterium, hepatitis virus, and salmonellae all of which are hazardous to human health. In the developing world, 90% of the sewage are discharged, untreated into surface waters. Treatment usually removes only the pathogens so the nutrients remain to pollute the water.

3.1.Sustainable development and Agenda 21

Sustainable development was the central theme of the UN Earth Summit at Rio de Janeiro in 1992, which called on governments to produce their own strategies for sustainable development. In the UK, the Government updated its national strategy in May 1999. Parallel to this the Local Government Management Board has published Local Agenda 21 - A framework for local sustainability. Local authorities have their own Agenda 21 strategies.

Cities, towns and villages create demands on the environment by using resources and producing waste. The built environment is therefore one area where the strategies of sustainable development should be put into practice.

Sustainable development and Local Agenda 21 was introduced to manage the balance between social, economic and environmental requirements minimising the conflict that can exist between economic development and the protection of the environment.

Sustainable development as defined by the Brundtland Commission (a commission chartered by the United Nations) is "development which meets present needs without compromising the ability of future generations to achieve their needs and aspirations."

Sustainable drainage is a concept that includes long term environmental and social factors in decisions about drainage. It takes account of the quantity and quality of runoff, and the amenity value of surface water in the urban environment. Many existing urban drainage systems can cause problems of flooding, pollution or damage to the environment and are not proving to be sustainable.

Drainage systems can be developed in line with the ideals of sustainable development, by balancing the different issues that should be influencing the design. Surface water drainage methods that take account of quantity, quality and amenity issues are collectively referred to as Sustainable Drainage Systems (SUDS). These systems are more sustainable than conventional drainage methods because they:

- Manage runoff flowrates, reducing the impact of urbanisation on flooding
- Protect or enhance water quality
- Are sympathetic to the environmental setting and the needs of the local community
- Provide a habitat for wildlife in urban watercourses
- Encourage natural groundwater recharge (where appropriate).

They do this by:

- Dealing with runoff close to where the rain falls
- Managing potential pollution at its source now and in the future
- Protecting water resources from point pollution (such as accidental spills) and diffuse sources.

They may also allow new development in areas where existing sewerage systems are close to full capacity, thereby enabling development within existing urban areas.
Urban drainage is moving away from the conventional thinking of designing for flooding to balancing the impact of urban drainage on flood control, quality management and amenity.

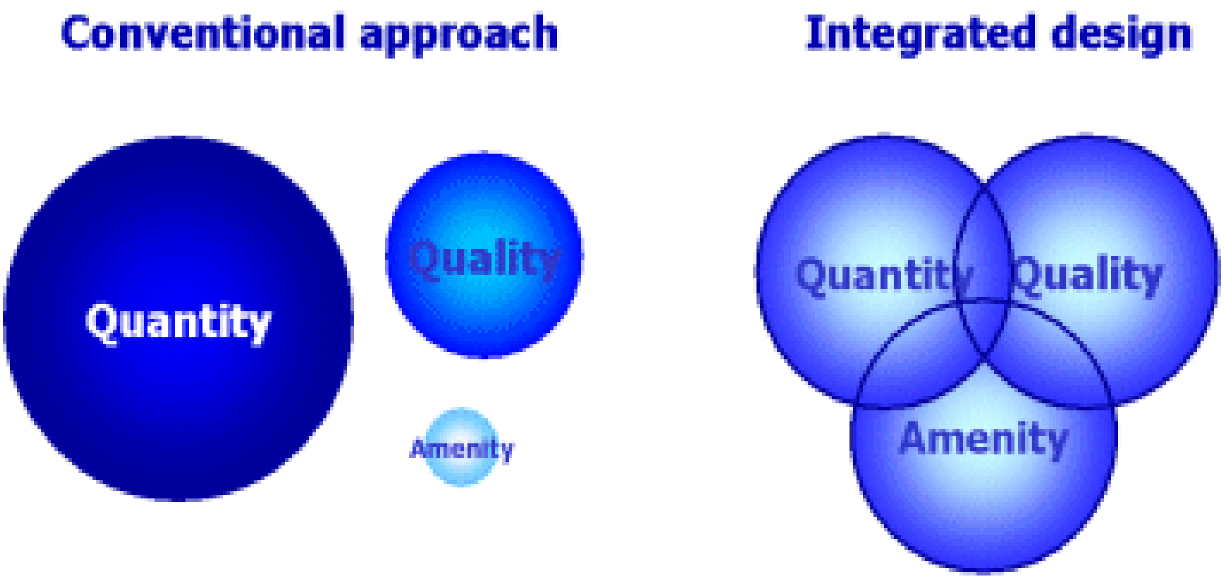

Fig. 4: Urban drainage: conventional approach and integrated design

4 Conclusion

As population grows the urbanisation of society is inevitable. Urbanisation leads to an increased impact on the environment; the 'ecological footprint' of cities is spreading.
The impact of growth on all areas of society must be acknowledged. Sustainable growth requires an evolution in the way urban areas carry out their activities such as resource use and the movement of people and goods. The physical infrastructure in addition to social and economic processes must evolve to acknowledge the challenges of growth.
Sustainable development has been defined as development that meets the needs of the present without compromising the ability of future generations to meet their own needs. However, sustainable urban development implies a process by which sustainability can be attained, emphasising improvement, progress and positive change, incorporating both environmental and social dimensions.
Sustainable urban development highlights the need for reform of market mechanisms to achieve environmental goals and the achievement of a balance with social and economic considerations.
Several themes common to all definitions of sustainable urban development have emerged:

- A change in the quality of growth.
- The conservation and minimisation of the depletion of non-renewable resources.
- A merging of economic decisions with those on the environment.
- A strong consideration of the needs of future generations.

"By the year 2000, half the world's people will be living in cities. The urbanization of society is part of the development process, and cities generate 60 percent of gross national product. A growing number of cities, however, are showing symptoms of the global environment and development crisis, …." *Agenda 21 ("The Earth Summit") and UN Sustainable Cities Programme.*
Cities should be healthy, providing housing and employment opportunities, meet environmental standards and be sustainable. Sustainability needs to be a addressed on a global scale, reforms need to concentrate on the interaction of the urban environment with the global economy and environment.

Various effects determine the amount of water in the system and can result in extremely negative consequences for river watersheds, lake levels, aquifers, and the environment as a whole. Therefore, is vital to learn about and protect our water resources.

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Impact of Drought on Wheat Yields in Different Production Regions

Šťastná Milada [a], Eitzinger Josef [b]

[a] Institute of Landscape Ecology, Mendel University of Agriculture and Forestry Brno, Zemědělská 1, 613 00 Brno, Czech Republic, stastna@mendelu.cz
[b] Institute of Meteorology and Physics, University of Agricultural Sciences, Türkenschanzstraße 18, A-1180 Wien, Austria, sepp@tornado.boku.ac.at

Abstract

The analysis of the impact of different annual drought patterns was carried out on winter wheat by using selected simulation model CERES-Wheat. The analysis includes different soil conditions and crop management options in the Czech Republic and Austria. The water stress indicators were modelled for determinated wheat growth stages under the present climate conditions and drought occurrence pattern for water limited production. Adaptation measures such as soil cultivation and mulching can significant improve the rooting conditions and soil water storage capacity. Several of these impact factors are analyzed and quantified in our simulation study in view to assess impacts on wheat drought sensitivity, growth and yield and its consequences for sustainable winter wheat production of the selected region.

Key words: drought, winter wheat, model, production

1. Introduction

Shortage of water is an important, and in some cases, the chief cause of variation in wheat yields in many parts of the world. Also the impact of frost, heat, drought, diseases, insects, and weeds can be more accurately predicted with a clear picture of the relationships between growth stage and plant response to stress. The impacts of climate variabilities and change on crop production and water balance have been studied with use of crop models in many studies (Eitzinger et al., 2000; Semenov and Porter, 1995; Bacsi and Hunkár, 1994; Mearns et al., 1992). The hydrological cycle will be affected through a change in precipitation, evapotranspiration, the magnitude and timing of run-off, as well as through a possible change in intensity and frequency of floods and droughts (Watson et al., 1996). Increased CO_2 concentrations can affect plant growth directly through stimulation of photosynthesis and reduction of transpiration and, as a result, can improve water use efficiency (WUE) (Rosenberg et al, 1990). Crop simulation models for a variety of crops and applications including soil water balance assessment have been described (e.g. Majerčák et al., 1994; Robinson et al., 1992; Wilhite, 1993). The aim of the study was to highlight the critical water balance parameters and water stress situations for the selected cultivars under the present climatic conditions (1xCO_2 weather) and modified (2xCO_2) climate scenarios (combined effect).

2. Material and Methods

Two experimental fields were chosen for the study. The first one in Žabčice (latitude 49° 01′ N, longitude 16° 37′ E and altitude 179 m above the see level) is located in the south part of the Czech Republic. The long term mean of yearly precipitation is 480 mm, the mean annual temperature is 9,3 °C. The second experimental field in Gross-Enzersdorf (Marchfeld), is located within the same climatic region (latitude 48° 12′N, longitude 16° 34′E and altitude 153 m above the see level) in north-eastern part of Austria. The mean annual sum of precipitation is 577 mm and the mean annual temperature is 9,9 °C. Winter wheat (*Triticum aestivum* L.) was grown as an experimental crop at both localities. Cultivar "Perlo" in Gross-Enzersdorf and cultivar "Hana" in Žabčice stations, respectively, to use experimental data for model validation. The soil type in Žabčice belongs to the subgroup Oxyaquic Cryofluvents (USDA Classification, 1975). The soil at the experimental field in Gross-Enzersdorf could be classified as the soil type 19 according to the Austria Soil Classification (ÖBK). The soil is described as chernozem on fine calcareous sediments over gravel and sand.

The CERES (Crop Environment REsource Synthesis) - Wheat model (Ritchie and Otter, 1985) was designed to simulate the effects of cultivar, planting density, weather, soil water, and nitrogen on crop growth, development, and yield. There are four groups of input data necessary in order to prepare and run model simulation. The minimum data set of the weather data includes daily values of maximum and minimum temperature, global radiation and precipitation. Genetic coefficients were derived partly from literature sources and partly from experimental data from test sites. Soil input data were derived from soil pits that were situated directly at the experimental site. The grain yield has been selected as the evaluation parameter for CERES-Wheat model in both localities. To generate series representing changed climate conditions, the generator parameters were modified on the basis of the climate change scenario. The weather generator has been validated in detail in Dubrovský (1996, 1997) and was found satisfactory for use in the crop growth modelling. Recent transient runs of general circulation models (GCMs) were used to develop climate change scenarios (Dubrovský et al., 2002). Based on the results obtained, the GCM that best reproduced the present climate was selected to define the scenario ECHAM4/OPYC3 (ECHAM).

Input weather files were created to run the simulations in the selected locations. The observed weather data of the period of 1985-1993 were used for the simulations. For the 2xCO_2 weather the monthly changes in the relevant scenario was applied to the daily weather data. Winter wheat growth and development were simulated for 2 (a-b) various conditions: **(a)** present conditions (1xCO_2 weather, 330 ppm), representing no change in used weather input files (present climate) and in CO_2 concentration in the atmosphere (330 ppm), **(b)** combined effect (2xCO_2 weather, 660 ppm), representing a change in weather input compared with the present climate (according to scenarios) and in CO_2 concentration in the atmosphere (660 ppm). The obtained simulation results were analyzed to assess the impact of water balance and water stress to the endurance of wheat growing stages by Zadock (Zadock et al., 1974) as well as on photosynthesis and growth in these stages.

3. Results and discussions

The model was validated using observed and simulated grain yields for the years 1985 to 1993 after successful calibration at both locations with coefficient of determination 0.73 and standard deviation 582 kg.ha^{-1} for Žabčice ("Hana" winter wheat cultivar) and 0.76 with standard deviation 696 kg.ha^{-1} for Gross-Enzersdorf ("Perlo" cultivar). Simple linear regressions were computed to determine the R^2 value between observed and simulated data and the simulated yield percentage within 10 per cent of the observed yield.

To show the impact of soil water balance and water stress on wheat yield for particular growing stages for both localities, yields and water stress values 0-1 (0 – no water stress, 1 - the highest water stress) of the years 1985-1993 were simulated by CERES-Wheat model.

Present conditions (1xCO_2 weather, 330 ppm) showed folloving results: Simulated rain-fed winter wheat yield values varied by around 3000 kg.ha^{-1} at both locations, if no groundwater impact was assumed, which is less than half of the simulated potential yield (no water stress) of about 6600 kg.ha^{-1} at both sites. Such low yields occurred because of significant water stress during most growing stages. One of the reasons for the high water stress levels during the growing period was the low initial soil water content of winter wheat in autumn, which was set at the wilting point to show more clearly the reaction of winter wheat to water deficit under extreme conditions. These conditions are realistic for Gross-Enzersdorf in some years, but not for Žabčice, where groundwater has an impact on the rooting zone. When groundwater impact was simulated at Žabčice, the yield under rain-fed conditions (6571 kg.ha^{-1}) came close to the potential yield for that site due to only very low water stress during all phonological stages. The duration of the simulated total growing period, based on the real local conditions, differed by 37 days between the two sites, caused only by the difference during winter dormancy (Zadock´s stage 0-30). Also, the sowing date in autumn was set 10 days earlier at Žabčice.

Combined effect (2xCO_2 weather, 660 ppm) increased CO_2 concentration affect wheat growth directly through stimulation of photosynthesis, reduced transpiration and improved water use efficiency, as e.g. reported by Rosenberg et al. (1990). Rain-fed yield was simulated and the best results in terms of absolute yield and less water stress being obtained with the ECHAM scenario at Žabčice (groundwater impact, 7496 kg.ha^{-1}) at Gross-Enzersdorf (3806 kg.ha^{-1}). There was no indication of water stress at any growing stage for the groundwater impact case at Žabčice. The total growing period at Žabčice decreased 31 days on average for the scenario compared with present conditions, but this was caused only by the shortened winter dormancy period (represented by Zadock stage 0-30). As no water stress occurred, the simulated yields were relatively high (7455 kg.ha^{-1} average), almost the level of potential yield, and increased 13.5 per cent compared with the present weather case. Assuming no groundwater impact at Žabčice, the rain-fed yield increased 52.5 per cent to only 5496 kg.ha^{-1} on average for all scenarios. At the Austrian location (Gross-Enzersdorf , no groundwater impact) rain-fed yield was lower than for the same case (without groundwater impact) at Žabčice. Water stress at Gross-Enzersdorf occurred regularly during the growing period and the yield increased about 39.1 per cent with a reduction in winter dormancy of eight days (Zadock stages 0-30) on average. Except for the ECHAM scenario at Gross-Enzersdorf, the highest increases in yield compared with current conditions were found for the combined effect at both locations in cases where there was no groundwater impact. However, at Žabčice the highest absolute yields were obtained with groundwater impact.

The main explanation of continuously higher water stress levels at the Austrian location (and by that a stronger decrease of grain yields) compared to Žabčice with groundwater impact is related to differences in the soil water balance during the winter wheat growing period (Fig. 1), as there is no ground water impact to the rooting zone in Gross-Enzersdorf. The comparison between total biomass accumulation and soil water balance at both experimental sites shows a big difference between the two sites. Through much lower initial available soil water storage in Gross-Enzersdorf with no impact from groundwater, soil water deficit occurs faster despite the total precipitation amount is higher than in Žabčice (Fig. 1) and its distribution is very similar.

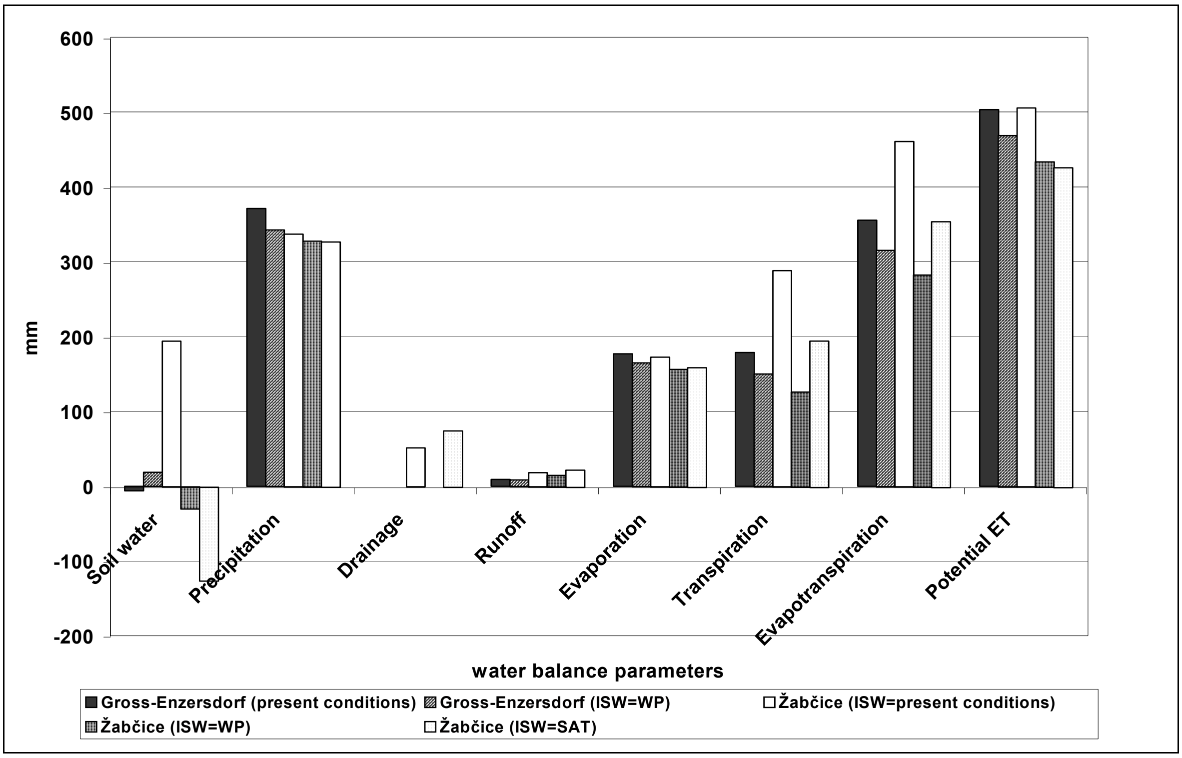
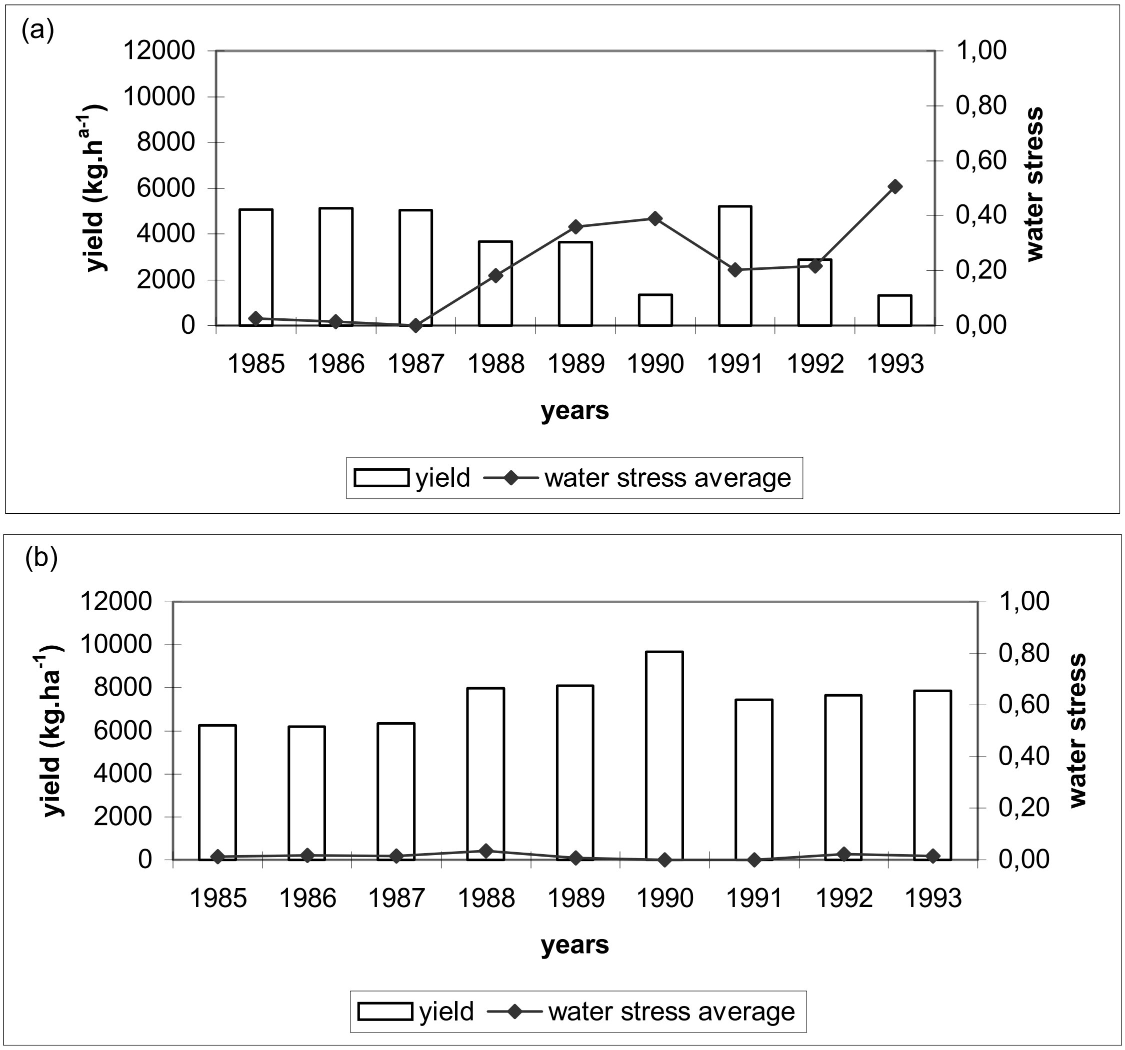


Fig. 1: Comparison of soil water balance parameters – average values of nine simulated years based on 1985-1993 daily weather in Gross-Enzersdorf and Žabčice locations. ISW (initial soil water); WP (wilting point and no groundwater impact); SAT (soil saturation and groundwater impact).

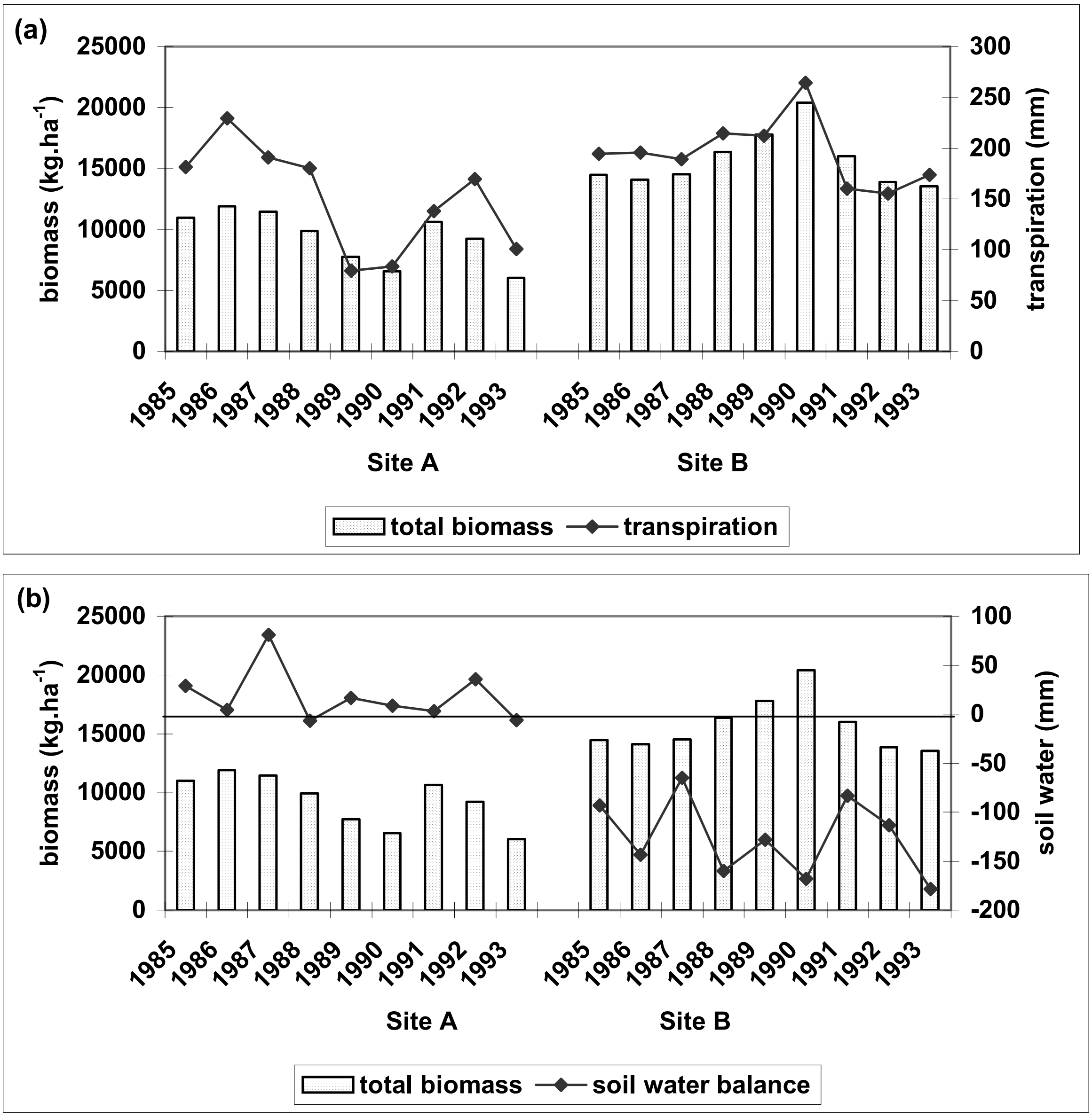
The average lower soil water storage during the growing period leads to less biomass accumulation and lower transpiration through frequently occurring drought stress together with no drainage, low runoff and higher evaporation (less biomass and ground cover) in Gross-Enzersdorf. Žabčice in contrast has much higher soil water storage in the rooting zone through the impact of groundwater, which can act as a buffer during drought periods.

Yields and water stress values 0-1 (0 no water stress, 1 the highest water stress) for 1985 to 1993 simulated by the CERES-Wheat model with the modified weather files from the ECHAM scenario (combined effect) were chosen to show the impact of soil water balance and water stress on wheat yield for particular growing stages for both localities. ECHAM shows a strong decrease in precipitation in April of 20 %, which highlights potential water stress effects during this critical month under changed climate. During the first Zadock´s stage only low water stress (less then 0.1) occurred at either locality, and this stress level did not influence the final wheat yield, although Žabčice recorded this stress every year except for 1990. During the second stage, very high water stress occurred in 1989 and 1991 at Gross-Enzersdorf compared with Žabčice, where only low stress occurred in 1988. However, high values at this stage did not have a significant impact on the grain yield as well (e.g. high water stress level, but also high yield in 1991). Simulated water deficit appeared also during the third stage at Gross-Enzersdorf, where the water stress values reached 0.7 in 1989 and 1993.



Figs. 2 a, b: Mean water stress levels of winter wheat related to simulated rain-fed yield of a) Gross-Enzersdorf (no groundwater impact to rooting zone) and b) Žabčice (groundwater impact to rooting zone) for the years 1985-1993 under the ECHAM scenario (combined effect).

The most yield-sensitive growing stage was during the grain-filling period at Gross-Enzersdorf (the fourth and especially the fifth Zadock´s stage), which can be seen by comparing the yield and water stress levels in 1990 and 1991. The highest water stress indicators correspond directly with the lowest yields (1990 and 1993) for the whole nine-year period. Figures 2a, b shows a comparison of average water stress and wheat grain yield values for all stages at both localities In general, high water stress values during 1990 and 1993 caused very low yields, but the mean stress value in 1989 and 1991 did not significantly influence the yield, because of a lower water stress level during the grain-filling period. This result confirms the different sensitivity of winter wheat growing stages to water stress occurrences that has been demonstrated experimentally (Kastelliz and Ruckenbauer, 2000). The continuously higher water stress levels at the Austrian location (and hence a greater decrease in grain yields) compared with Žabčice can be explained by differences in the soil water balance during the winter wheat growing period (Figs. 3a, b). Because of the much lower available soil water at Gross-Enzersdorf, soil water deficit occurred faster, despite the fact that total precipitation was higher than at Žabčice and the distribution was very similar (not shown). This resulted in less biomass accumulation and lower transpiration (Fig. 3a) at Gross-Enzersdorf.



Figs. 3 a-b: Relation between simulated total biomass and transpiration (a) and soil water balance (b) of the winter wheat growing period for the ECHAM climate scenario.

There was also no drainage, low runoff and higher evaporation (less ground cover). Žabčice, by contrast, had much higher soil water storage in the rooting zone by the impact of groundwater, which can act as a buffer during drought periods. Total biomass accumulation and yield was therefore significantly higher at Žabčice for rain-fed conditions as no water shortage occurred in spite of increased soil water use. Fig. 3 b shows the calculated simulated soil water balance deficit. At Žabčice negative values are shown for the winter wheat growing period, which would represent the input of groundwater, if we assume no change in soil water storage on a year-to-year basis. It was 194 mm during the growing period under present conditions and 126 mm for the ECHAM scenario. The simulated hypothetical case of no groundwater impact at Žabčice (initial soil water content in autumn was set at the wilting point) highlights the importance of groundwater (or irrigation) for crop yields at that site for current conditions as well as under the applied climate scenarios. With no groundwater impact or decreasing groundwater table, rain-fed yield levels at Žabčice would decrease to levels similar to those at Gross-Enzersdorf. It also clearly shows the simulated increase of water use efficiency under 2xCO_2 scenarios: despite higher yield levels, crop transpiration significantly dropped compared to current conditions.

4. Conclusion

Both agricultural sites with similar climatic conditions showed a simulated decrease in water stress and an increase in yields under future climate scenarios (combined effect of CO_2 increase in the atmosphere and change in weather) under the model assumptions and limitations. The impact of groundwater to the rooting zone showed strong impact on water balance and yield level at the site in Žabčice and is the main reason of the difference in yield levels between the two locations. There was found also a shortening of growing period for both sites under unchanged production technique. There is strong evidence that, especially for soils with low soil water storage capacity or no groundwater impact to the rooting zone, irrigation or water saving production techniques will remain their importance to reach the full production potential.

Acknowledgement

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Threshold phenomena of soil draught starting

Július Šútor, Milan Gomboš, Jozef Ivančo
Institute of hydrology of the Slovak Academy of Sciences, Bratislava

Abstract

Extreme meteorological phenomena also cause an extreme reduction of the water supply in the soil aeration zone. This reduction is accompanied by certain typical states. One of the limit states is referred to as soil drought. It corresponds to such a water content when plants suffer from a permanently deficient water supply and they wilt. Water content is characterised by hydrolimits in soil physics. The hydrolimit of wilting point corresponds to soil drought. The characterising point in clay-loam soils is also the water soil content when cracking process starts.

The wilting point and moisture level were used as soil drought indicators in this paper. The monitored values of water supply in soil aeration zone of the Žitný ostrov and the East Slovakia Lowland are the examples.

Introduction

Water supply in the study horizon of unsaturated zone is under a permanent effect of water flows across its delineated limits. Taking into account the total thickness of aeration zone, the soils surface and ground water level are its upper and lower limits respectively. The water flows are realised in both directions by infiltration and evapotranspiration processes across the upper limit. The water flows are also realised across the lower limit in both directions i.e. by capillary flow into the unsaturated soil zone and by penetration into the lower horizons or into the ground water level.

The unsaturated zone responses to the water flow crossing its limits by
changing its overall volume in time depending on intensity of the flows and variability of their direction.

Climatic changes which manifest through

- drop of precipitation,
- rising temperature and the accompanying phenomena,
- prolongation of dry spells and
- regional decrease of the ground water level,

affect the dynamics of the water soil supply by influencing the water flowing across the upper and lower limits of the soil aeration zone.

The overall water volume and dynamics in unsaturated soil zone in the site of interest is identifiable in two ways:

The first relies on direct monitoring of its vertical moisture with simultaneous observation of the position of ground water level (Šutor, 1999). Additional relevant information about the site or water content is the meteorological data provided by the basic network of the Slovak Hydrometeorological Institute.

The second procedure relies on calculations. Numeric simulation using mathematical models of water regime of soil aeration zone has been proved as a successful method. While monitoring provides data in real time or retrospective data, the data of numerical simulation can be also obtained in view of developmental trends of climatic changes.

Extreme drop of water supply in soil aeration zone leads to soil drought. This situation can be quantified from various aspects. This paper presents such quantification from the viewpoint of canopy.

Material, methods and results

Two characteristics are selected for the quantification of the water content in soil aeration zone. The first is derived from the analysis of the monitored water content regime in a

particular locality during vegetation period. The second is the basic characteristics of soil shrinkage of clay-loam soils determined in laboratory conditions. The methodological procedures used are different for each of the two.
The methodological procedure in the first case is based in processing of the regime of integral water contents in soil aeration zone at the locality Lehnice in the Žitný ostrov region. (Šútor – Takáč – Sobocký, 2001). The monitored regime of water content in soil aeration zone in period from March 7th to November 9th 1993 was chosen for analysis. The aeration zone thickness of the site was 80 cm. The soil gravel interface is in the depth 95 to 100 cm in this locality. Soil hydrolimits are quoted in tab.1. Fig 1 presents the regime of integral water content during the monitored period along with the regime of integral water content corresponding to the individual hydrolimits (FC, PDA a WP).

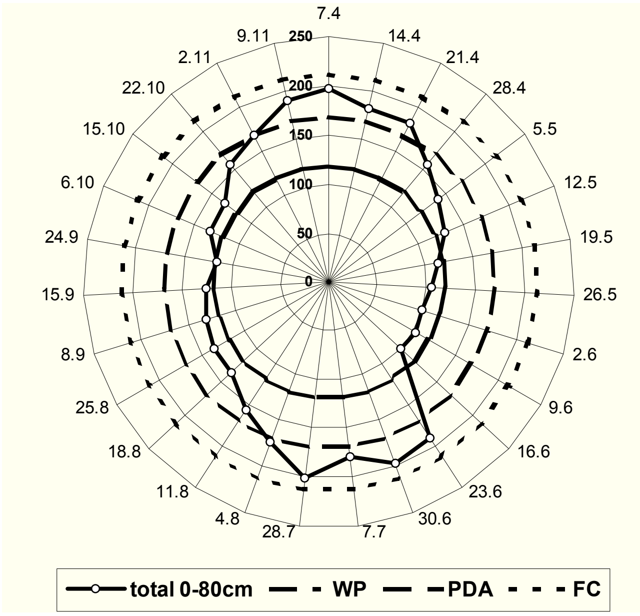

Fig.1 Total soil water content of the upper 0.80 m soil layer measured on site Lehnice (Field capacity – FC, the soil water content defining limited availability for plants – PDA (corresponding to p. F 3.3), and the wilting point – WP).

The structure of the chosen radial graphs allows for visualisation of the regime of the soil water content with the view of the quoted hydrolomits, i.e. with the view of water supply in the canopy. The centre of the concentric circles corresponds to zero water content and the individual circles correspond to the level of the water content as expressed in mm of water column. Radius vectors depart radially from the centre to the extreme on the circumpherential circle where their point of intersection is marked by the date of water content measuring. The

Tab.1 Soil hydrolimits

Depth	WP	PDA	FC
[cm]	[$cm^3.cm^{-3}$]	[$cm^3.cm^{-3}$]	[$cm^3.cm^{-3}$]
0-40	19,6	23,3	28,8
41-80	15,8	18,9	24,2

number of radius vectors equals the number of gauging in 1994. The monitored water values corresponding to the date of measuring along with the values, which correspond to soil hydrolimits, are inserted in this structure.

The above mentioned figure suggests that water supply in the soil aeration zone for the time from January to the third decade of March oscillates within the interval between FC and PDA. Water supply in this time interval documents the readiness for the start of vegetation period. Starting from this term until mid-May water supply drops as low as to the WP limit. The WP point announces the start of soil drought. It is then indicator of soil drought for the particular locality. It is followed by decrease of water supply below BV where it stays until mid-June. This period is characterised by the deficit of water necessary for the development of canopy. Decrease of water supply below the BV point is the threshold value for the soil drought. Its impact depends on adaptability of the canopy, duration of this period, and the degree of water supply decrease below BV. Irrigation and rainfall from the mid-June to the first August week optimise water supply and its values rise into the interval between FC and PDA. They drop again to the WP limit in the following period, i.e. from the beginning of August. The autumn rainfall makes them rise to reach the PDA values by the beginning of November. This trend continues until they reach the FC value before the end of year.

Assessment of the start of soil drought relying on water content monitoring in soil aeration zone is comparatively laborious and financially demanding. A somewhat more accessible method is based on the numerical simulation by means of mathematical models of soil water regime (Šimunek et al., 1997: Novák et al., 1998). The second methodological procedure concerns clay-loam soils (hereafter only "heavy soils"). The relatively long inter-precipitation period creates conditions favouring extreme drying of soil profile in case of heavy soils. As the heavy soils shrink with the decreasing water content in the above-described conditions, when moisture reaches certain value, the formation of cracks starts. The longer this period, the larger and deeper cracks.

The start of crack formation is the first symptom of the onset of soil drought.

Soil shrinkage also depends, apart from the water content, on the content of particles smaller than 2 μm. In our case, i.e. in extreme decrease of moisture the origin of cracks moves into the area with comparatively lower content of particles smaller than 2 μm. It means that the cracks in heavy soils appear where they were not observable before. The typical example were the soils of the Eastern Slovakian Lowland in 2000 when in spring extreme dry period the occurrence of cracks was observed in soils containing 37 % of particles smaller than 2 μm.

Soil shrinkage is used for further analysis of soil drought of heavy soils of the East Slovakian Lowland. Soil shrinkage can be defined in various ways, which are the result of the relationship between the soil volume and its moisture. The most frequent form of soil shrinkage is the relation (Haines, 1923; Bronswijk, 1988) between the moisture ratio and void ratio of soil aggregates. The moisture ratio **v** and the void ratio **e** are defined as follows:

v - water volume / soil volume, (1)

e - void volume/ soil volume. (2)

The use of the moisture ratio and void ratio is related to the soil water content W and void content P. Transformation of **v** a **e** characteristics to W a P can be simply expressed by the following relations :

$$W = v / (1+ e) \qquad (3)$$

$$P = e / (1+ e) \qquad (4)$$

The basic form of expression of shrinkage of heavy soils is quoted in its general form in fig.1

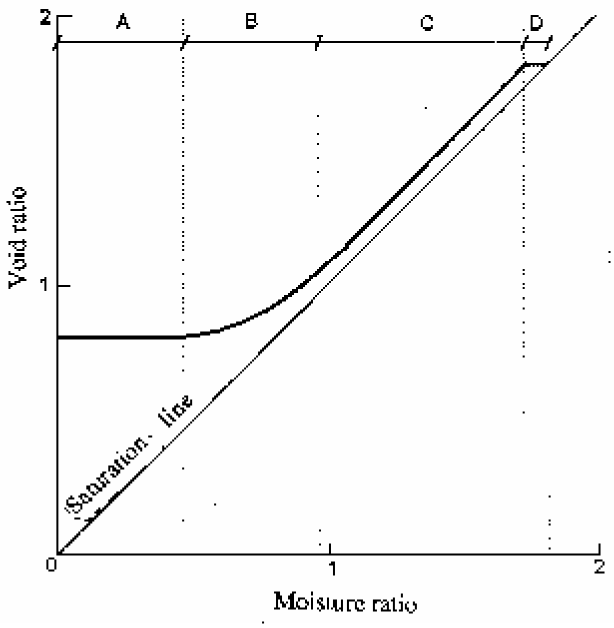

Fig.2 The general form of expression of the basic characteristic of soil shrinkage of heavy soils (The shrinkage stages: A-zero, B-rezidual, C-normal, D-texture).

Fig. 2 suggests that there exist four shrinkage stages of heavy soil (Haines, 1923):
(1) Normal shrinkage. The decrease of the volume of clay-loam aggregates. The volume of clay aggregates decreases directly proportionally to the volume of drained water.
(2) Residual shrinkage The volume of aggregates is constantly diminishing but the water loss is greater than the change of the volume. Air penetrates into the pores of aggregates and causes the origin of unsaturated zone.
(3) Zero shrinkage. Soil particles reached configuration of its volume weight. The water loss equals the volume increase of the air content in the pores of aggregates.
(4) Texture shrinkage
Transition from the normal to residual shrinkage (see fig. 1) is accompanied by the start of crack formation and announces extreme drying of heavy soils. Transition area from the residual shrinkage to zero shrinkage in the first stage corresponds to water content of the wilting point. Indication of soil drought of heavy soils in its first stage is the moisture ratio when crack formation starts and the moisture ratio for the wilting point characterises the second stage.
Basic characteristics of shrinkage (Šútor-Gomboš, 2000) were obtained for heavy soils of selected localities in the East Slovakia Lowland **(Fig.3)** for the interval of the content of particles smaller than 2 μm determined by the values < 26 ;63 > /%/. The graphic form of processing of the type **(Fig. 3)** makes possible to use them as indicators of soil drought for heavy soils of the East Slovakian Lowland. Basic characteristic of heavy soil shrinkage on the East Slovakian Lowland.

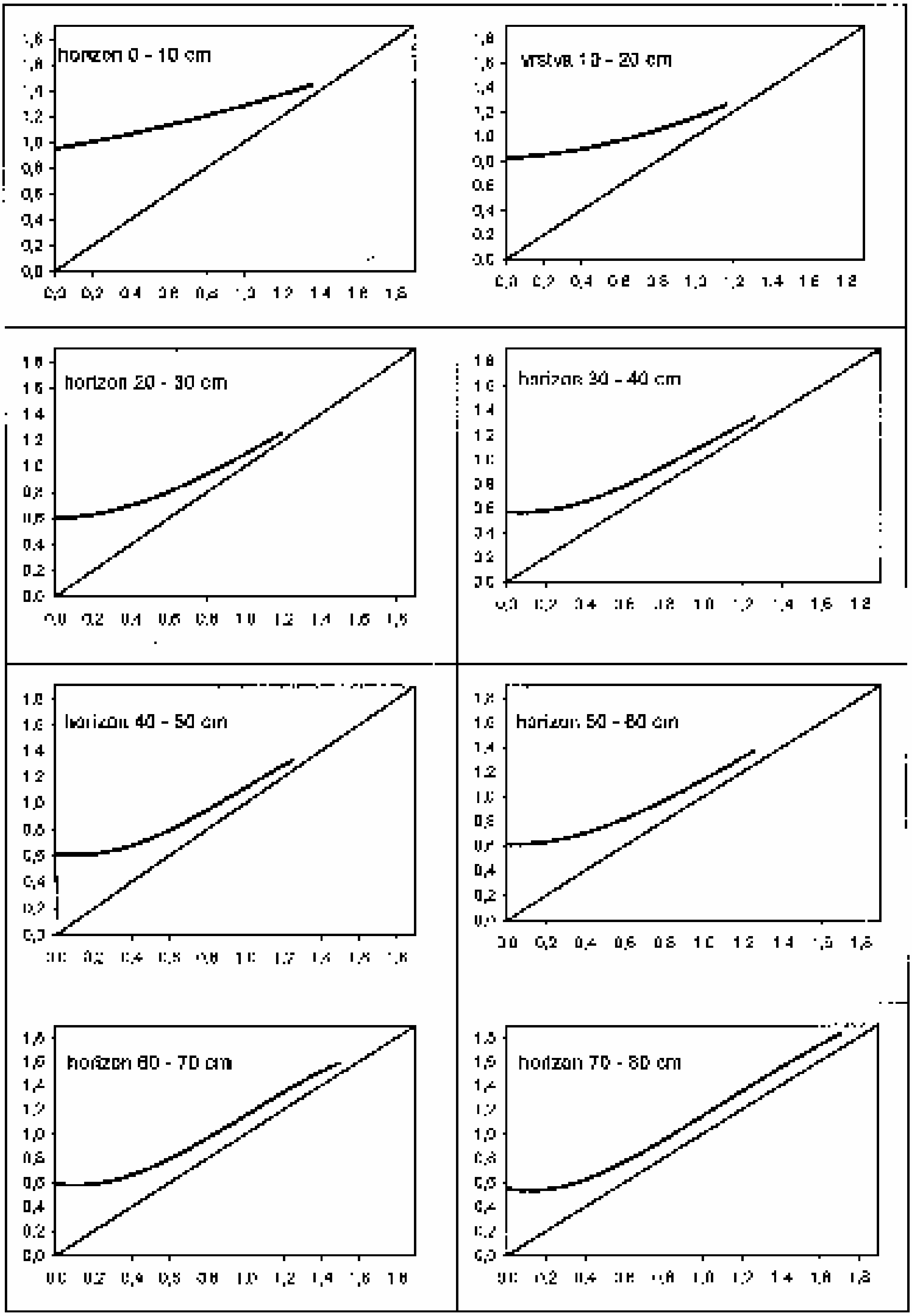

Fig.3 Basic characteristic of heavy soil shrinkage on the East Slovakian Lowland.

Conclusion

Analysis of the monitored water content regime in the soil aeration zone for the Žitný ostrov region documents the possibility to use the integral water contents in soil corresponding to the wilting point, as indicator of the start of soil drought. Determination of water soil regime for a selected thickness of soil aeration zone is laborious and financially demanding. Numerical simulation on mathematical models of soil water regime seems to be more an advantageous method of determination of integral regimes of water content in soil aeration zone. Indicators of soil drought of heavy soils are analysed making use of basic characteristics of soil shrinkage. The first stage of soil drought starts with crack formation and the second stage corresponds to the moisture ratio of wilting point. Their application as soil drought indicators is facilitated by the basic shrinkage characteristics of heavy soils of the East Slovakian Lowland assessed by experimental research.

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Some Environmental Aspects of Civil Engineering

Lidija Tadić
Faculty of Civil Engineering, Crkvena 21, 31 000 Osijek, Croatia, E-mail:ltadic@most.gfos.hr
Zdenko Tadić,
"hidroing" Križanićev trg 3, 31 000 Osijek, Croatia, E-mail:zdenko.tadic@os.htnet.hr

ABSTRACT

Last decade environmental aspects of civil engineering started to be very important. State of environment became serious and some actions on the global level were undertaken. Proposed action plans and directives had main purpose to slow down or even stop uncontrolled deterioration of environment in general. It means that our way of life and development must be in favour of future generations.
Civil engineering, together with physical planning and architecture can play significant role. In the field of road construction, industrial and residential buildings, materials, process of construction itself and especially in hydrotechnics, engineers must be aware that their constructions last a long period of time, most of them cause change of land use and urbanisation. Some of the impacts appear many years after construction was finished; some of them are indirect or cumulative. Some of the consequences of our structures are beneficial, but they are still have impacts on the environment. The range of considered environmental impacts includes artificial components of the environment as well as natural. So, we can speak about aesthetic, cultural, social or historic impact categories. It means that evaluations of potential impacts necessarily need to be interdisciplinary.
This paper is going to present overview of legislation and practise in the field of environmental aspects of civil engineering in Croatia. Besides, there are a lot of problems in implementation of protection measures. Some of them are social and economic, some of them are related to the lack of communication and lack of knowledge about some processes. Successful and applicable environmental protection must be based upon interdisciplinary communication and co-operation.

Key words: environment, impacts, civil engineering, interdisciplinary approach

1. INTRODUCTION

Environmental protection became part of all segments of life during last decade. It does not mean that it was not known in the past. Some aspects of environmental protection were applied many centuries ago, consciously or not. For example, constructors of roads and aqueducts built them to suit the landscape by usage of materials that suit local circumstances. Soil conservation was practised, too.
There is no doubt that all construction activities have an environmental impacts on natural resources, directly, on the occupied space, and indirectly, as social and economic consequences of their implementation. Extension of these impacts can range from very complex to minimal (3).
Environment is so precious to the humans because it serves three types of life functions:

1. General life support –soil, air and water as a basic elements that give us food, forest products and biomass fuel. Wider consideration of life support functions include living space, climate, biodiversity and natural beauty
2. Supply of raw materials and energy – construction materials, raw or basis for manufactured products essential for construction , minerals
3. The "sink" function – absorption in soil, vegetation, air and water of the waste products from natural processes and human activities

Some of the elements, like soil and water, serve all three types of function. It makes these resources so fragile or vulnerable (1).

All possible impacts on environment can be divided into two groups:

1. Impacts on land resources or physical environment – climate, geology, soil, hydrology, landforms and natural habitat. Again, water has a special place and its protection, in qualitative and quantitative sense, becomes one of the imperatives of development. Besides, water structures, dams and reservoirs, are one the most controversial construction, considering the environmental protection. In one hand their benefits are huge (water management – risk control on floods and droughts, water supply, energy, fishery, tourism), in the other hand environmental impacts are also serious (loss of property, historical and natural heritage, land use change, sometimes displacement of people and change of biological composition).
2. Impacts on cultural resources (landscape and heritage) – constructions may cause change in natural and man-made scenery. They may cause obstruction in landscape by visual impact of structure, by usage of inappropriate material, or by inappropriate function

Construction projects are planned to benefit group of people and improve their quality of life. But, quality of human life is a subjective category – some people feel the benefit of improvement, some people, the same construction feel as a diminishing of the quality of their lives. So it is important to be aware of the gains and losses during the process of planning.
From this brief discussion, it is obvious that constructions and the environment have mutual and complex relationship. In between these two items are economic welfare, social acceptability and legislation frame. The approach which put together all of these key items is concept of sustainability. Most of the countries accepted this concept as only way of development that ensures preservation of natural resources, environmental protection and satisfaction of human needs. Implementation of this concept is comprehensive job which must be involved in all segments of life.
Talking about sustainability of construction projects there are three groups of people that share responsibility for its successful implementation - stakeholders (owners, investors, developers, voters or bodies of public opinion in general), professionals (planners, constructors and other professionals) and regulatory authorities who set environmental and other criteria and may approve and disapprove implementation (2).
This complex process of an achievement effective environmental protection will be discussed on the Croatian example.

2. ENVIRONMENTAL PROTECTION IN CROATIA

Environmental protection became very important issue in the process of planning and long term development in Croatia, as far as in other European countries. According to National Strategy on Environmental Protection (2002), it is important to increase a necessity environmental protection and convert it to clear, comprehensive and long term concept. Two issues are crucial for development of the Strategy (8):

1. **Concept of sustainable development** – environmental protection must be harmonised with economic and social development and vice versa- economic and social development can be obtained only by including environmental protection. It means that environment becomes one of dimensions of economic and social development, included in all projects: road construction, energy production, agriculture, tourism, physical planning.

2. **Process of becoming member of the European Union** – implementation of European legislation and criteria in the field of environmental protection

There are two basic groups of problems or priorities.

Table 1. List of priorities in environmental protection

THE FIRST GROUP OF PRIORITIES	THE SECOND GROUP OF PRIORITIES
1. waste	1. chemicals
2. water	2. climate change
3. air	3. risk management-industrial accidents
4.Adriatic Sea, islands and coast	4. risk management- nuclear accidents and radiation
5. soil	5. genetically modified organisms
6. nature and biological diversity	
7. urban area-land use	

The most of the problems belong to the fields in which civil engineering, together with legislation, economy, education etc. can make a significant improvement. Comprehensive and interdisciplinary approach is needed

2.1. STATE OF THE ENVIRONMENT

Part of the Strategy is The Report on the State of the Environment in Republic of Croatia. It presents the most serious problems related to the environment and possible solutions or recommendations for their solutions for future managing and protecting. In this moment state of the environment is not alert, but in some segments of environment there is a tendency of deterioration of its quality. The another problem is lack of monitoring, so real state of the environment is unknown.

2.1.1. LAND USE

Density of population is not uniform over the country, as far as economic development. Urban density depends on economic development and transportation possibilities. Land use change, enlargement of urban area, including infrastructure, causes losses of agricultural land, deforestation, change in run off conditions, increasing of solid waste and waste water generation. All of these issues are closely related to the environment.

Land use structure is especially sensitive in carst region (along the Coast), which is very vulnerable to the human impacts, and at the same time very interesting for the investments. Very often, it results with inadequate selection of location for the buildings, hotels and other maritime structures.

Solution can be in better physical planning. According to Report on the State of Physical Planning it is necessary to develop smaller cities (decentralisation), better plan of residential areas and increase efficiency of transportation. Besides, industrial development must be situated near resources. Rural areas need a serious revitalisation, economic and infrastructural development (7).

In further actions legislation will be much more strict about uncontrolled constructions and environmental devastation.

2.1.2. WASTE

The worse situation is in waste disposal and it is far below EU standards. The amount of waste is increasing continuously without reliable data about quantities. Inadequate disposals endanger all natural resources – air, soil and water, human health and nature in general. Land filling is practically the only solution for solid waste disposal. That is the reason why is waste the first of the priorities in the National Environmental Action Plan.

Organised municipal waste collection covers less than 60 % of the population. About 98 % of total disposed waste ends up in 160 official dumping sites which, with rare exceptions, have not been adequately engineered and have no basic protective measures. Only seven landfills have been granted operating licences. Special problem is hazardous waste. In about 80 dumping sites it is disposed together wit municipal waste, causing serious environmental pollution.

Portion of municipal waste which is sorted as a secondary raw material is only 11 % and level of recycling is very low.

Activities that must be promote are sorted by hierarchy: minimisation of waste generation, waste sorting and composting in order to recycle them and prepare for reuse, burning for the generation of energy and in the end of desirable solutions are landfills.

2.1.3. SOIL

Arable land covers approximately one half of the total area of Republic of the Croatia. It is potentially, one the major source of soil pollution. Current general soil contamination levels are still satisfactory. In the most of the counties medium concentration of cobalt, zinc, manganese, and iron is bellow limit values for agricultural land, and medium concentration of arsine and copper exceeds moderately high values. The main problem is lack of regular and permanent monitoring of soil.

There can not be neglected contribution of major highways to soil contamination with heavy metals.

The other type of soil deterioration, like erosion by water is not very significant. It can occur on the slopes of the hilly regions which are mostly covered by forests.

The situation in soil pollution is still under control but it does not mean that activities like introducing the environmentally acceptable production of food and more efficient soil conservation methods are not needed.

2.1.4. INLAND WATERS

The water resources are relatively rich in quantity but its quality level is lower than it is desired. The amount of potable water is relatively large and 85 % of water for water supply systems comes from groundwater reserves. All population is still did not connected to the public water supply system, only 73 %. According to development programs all population should be connected to the public water supply system
The water quality and quantity are monitored through the nation-wide monitoring network.
Community water policy requires a transparent, effective and coherent legislative. In order to achieve this requirements, it was necessary to establish overall framework for action. Water Framework Directive of EU should provide such a framework to co-ordinate, integrate and further develop the overall principles and structures for protection and sustainable water use (5).

2.1.4.1. Waste Water

About 60 % of population , mainly in cities, are connected to the sewage system. The typical sewage system is a combined drainage system. Only 21 % of waste water is treated, even the largest cities have no treatment of waste water. Over that, 81 % is mechanically pre-treated, about 6 % is biologically treated, and 13 % of water used in industry is pre-treated. It is also very important problem in the preservation of inland water.
Problem of waste water disposal is also very serious along the Adriatic Coast- densely populated area with high peaks in discharge of waste water during summer months.
In the following years, construction of sewage system in some 70 towns and some 20 waste water treatment plants, will have to be carried out. It will make a significant improvement on environment, soil and water.
Project of the national importance, which is already in progress, Integrative Protection of Kaštel Bay at the Adriatic Coast, will solve the problem of wastewater of densely populated area around Split, the biggest town at the Adriatic Coast.

2.1.4.2. Extreme Hydrological Phenomena

Last years extreme hydrological phenomena- floods and droughts became very frequent, due to the global warming of the Earth. More than 15 % of national territory suffer from floods. The existing flood protection is only partly completed. Average annual damages from floods are presented on Figure 1.

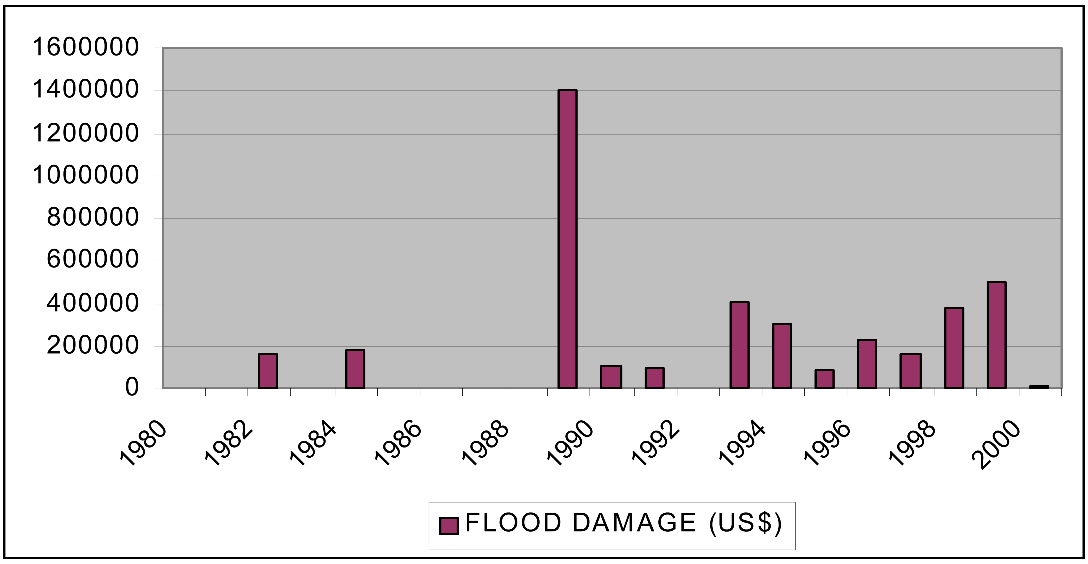


Figure1. Damages to the floods in the period 1980-2000 (State of the Environment Report-Republic of Croatia, 2001)

Besides, damages in agriculture due to droughts are even higher (Figure 2).

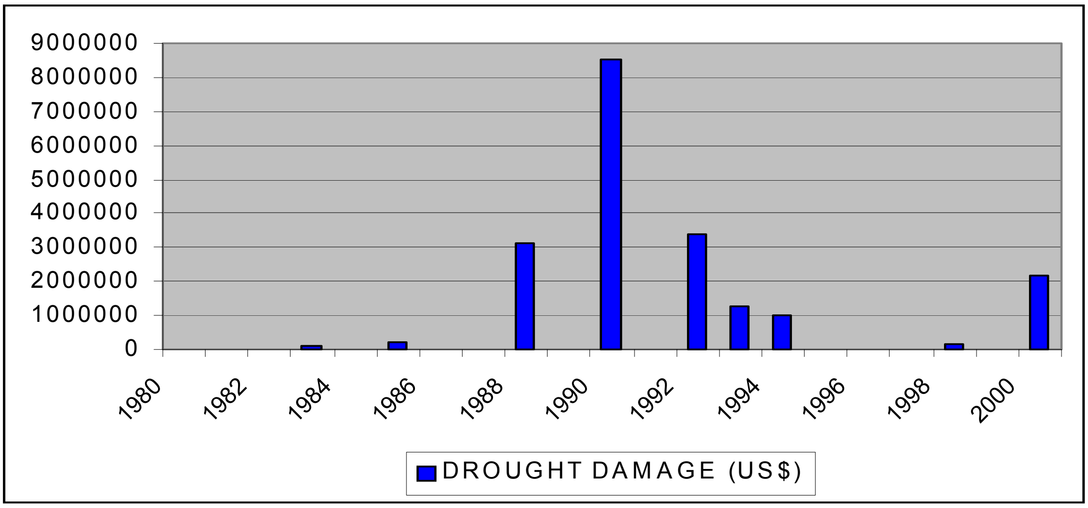
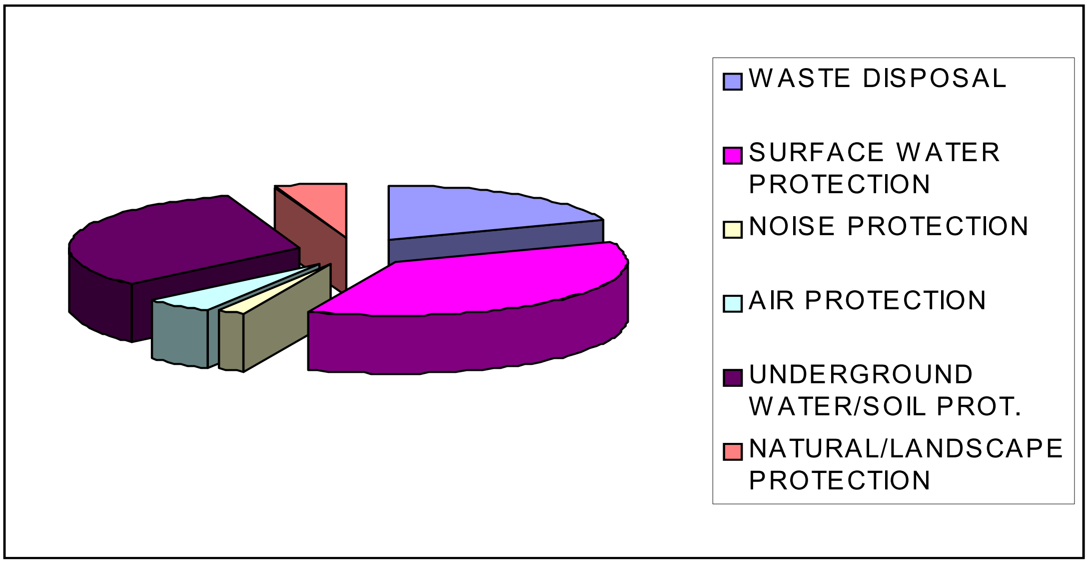


Figure 2. Damages to the droughts in the period 1980-2000 (State of the Environment Report-Republic of Croatia, 2001)

Practically there are no systematic irrigation systems (less than 1% of agricultural land). Average annual damages from natural disasters recorded in Croatia in the period from 1980-2000 are over 250 million of US$, 38 % of that sum refers to damages caused by droughts, and only 7 % are cased by floods. It means that water management needs serious improvement. Such a high frequency of floods and droughts can not be considered as a sustainable water management. Solutions are primarily in construction of dams and reservoirs.
Above mentioned problems and priorities are followed by protection of cultural heritage, project of increasing of efficiency in energy supply and improvement of legislation and its co-ordination with European legislation.

2.2. FINANCING AND LEGISLATION

Environmental investments in Croatia were about 0.22% of GDP per annum in the period from 1997-2000. Such low level of investments is not sufficient for the fulfilment of the existing legal obligations, while these amounts will have to increase multi-fold to comply with the EU standards. The majority of investments have been recorded in the protection of surface and ground waters (69%), and waste (19%), Figure 3.
Besides the national budget, other resources of financing comes from Croatian Waters (concession fees, ownership fees), international financing institutions and bodies (World Bank, EU programmes, UNDP), financing by loans of international financing institutions (International Bank for Reconstruction and Development, European Investment Bank).
Part of these funds has been using for development of legislation. Some environmental sectors have quite good regulatory basis (considering surface water and ground water), but some of them still need to be complied with European



Figire 3. Investments into the environmental protection per sectors in the period 1997-2000 (State of the Environment Report-Republic of Croatia, 2001)

legislation. For example, it is in the process of preparation law which encourage sustainable building. According to this law, if the building methods are friendly to the environment (considering usage of recycled material, renewable resources of energy, healthy way of production in industry, treatment of waste water, reuse of water etc.) that requested by law or common practice, investor will be entitled to reduction or remission of administrative taxes.

3. CONCLUSION - CHALLENGES FOR THE FUTURE

Previous discussion can be summarised through the few general recommendations and propositions for the future, in order to make constructions more sustainable(2).

1. Identification of regional issues and opportunities which make different approaches to development and construction appropriate to different circumstances
2. Construction development to suit site conditions, employment patterns and skills and available construction materials
3. Rational use of land with preservation of natural heritage and space for human amenity

4. Controlling and improving the water resources in the quality and quantity
5. Preparing design of structures to maximise quality in terms of practicable performance, social benefit, aesthetic appearance and longevity
6. Providing engineering support to implementation options for energy production, disposal and reuse of wastes
7. Improving the methods of assessing and accounting for risk in project cost and benefit estimates, to compare alternative options and recommend the most satisfactory solution in the long term
8. Improving the monitoring system and information technology
9. Promotion of interdisciplinary approach
10. Promotion of sustainability through the legislation

These general recommendations must be applied according to regional, natural, economic and social circumstances of each country, following its own list of priorities.
Besides, fulfilment of these statements is possible only by interdisciplinary and comprehensive approach.

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Water Losses in Water Distribution System

Katarína Tóthová

Slovak University of Technology, Faculty of Civil Engineering, Department of Sanitary Engineering, Radlinského 11, 813 68 Bratislava Slovak Republic, E-mail: tothova@svf.stuba.sk

Abstract

Recent social and economic changes forced Slovak water companies to pay attention to problems concerning water losses in distribution systems. The quality of water lost is important indicator of the positive evolution of water distribution efficiency, both in individual years and as trend over a period of years. High and increasing annual volumes of water losses, which are an indicator of ineffective planning and construction, and low operational maintenance activities, should be the triggers for initiating an active leakage programme. A leak-free network is not realisable technical or economic objective, and a low level of water losses cannot be avoided, even in the best operated and maintained systems, where water suppliers pay a lot of attention to water loss control. In the Slovak Republic volume of water losses in distributions systems represent in average 23 % of produced drinking water. The objectives of this paper will summarise the terminology of annual water balance, recommend how the annual volume of real and apparent losses should be calculated and recommend the most appropriate Performance Indicators for international use.

1 Introduction

The actual quantity of water lost from a water distribution system will vary from utility to utility depending upon local factors such as topography, length of mains, number of connections and standards of service, and upon how well the system is being operated and maintained. The problem of losses are

- technical (not all the water supplied by a water utility reaches the customer)
- financial and economic (not all the water supplied is paid for)
- terminology (lack of standardised definitions of water losses)

The annual volume losses consist of two separate types of losses - real (physical) and apparent (non-physical).

2 Standard definition for international use

Any discussion relating to losses must be preceded by a clear definition of the water balance components. However, there are significant differences in the definitions used in different countries. Unnecessary misunderstandings arise because of differences in the definitions used by individual countries for describing and calculation losses. Also, traditional performance indicators often given conflicting impressions of true performance in controlling water losses. In 1996 the Operation and Maintenance Committee of the IWA's Distribution Division set up a Task Force to review existing methodologies for international comparisons of water losses from water supply systems. It had two key objectives

- to recommend a standard international terminology for calculation of real and apparent losses using a annual water balance
- to review Performance Indicators (PI) for international comparisons of water losses and recommend preferred PIs

The IWA standard international water balance (Fig.1) combines elements of best practice from different countries. For countries that already have a published national standard terminology it is usually quite easy to identify and transpose the components. For countries and undertakings that do not yet use a standard terminology (also Slovak Republic), or are contemplating improvements to an existing methodology, the IWA standard water balance is a most useful 'best practice' guide to follow. All components are initially expressed as a volume per year.

<table>
<tr><td rowspan="9">System input volume [m^3/rok]</td><td rowspan="4">Authorised Consumption</td><td rowspan="2">Billed Authorised Consumption

[m^3/year]</td><td>Billed Metered Consumption (incl. water export</td><td rowspan="2">Revenue Water</td></tr>
<tr><td>Billed Unmetered Consumption</td></tr>
<tr><td rowspan="2">Unbilled Authorised Consumption

[m^3/year]</td><td>Unbilled Metered Consumption</td><td rowspan="7">Non - revenue Water</td></tr>
<tr><td>Unbilled Unmetered Consumption</td></tr>
<tr><td rowspan="5">Water Losses</td><td rowspan="2">Appearent Losses [m^3/year]</td><td>Unauthorised Consumption</td></tr>
<tr><td>Customer Metering Inaccuracies</td></tr>
<tr><td rowspan="3">Real Losses [m^3/year]</td><td>Leakage and Overflows at the Storage Tanks</td></tr>
<tr><td>Leakage on Transmission and/or Distribution Mains</td></tr>
<tr><td>Leakage on Service Connections up to point of Customers metering</td></tr>
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Fig. 1: Annual Water Balance [m^3/year]

2.1 System input volume is the volume of water input to a transmission system or a distribution system

2.2 Authorised Consumption is the volume of metered and/or unmetered water taken by registered customers the water supplier for domestic, commercial and industrial purposes. It includes water exported. Authorised consumption includes items such as fire fighting and training, flushing of mains or sewers, street cleaning, public fountains, frost protection. These may be billed or unbilled, metered or unmetered according to local practice.

2.3 Water Losses consist of real and apparent losses, and are not identical to the previous used term unaccounted-for water (UFW). It is the most frequency error and mistake in water losses terminology.

Apparent losses consist of unauthorised consumption (illegal use), and all types of inaccuracies associated with production metering and customer metering.

Real losses are physical water losses from water system up to the point of customer metering. The volume lost through all types of leaks, bursts and overflows. Components of real losses are:

- background losses from very small undetectable leaks (low flow rate, long duration, large volume)
- losses from leaks and bursts reported to the water supplier (high flow rate, short duration, moderate volume)
- losses from unreported bursts, found by active leakage control (medium flow rate, duration and volume depends on leakage control)

For each system, there are several key local influences, which constrain the possibilities for managing real water losses, and which need to be recognised when selecting Performance Indicators to access the effectiveness of managing real losses:

- the number of service connections
- the location of customer meter an the service connection

- the length of mains
- the average operating pressure
- infrastructure condition, materials, frequencies of leaks and bursts
- type of soil and ground conditions, insofar as they influence the proportion of leaks and bursts which show quickly at the ground surface.

The greatest proportion of the annual volume of real losses occurs from leaks and bursts an the service connections.

The difference between the system input and all billed water (only a part of the authorised water) is <u>*non-revenue water*</u>. It should be defined in the same way as UFW.

3 Water losses management

What strategies are needed today for a comprehensive water loss control programme? Some utilities only initiate repairs from reported leaks that surface above ground. More progressive utilities have undertaken pressure management, and active leakage control programmes to search out unreported leaks and undertake expedient repairs to minimise losses. But how low can water losses be reduced? These issues are currently under debate at the IWA Water Loss Task Force. These initiatives include pressure management, active leakage control, speed and quality of repairs and pipe materials management as shows the figure 2.

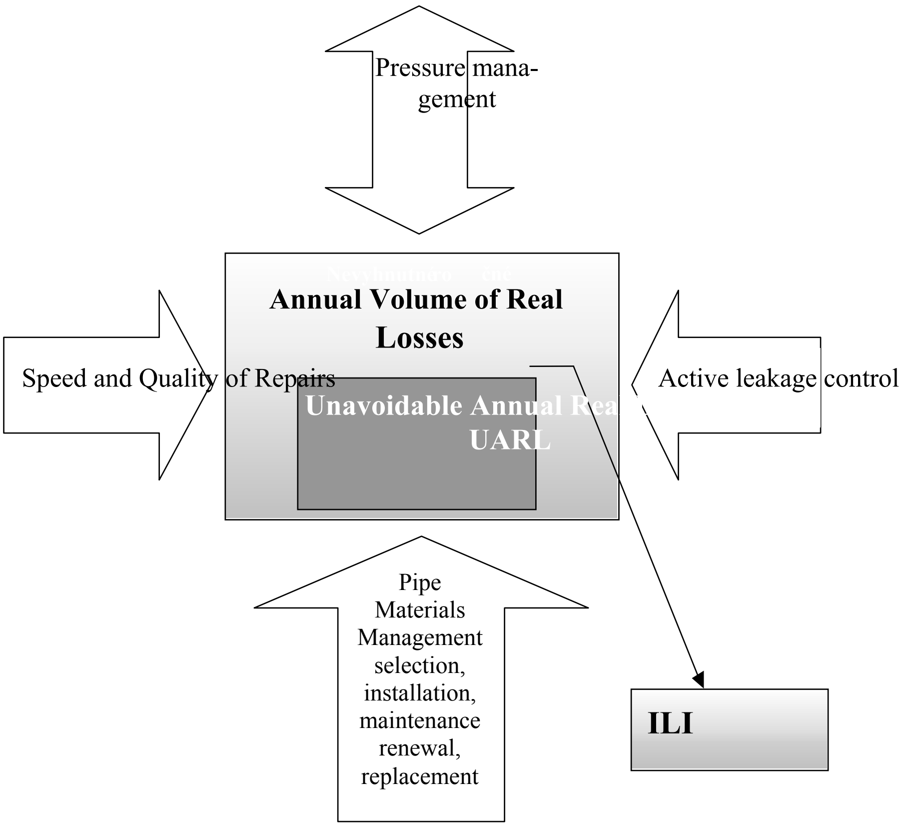

Fig. 2 The four basic leakage management activities that constrain annual real losses

4 Technical performance indicators

Traditional performances indicators (PI) for losses which are most widely used in different parts of the world to make comparisons are:

- as % of input volume
- as m^3 per length of mains per day or hour
- as m^3 (or litre) per service connection per day or hour

4.1 % of Volume Input

Percentage of system input volume is an appropriate measure to define the financial views of water losses. But regarding the technical view PI as % of volume input is unsuitable for assessing the efficiency of management of distribution system. This is because Real losses expressed as % of system input are very strongly influenced by consumption. For example during last years in Slovakia water losses as % of volume input are increased, although volume in m^3 is decreased with volume of water input. The increase of percentage value does not mean that the technical condition is deteriorating and it does not mean that the water losses are going bigger.

4.2 Technical Indicator for Real Losses (TIRL)

The international experience shows that the grates proportion of losses occurs to on service connections(over 70 % of real losses). Therefore the best of traditional PIs for comparing performance in managing real losses in distribution system is ***litre/service connection/day***. It presents Technical Indicator for Real Losses (TIRL) - it should be the annual volume of real losses divided by the number of service connection.

Only for main pipes (less that 20 service connections) is appropriate to formulate water losses in units ***litre/km/day***.

4.3 Unavoidable Annual Real Losses (UARL)

A more detailed interpretation of TIRL values can then be obtained by comparing the TIRL with a 'best estimate' of Unavoidable Annual Real Losses (UARL) which allows for local conditions of connection density, location of customer meters and average operating pressure, if all aspects of leakage control were being managed to the highest technical standards. The best unit of UARL is ***litre/service connection/day***

The calculation of UARL presents Lambert [1]. The figure 3 shows the resulting value of components of UARL.

infrastructure components	background leakage	reported bursts	unreported bursts	UARL total	units
mains	9,6	5,8	2,6	18	litre/ km mains/ day / metre of pressure
service connections to edge of street	0,6	0,04	0,16	0,8	litre/ service connection/ day / metre of pressure
underground pipes, edge of street to customer meters	16	1,9	7,1	25	litre/ km underg. pipe/ day / metre of pressure

Fig. 3: Components of Unavoidable Annual Real Losses (UARL)

4.3 Infrastructure Leakage Index (ILI)

In figure 6, the large central square represents the current annual real losses which tend to naturally increase with time. The current annual real losses volume is constrained by the four leakage management activities shown. The lowest possible achievable annual volume of real

losses is the UARL (the central smaller square). The ratio of the current annual real losses to the UARL is the infrastructure leakage index ILI. The ILI is a non-dimensional PI, which allows overall infrastructure management performance in control of real losses to be assessed. Values of ILI calculated for 27 actual situations in 20 countries, which were used to test the validity of the methodology, ranged from 0.8 up to just above 10.0. Well-managed systems in very good condition would be expected to have ILI value close to 1.0, with higher values for older systems with infrastructure deficiencies.
The ILI is a measure of the overall effectiveness, at current operating pressures, of management of the infrastructure to control real losses, through pipe materials management, speed and quality of repairs and active leakage control.

5 Conclusions

In well-operated system, water losses should be continuously monitored and controlled, and noted each year in an annual report.
Some of the conclusions of the IWA Task Force on Water Losses represent quite a radical departure from traditional practices. However, these traditional practices have meant that:
- there has been no standard terminology for water balance calculations
- water undertakers have been able to choose to use any of several performance indicators which gave conflicting impressions of performance in managing real losses
- %s have continued to be widely used as a primary PI for management of real losses, despite national technical committees in Germany, UK, USA, South Africa and the economic regulator in UK having drawn attention to the undue influence of consumption on this PI on many occasions over the last 20 years

There is now a new, standard and challenging methodology which water undertakers can use to evaluate performance in management of real losses in transmission and distribution systems. The approach can be used for comparisons between supply systems with diverse characteristics:
- within a single large water undertaking
- within the same country
- within different countries

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Hydrologic Analysis of Low Streamflow of the Sava River near Županja

Dušan Trninić and Tomislava Bošnjak

Meteorological and Hydrological Service, Grič 3, 10000 Zagreb, Croatia, **trninic@cirus.dhz.hr** and bosnjak@cirus.dhz.hr

Abstract

The paper reports on hydrological analysis of low streamflow of the Sava River near Županja carried out on discharge dana for period 1945-1999 (N = 55 years), and evaluates hydrological drought from 2003 (still present). Constant low streamflow periods were singled out, Δt = 10, 20, 30, 60, 90, 120 and 150 days. Interrelations were determined between average minimum discharge (ordinate) – duration (abscise) – probability (parametric presentation) as shown in Fig.1. Referenced characteristic discharges were determined along with maximum annual water deficits. Two characteristic low streamflow quantities were selected for the referenced discharges Q., i.e. $Q_{30,80\%}$ and $Q_{30,95\%}$ (minimum mean 30-day discharge, probability 80 and 95 %), which are the most frequently used values in the hydrological practice. Maximum annual water deficits were calculated for these two referenced discharges. Ten largest deficits during the period under consideration presented i Tables 1. and 2. Taking minimum mean 30-day discharge and using an equation that defines time of the low streamflow occurrence, mean occurrence time (date) was determined for so defined low streamflow. As could be concluded from the obtained results, the Sava River low streamflows, are most frequent during falls (september, October, November) and summers (June, July, August). An assessment of climate changes is highlighted and their possible impacts on the low streamflows. Conclusions are given and some measures proposed for mitigation of damages caused by the hydrological draughts ([e.g. increase in low streamflows)

1 Introduction

Problems related to the low streamflows are rather complex and affected by a number of factors, both natural (climate, hydrlogeologic, morphology, etc.) and antropological (urban development, irrigation, water works, public water supply of urban areas, water conveyance, hydroelectic, thermal and nuclear power plants, etc.). Human impact on the low streamflow regime and importance changes both in its character and in intensity. This impact coresponds with the level of social, economic and technical development, climate conditions and river hydrological regime. In practice, we have frequently encountered the need to define the low streamflow regime. The low streamflows are typical for their variability in space and time, and confirm the fact that the smaller the catchment area the greater the complexity.

The best coverage of the low streamflow issues is attributed to the autors such Bonacci (1977, 1993), Trninić (1984, 1986, 1994), Zelenhasić et.al. (1986). Two books should be singled out, Methods of Computation of Low Streamflow (UNESCO; 1982) and Antropogenie izmenenia vodnosti rek (Šiklomanov, 1979), the key concepts of which were used in the present paper.

2 Statistical Analysis of Typical Low Streamflows

The paper presents hydrological analyses of low streamflows in the Sava River near Županja based on the hydrological dana for the period 1945-2002 (N = 58 years). Statistical analysis of the low streamflow periods is carried out so that constant low streamflow periods are singled out: Δt = 10, 20, 30, 60, 90, 120 and 150 consecutive days in a calendar year. Average minimum discharges are calculated for all these periods, and minimum discharges for each period singled out along with the corresponding dates of the analyzed low streamflow period start and end. Each sigled out series with constant duration was adapted by using the most favorable distribution of calculated minimum average discharge series. The resulting theoretical values of average minimum discharges gave a relation of average minimum discharge (ordinate) – duration (abscissa) – probability (parametric presentation). The relation is shows in Fig. 1.

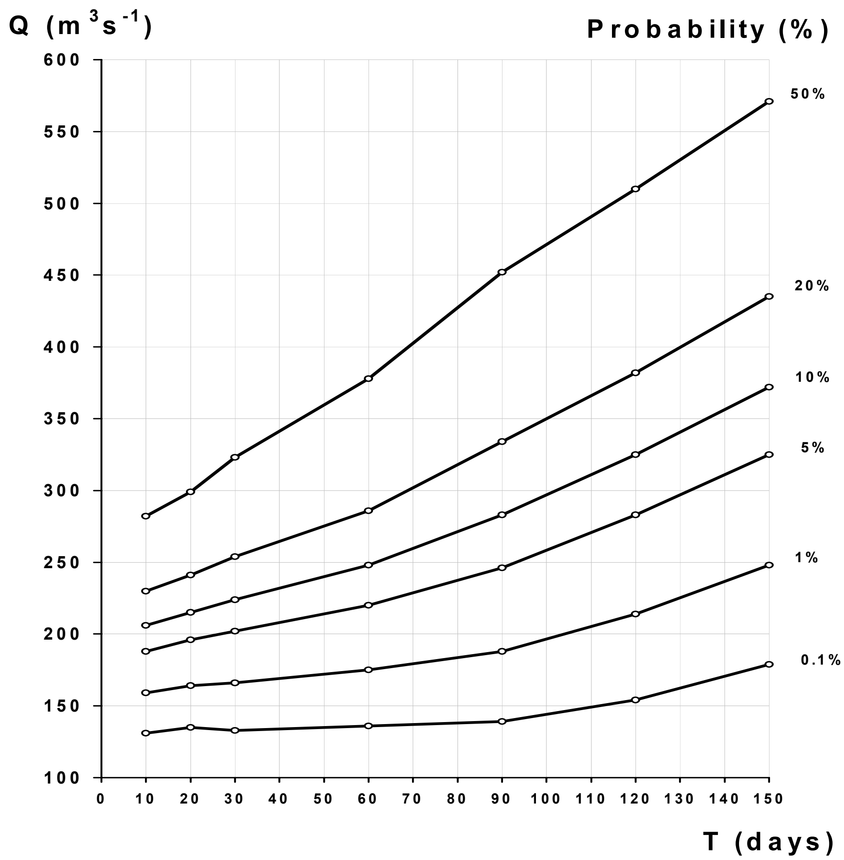

Fig. 1: Distribution of low streamflow intervals of defined duration for the Sava River near Županja for period 1945-2002

Figs. 2. and 3. illustrate trends of mean and minimal annual water levels, and mean and minimum annual discharges of the Sava near Županja for the period 1945-2002. The dana presented in Fig. 2. point to the trends of mean and minimum annual water level decrease by 1.21 and 1.12 cmm a year. The dana presented in Fig.3. show the trend of mean annual discharge decrease by 1.15 m^3/s, while practicaly no trend have been recorded for minimum annual discharges.

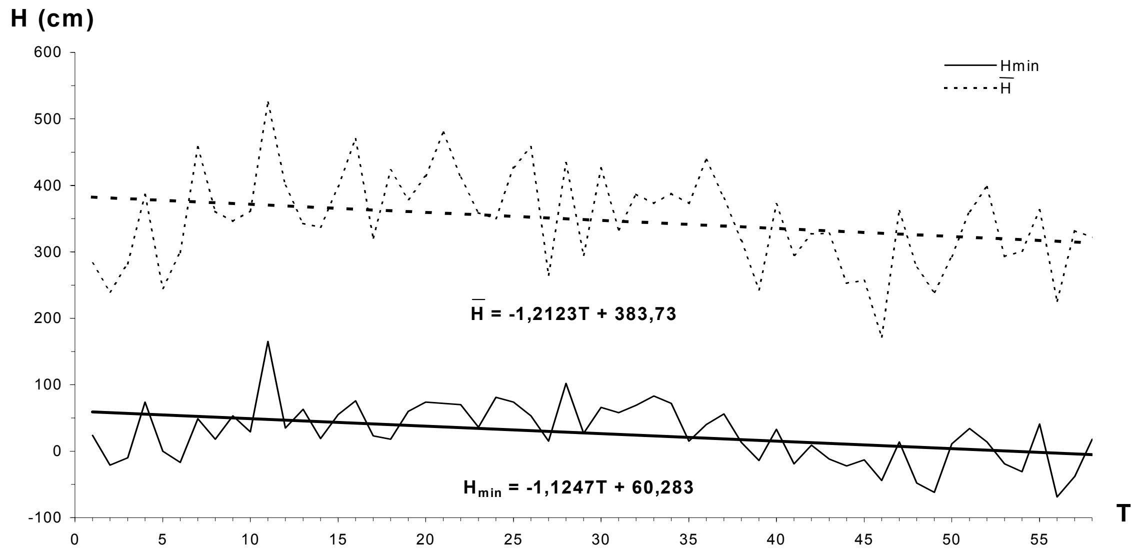

Fig. 2: Trends of the mean and minimal annual water levels of the Sava River near Županja for the period 1945-2002

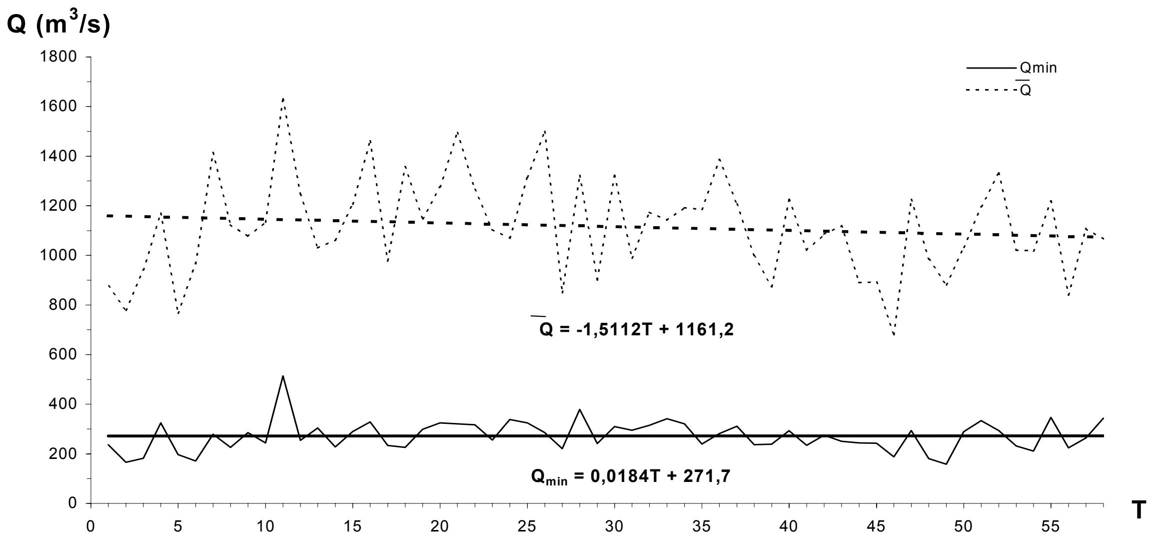

Fig. 3: Trends of the mean and minimal annual discharge of the Sava River near Županja for the period 1945-2002

3 Maximum Annual Volume Deficit

These typical low streamflows, i.e. $Q_{30,\ 95\ \%}$, $Q_{30,\ 80\ \%}$ (minimum mean 30-day discharge with 95 and 80 % probability), and $Q_{95\ \%}$ (discharge duration 95 % of an average duration curve), which are most frequently used in hydrological practice were chosen as referenced discharges Q_{REF}. Maximum annual deficits for the Sava near Županja for the period 1945-2002 were calculated for the selected reference discharges.

Table 1. shows seven (all available) maximum annual volumes less than the reference discharge $Q_{30,\ 95\ \%}$ = 202 m^3/s, and Table 2. shows ten maximum annual volumes of isolated water eaves less than referenced discharge $Q_{30,\ 80\ \%}$ = 254 m^3/s. Table 3. shows ten maximum annual volumes less than the reference discharge $Q_{95\ \%}$ = 297 m^3/s.

The conclusion drawn from the dana presented in Tables 1. 2 and 3 is than the most intensive hydrological droughts within the analyzed period (1945-2002) were recorded during the periods 1946-1950, 1990-1993, and in the years 1971 and 2000. It will be interesting to see what will be the ranking of the hydrological drought of the year 2003 which is still ongoing while this paper is being written.

Table 1 Seven maximum annual volumes isolated waves under the reference discharge Q_{REF} = 202 m^3/s of the Sava River near Županja for period 1945-2002

No.	Year	Q_{min}		Period	Volume	Duration	Dry or rainy year
		Date	m^3/s		10^6 m^3	days	
1	2	3	4	5	6	7	8
1.	1993.	25.08.93.	159	15.08-30.08.	37,84	16	very dry
2.	1946.	04.10.46.	166	28.09-14.10.	36,89	17	very dry
3.	1950.	16.09.50.	171	01.09-21.09.	26,01	21	dry
4.	1947.	25.10.47.	182	11.10-28.10.	17,88	18	dry
5.	1992.	05.10.92.	181	21.09-06.10.	14,43	16	dry
6.	1990.	04.09.90.	190	31.08-09.09.	7,344	10	very dry
7.	1949.	30.10.49.	197	30.10-02.11.	1,210	4	very dry

Table 2 Ten maximum annual volumes isolated waves under the reference discharge Q_{REF} = 254 m^3/s of the Save River near Županja for period 1945-2002

No.	Year	Q_{min}		Period	Volume	Duration	Dry or rainy year
		Date	m^3/s		10^6 m^3	days	
1	2	3	4	5	6	7	8
1.	1946.	04.10.46.	166	03.08-29.10.	391,6	88	very dry
2.	1947.	25.10.47.	182	11.09-04.11.	229,4	55	dry
3.	1950.	16.09.50.	171	15.08-22.09.	172,0	39	dry
4.	1993.	25.08.93.	159	29.07-01.09.	163,6	35	very dry
5.	1992.	05.10.92.	181	15.09-08.10.	110,1	24	dry
6.	1990.	04.09.90.	190	17.08-12.09.	100,6	27	very dry
7.	1949.	30.10.49.	197	12.10-06.11.	67,22	26	very dry
8.	2000.	28.08.00.	224	15.08-08.09.	45,27	24	very dry
9.	1971.	30.08.71.	221	12.08-31.08.	38,79	20	very dry
10.	1985.	12.10.85.	236	01.10-03.11.	35,16	34	dry

Table 3 Ten maximum annual volumes isolated waves under the reference discharge Q_{REF} = 297 m^3/s of the Save River near Županja for period 1945-2002

No.	Year	Q_{min}		Period	Volume	Duration	Dry or rainy year
		Date	m^3/s		10^6 m^3	days	
1	2	3	4	5	6	7	8
1.	1946.	04.10.46.	166	25.07-30.10.	727,0	98	very dry
2.	1950.	16.09.50.	171	03.07-26.09.	473,0	86	dry
3.	1947.	25.10.47.	182	30.08-05.11.	468,7	68	dry
4.	1992.	05.10.92.	181	06.08-08.10.	391,3	64	dry
5.	1990.	04.09.90.	190	31.07-28.09.	349,2	60	very dry
6.	1993.	25.08.93.	159	17.07-06.09.	330,6	52	very dry
7.	2000.	28.08.00.	224	27.07-04.10.	261,7	68	very dry
8.	1949.	30.10.49.	197	22.09-08.11.	240,4	48	very dry
9.	1985.	12.10.85.	236	17.09-06.11.	205,2	51	dry
10.	1971.	30.08.71.	221	08.08-02.09.	129,3	26	very dry

Table 4. shows number of low streamflow occurrences as per seasons for three referenced discharges Q_{REF} = 202, 254 and 297 m^3/s of the Sava River near Županja (period 1945-2002). A conclusion drawn from the analysis is that the low streamflows occur in summer and fall, have not been recorded during spring, while their occurrence in winter was recorded in a symbolic number of cases.

Table 4 Season of low streamflow defined for three analyzed referenced discharges of the Sava River near Županja for period 1945-2002

Q_{REF} m^3/s	**Year's seasons**			
	Winter	**Spring**	**Summer**	**Autumn**
	(XII, I, II)	**(III, IV,V)**	**(VI, VII, VIII)**	**(IX, X, XI)**
202	-	-	1	6
254	2	-	11	13
297	2	-	14	33

4 Low Streamflow Increase

One of measures for mitigation of damages caused by the hydrological drought is increase in low streamflows, both surface and underground. A concrete project for increase in low streamflows is based on construction of six multipurpose reservoirs to be buit in the upper section of the Sava catchment.

5 Climatic Changes and Low Streamflows

During the last twenty odd years the climatologists have noticed an increased occurrence of unusual climatological events with obvious consequences, such as frequent droughts, floods and the like. According to some scenarios, the most important impact of the climate changes on the hydrological cycle shall be increased frequency of extremes, such as floods and droughts, leading to increase in number and intensity of these extreme hydrological phenomena.

Previously recorded climatic and hydrological data are used in designs of numerous capital investment hydraulic engineering projects which are planned to last for 50 to 100 and more years. The climate changes shall turn these data into unrealistic background information on future conditions. Therefore, the deing engineers should account for possible.

6 Conclusions

The increasingwater demand and quality requirements ask for further investigations focusing primarily on low streamflows. The low streamflow analyses will certainly he used as a basic hydrological input by numerous water users, particularly for land-use planning, water supply of settlements and industry, power generation, agriculture, river navigation, special needs, tourist industry and the like. The Sava low streamflows have been recorded in sammer and fall, which is an important input for the water demand planning. The attention should particularly be paid to possible climate changes and their adverse impact on the hydrological cycle components. These changeshould be accounted for in designing water works and other structures.

In conclusion, it must be underscored that the issues related to the low streamflows in the open watercourses are very complex and ask for expert and thorough multidisciplinary approach with more quality and reliable measurements.

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River Beds and Stream Bank Monitoring

Jaroslav Veselý, Jana Pařílková, Zbyněk Zachoval

Brno Technical University, Department of Civil Engineering, Laborotory of Water Management Research of Institute of Water Structures, Veveří 95, 662 37 Brno, Czech Republic, vesely.j@fce.vutbr.cz, parilkova.j@fce.vutbr.cz, zachoval.z@fce.vutbr.cz

Abstract

The development of riverbed is systematically monitored for selected streams by collecting samples of bed load sediments in the long term. Their summary processing serves as basic data for management, maintenance and designing of streams.
A new, non-destructive, method for testing stability and failures of dikes and banks was been developed under grant No. 103/01/0057 and has been reliably tested under laboratory conditions. For information on their application in situ, see also e.g. technological park SINCROTRONE TRIESTE S. C. p. A. in Terst, Italy.
Solution of selected concrete cases is studied on physical models in the BUT FCE laboratory in Brno.

1 Introduction

Bed formation processes in streams and subsequent bank stabilisation by timely monitoring of disturbances and their maintenance are processes which vary in time and subject to the effects of weather in the first place.
The frequent occurrence of high waters in streams in the Czech Republic in recent years, and the need to protect human life and property, urgently draw our attention to the necessity of monitoring the watercourses in individual basins. The Laboratory for Water Management Research of the Institute of Water Structures has cooperated with Povodí Odry s.p. [Odra Basin State Enterprise] for a long time. They have been taking sediment samples since about 1952. A summary report has been compiled from this, serving not only Povodí Odry s.p. as the technical background in the management and maintenance of individual watercourses, but also as the background for planning and in the decision making in the security survey and supervision.
In the monitoring of the riverbed-formation processes in "difficult" stream sections, the use of models is warranted. A physical model is used, for example, for the planning of measures causing difficulties, and a mathematical model for obtaining a survey of the riverbed development. Here, as well as for in situ measurements, the high level of modern measurement techniques and the processor's skill are irreplaceable.
The handling of the grant project, GA ČR number 103/01/0057 "The Development of Methods for the Monitoring of Protection Dikes", resulted in the development and testing of a new method for non-destructive monitoring of banks stability and failures using electronic impedance spectroscopy.

2 Taking of samples of sediments and the modelling of problematic watercourse sections

Samples of sediments and suspended matter, of the top layer or the bottom bed have been collected at the Brno Technological University for a long time. An overall summary for the watercourses has been made with Povodí Odry, s. p., Ostrava, see, for example: Odra – the Course of Grain Size d_m and d_{90} along the Watercourse [6] (fig. 1), which includes Odra and its tributaries and the Olše and its tributaries.

We studied the meander formation on the boundary section of the Odra River for the WWF Auen, FRG [7] within an EU programme. The courses of the lowest bottom have been recorded from the viewpoint of the development from 1952 to 2000, as is evident in fig. 2.

These samples and their analyses are also important in the mathematical and physical modelling of watercourses and their structures. The classic processing is carried out by the authors' establishment in accordance with valid standards ČSN EN 932-x and ČSN EN 933-x.

The physical modelling of problematic watercourse sections and the subsequent solution has been carried out, for example, on the watercourse section under the weir in Vyšní Lhoty on the Morávka River, fig. 4 [8].

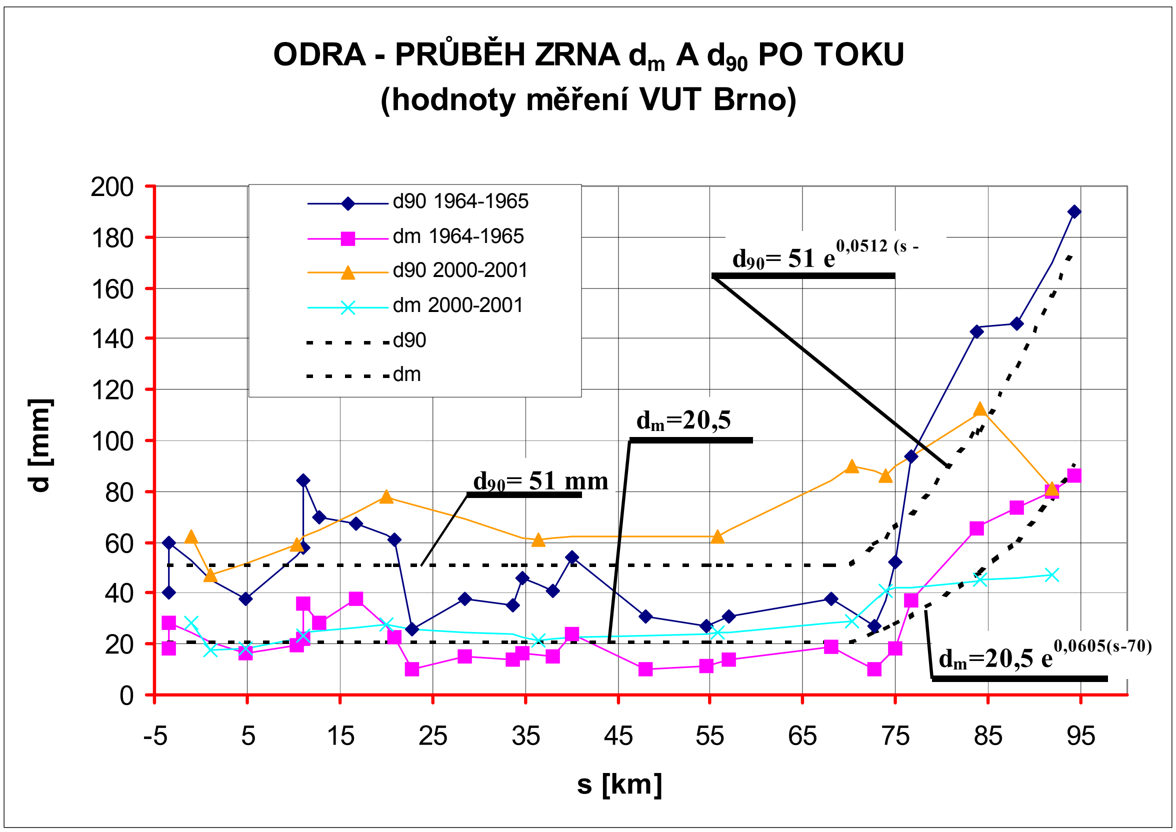

Fig. 1: Development of the grain size composition of the river bottom along the watercourse.

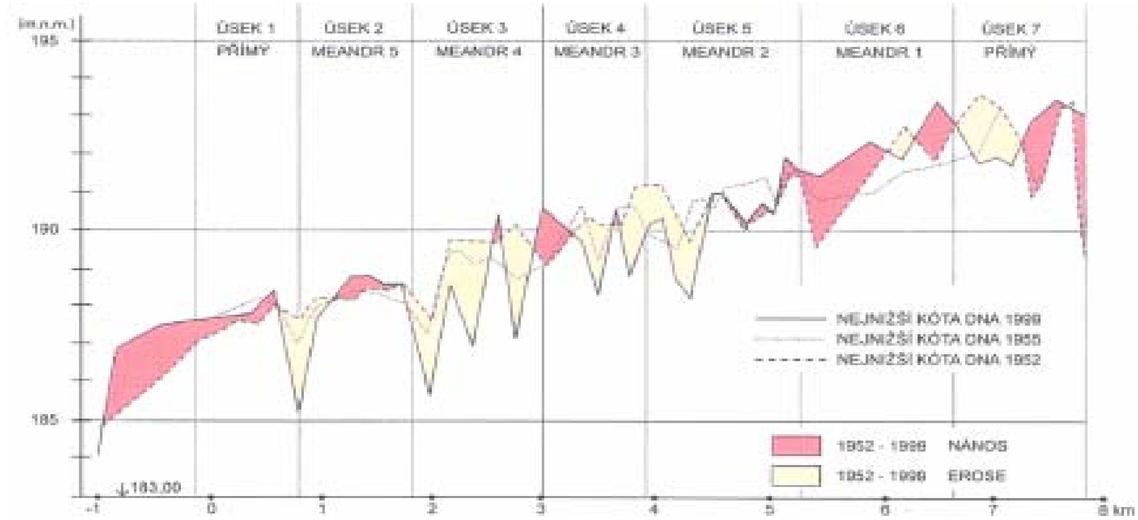

Fig. 2: Erosion and sedimentation in the boundary meanders of the Odra River, 1952-1999.

3 Mathematical modelling of water flow in river channels

The mathematical modelling of water flow in stream channels has become a necessity for the control of water flow in watercourse networks. It is very helpful in the monitoring of prognoses for water stream channel development and thus it helps in determining the optimal disposal of financial means.

Mathematical modelling is, in comparison with physical modelling, 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 affect the flow is subject to human factors. 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 modelling 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.

Even though the water flow in watercourse channels can be considered to be one-dimensional, in most cases, to some extent, we have ever more often met the requirement for multidimensional modelling. First and foremost, it is the water flow under floods when the water flows in the floodplain – two-dimensional flow models [10] – and the flow on structures, which is a purely three-dimensional task.

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 modelling 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 “pear-like structure”, it even plays a pivotal role. (fig. 3).

The authors' department has been involved in the problem of mathematical modelling for a number of years, and they have educated an experienced staff that understand the above-mentioned problem properly.

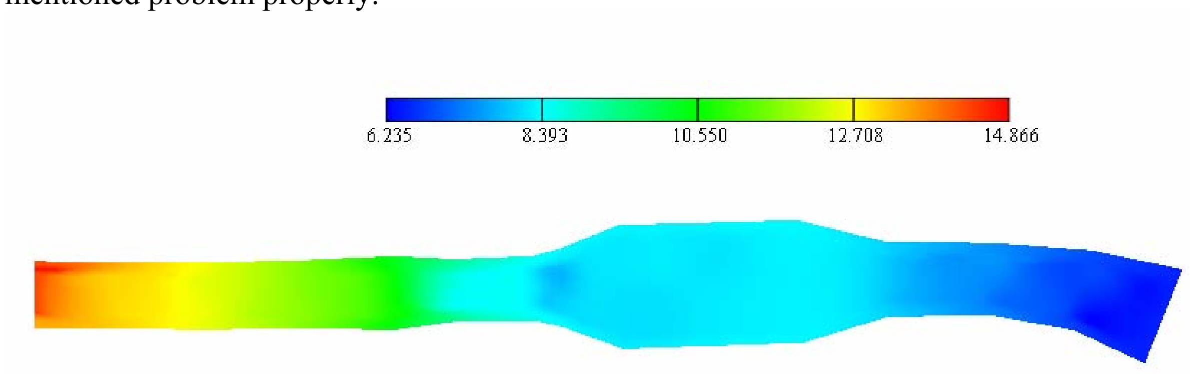

Fig. 3: Water level in the watercourse section with the pear-like structure (in meters) – a 2D model, software CCHE2D.

4 Physical modelling of problematic sections of watercourse sections

Physical modelling 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.) [8]. Physical modelling is based on the laws of model similitude.
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 modelling where the physical model serves for the calibration and the subsequent verification of the mathematical model.
Currently the problem of the reconstruction of two weir steps in km 12.038 and the river channel under the weirs on the Ostravice River in Hrabová, a part of Ostrava, before its mouth into the Odra was dealt with. The longitudinal scale is *1:50*. Its use is not only intended for water-management bodies, but also for local government.

Fig. 4: Watercourse section with the pear-shaped structure – a physical model; a problematic watercourse section due to silting under the weir in Vyšné Lhoty.

5 Electric impedance spectrometry

The electronic impedance spectrometry (EIS) has become, due to its descriptive nature, a popular analytic method and has ranked first in the study of physical and chemical properties of materials and living tissues. It was used in determining the chemical purity of materials, water content, concentration of solutions, material corrosion, pathological changes in cells, etc.

The basic principle of the method is the measurement of frequency characteristics of the impedance of the structure or material being measured. The frequency characteristics of the impedance Z can be generally expressed as a function of a complex variable:

$$Z(j\omega) = R + j\omega X \quad , \qquad (1)$$

where R is the resistance forming the real part of the impendence independent of the frequency,

X is the reactance, the imaginary part of the impedance, the size of which changes with the frequency,

ω is the cyclic frequency given in radians per second.

Method of measurement

Some methods of the measurement of electric impedance are shown, for example, in [1], [2]. The measurement of impedance using an impedance spectrometer developed in the laboratory of the HAAL Elektro, s.r.o. company [3] uses a comparative method, which is based on the comparison of the impedance, measured with a normal resistance R_{un}, the value of whose electric resistance is known, and its reactance is negligible in the frequency band considered.

The measurement date received in a PC [4] represent values of impedance of the equivalent circuit formed by a parallel combination of the resistance R and the capacity X, or inductance, where

$$X = j(\omega L - \frac{1}{j\omega C}) \quad . \qquad (2)$$

It must be emphasised that in case of parallel combination of the elements R, X, the calculation of the overall impedance issues from individual admitances (conductivities) of these elements and that it is true that:

$$Y = \frac{1}{jX} + \frac{1}{R} \quad , \qquad (3)$$

where the expression $G = 1/R$ represents the real part of the admitance (conductivity) and the imaginary part of the admitance can be written as $B = 1/X$. After rewriting the relation (3) using the stated notation, one obtains the analogy of the relation (1):

$$Z = G - jB \quad . \qquad (4)$$

The total impedance in the complex form is then expressed by the formula:

$$Z = \frac{1}{Y} = j\frac{XR^2}{R^2 + X^2} + \frac{RX^2}{R^2 + X^2} \tag{5}$$

By converting the expression (5) into the exponential form:

$$Z = |Z|\, e^{j\varphi_Z} \tag{6}$$

the module and the phase of impedance measured can be expressed.
The module $|Z|$ of the total impedance Z is expressed by the formula

$$|Z| = \sqrt{\left(\frac{XR^2}{R^2 + X^2}\right)^2 + \left(\frac{RX^2}{R^2 + X^2}\right)^2} = \sqrt{\frac{X^2R^2}{R^2 + X^2}} = \frac{1}{\sqrt{\frac{1}{X^2} + \frac{1}{R^2}}} = \frac{1}{|Y|}, \tag{7}$$

and the phase φ of the total impedance Z

$$\varphi_Z = atn\left(\frac{\mathrm{Im}}{\mathrm{Re}}\right) = atn\left(\frac{\frac{XR^2}{R^2 + X^2}}{\frac{RX^2}{R^2 + X^2}}\right) = atn\left(\frac{R}{X}\right) = -\varphi_Y \tag{8}$$

These values have been subsequently processed graphically. For compete information on the process being monitored, the graphical output has been supplemented with tabular processing in a text file, where the serial number of the measurement, the set frequency, the impedance of the real part, impedance of the imaginary part, the real part of the admittance, the imaginary part of the admittance, the impedance module, the phase of the impedance, the module of the admittance and the phase of the admitance are stated. This file can then be read into, for example, the MS Excel for further analysis.

The implemented apparatus meets high standards for accuracy and reproducibility of measurements. The following parameters were verified by metrology:

Range of impedance:	10 Ω – 10 MΩ
Frequency range:	10 Hz – 8 MHz
Accuracy of the impedance module:	+/- 0.2 % of the range
Phase accuracy:	+/- 0.2 °

The stated errors were only increased on the borders of the impedance ranges in phase angles close to *90*°, owing to the accumulation of rounding errors.

Owing to the requirements resulting from individual experiments, the apparatus was conceived in such a way as to enable the monitoring of the transition process (measurements on one pre-selected frequency) or the measurement of characteristics in the frequency band.

Great attention was paid to the shape, material and connection of the measurement electrodes. Laboratory experiments were performed with T-shaped aluminium electrodes, aluminium tubular electrodes (fig. 5) and with stainless steel rod-shaped electrodes that were evaluated as the most suitable from the viewpoint of handling. In individual experiments [5] two-electrode circuitry was used (voltage and current terminals were always connected to each of the measurement electrodes (fig. 4) or four-electrode circuitry (voltage and current terminals were connected to independent electrodes, which lowered the transition resistances between the electrodes and the measured environment, and it made it possible for the measurement apparatus to react to changes in the environment corresponding to the level of units of ohms, (fig. 6). It was just the four-electrode connection that proved to be highly sensitive to the slightest irregularities of the sample being tested in laboratory experiments (pits or material excess of the order of *1 cm*). This is why its further possibilities are now being examined in connection with the relative monitoring of the movement of the endangered banks and slopes.

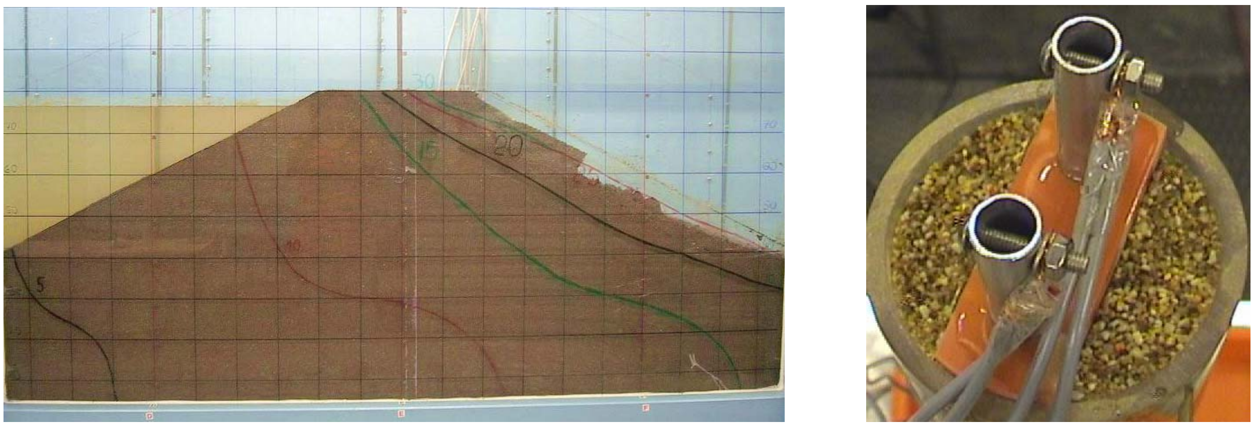

Fig. 5: Aluminium tubular electrodes – two-electrode connection

Fig. 6: Four-electrode connection in the monitoring of surface changes

6 Conclusions

Systematic modelling of watercourses is necessary both from the capacity and administration point of view. The use of the information obtained is also important from the flood-control point of view, both in extensive and local floods.
Mathematical and physical modelling provides the necessary accessories for the administration, organisation, and investment plans produced by watercourse administrators.
Electric impedance spectrometry is suitable as a non-destructive method for monitoring the stability and local watercourse levee failures. We also recommend it for more general use, particularly by watercourse administrators, or for monitoring protective dikes.

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Balance Of Precipitation Water And Nitrates Leaching In The Soil

Željko Vidaček, Mario Sraka, Danijela Vrhovec
Soil Science Department Faculty of Agriculture, University of Zagreb, Croatia

Abstract

Balance of precipitation water and nitrates leaching in the soil were measured at the lysimeter station Zagreb-Maksimir under the conditions of intensive crop rotation and humid continental climate of northwestern Croatian during the period 1990-1999. In natural state Garnier lysimeters contained homogeneous-isotropic soil of 0.6 m depth. Direct and/or indirect measurements included: precipitation, water percolation, supplies of available soil moisture, effective evapotranspiration and nitrate concentration in percolated water.

Annual precipitation amounted to 650.1 to 1040.7 mm and annual water percolation was 15 to 39% of fallen precipitation. Effective evapotranspiration ranged from 166.2 mm for winter vetch and rye grown in 1991/92 to 437.3 mm for soybeans grown in 1998. In the warm and dry part of the year (April-September) supplies of available water ranged from the field water capacity to the wilting point, and in the cold and humid part of the year (October-March) around the field water capacity.

In nitrogen fertilization with 24 to 156 kg of active ingredient per ha, nitrate concentrations in percolated water ranged from 0.2 to 260.5 mg NO_3^-/l. In the cold and humid part of the year with the highest leaching, some of the determined concentrations exceeded the maximum allowable ones (MAC), which pursuant to the statutory regulations of the Republic of Croatia amount to: less than 2.2 for water category I, 2.2 to 6.6. for water category II, 6.6 to 17.7 for water category III, 17.7 to 44.3 for water category IV, and more than 44.3 mg NO_3^-/l for water category V.

Keywords: water balance, alluvial soil, crops, water runoff, nitrates

Soil Science Department
Faculty of Agriculture
University of Zagreb
Svetošimunska 25, 10000 Zagreb
CROATIA
Tel: 00385-1-2393960
Fax: 00385-1-2393963
e-mail: **mario.sraka@agr.hr**

Plant Cultivation under Environmental Changing Conditions in Groundwater

Zvonimir Vukelic(1), Ivan Gotic(2), Marija Vukelic-Sutoska(3)

ABSTRACT

Tremendous successes have been obtained in Croatia and 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 Croatia and 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 maximaze economic benefits. The accurate planning of water resources systems is a complex interdisciplinary problem which involves complicated environmental, ecological and economical aspects.

1. INTRODUCTION

It seems that groundwater is more protected against pollution than surface water. Nevertheless it is still subject to the thermal, biological and chemical pollution from a wide spectrum of environmental, domestic, industrial and agricultural sources.
In fact, due to the very low natural flow velocities, groundwater pollution proves to be especially dangerous. Usually a ground water pollution becomes evident very late and decontamination or rehabilitation is, if possible at all, extremely difficult.

(1) Prof. dr. sc., University St Cyril and Methodius, Faculty of Civil Engineering, Skopje, Macedonia, vukelic@gf.ukim.edu.mk

(2) Prof. dr. sc., University in Zagreb, Geotechnical Faculty in Varazdin, Varazdin, Croatia

(3) Assist. dr. sc., University St Cyril and Methodius, Faculty of Agriculture, Skopje, Macedonia, marijavs@zf.ukim.edu.mk

2. PRIMARY FACTORS AFFECTING PLANT ESTABLISHMENT AND GROWTH

The plants grown for reclamation purposes are, as it is very well known, 94 to 99.5 percent carbon, hydrogen, and oxygen, and these are obtained by the plants from air and water. There are approximately eight environmental factors that have a major effect on plants : sunlight, temperature, carbon dioxide content (they are function in atmosphere and soil), animals and other plants (they are function above soil and in soil), water (primarily in soil), oxygen , nutrients and acidity (they are function in soil). In Figure 2.1 are shown relative amounts.

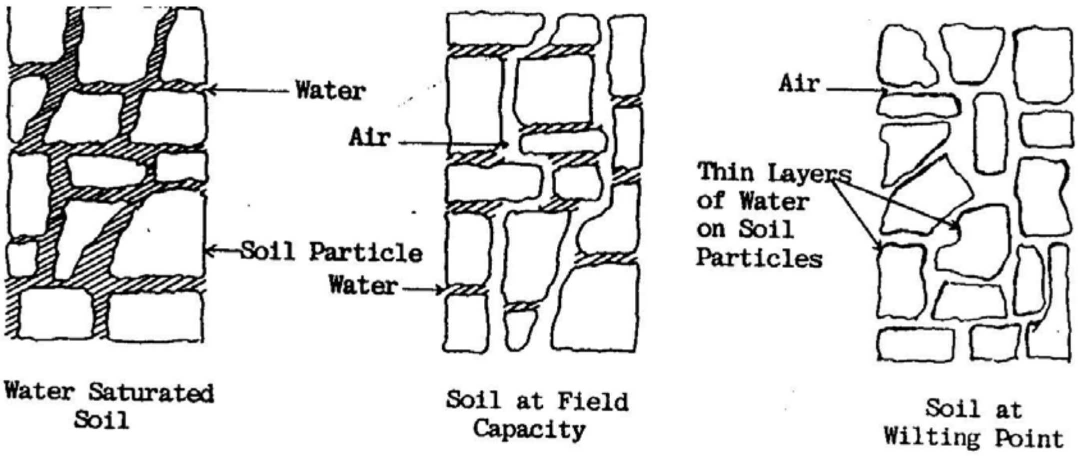

Figure 2.1-Relative amounts of water and air in a soil at three soil moisture conditions

There are four soil characteristics that have a major effect on the ability of soil to absorb water, store water, and allow plant roots to absorb water. These four characteristics are: (1) soil texture, (2) soil structure, (3) soil organic matter, and (4) soil depth.

Soil texture is determined by the relative percents of sand, silt, and clay found in a soil.Figure 2.2 shows the approximate effect of texture on a soil's ability to hold water. Soils that have a high sand content are said to have a coarse texture, and those with high silt or clay contents are said to have a fine texture. For example, if a soil was composed of 40 percent sand, 40 percent silt, and 20 percent clay, the texture classification would be loan. Soil structure can modify the effect of soil texture and significantly change soil moisture relationship. Two general types of pores exist in most soils. These two types are macropores and micropores. Soil structure is the aggregation of soil particles into clusters of particles that produce a characteristic form.

Organic matter is the rotting or decomposing remains of plants or animals. Organic matter not only has much to do with soil water, but it is also important in the soil nutrient cycle. The final soil characteristic affecting the availability of soil water to plants is soil depth. Soil depth may be defined as the distance from the soil surface down to anything that prevents a plant root from growing and absorbing water and nutrients.

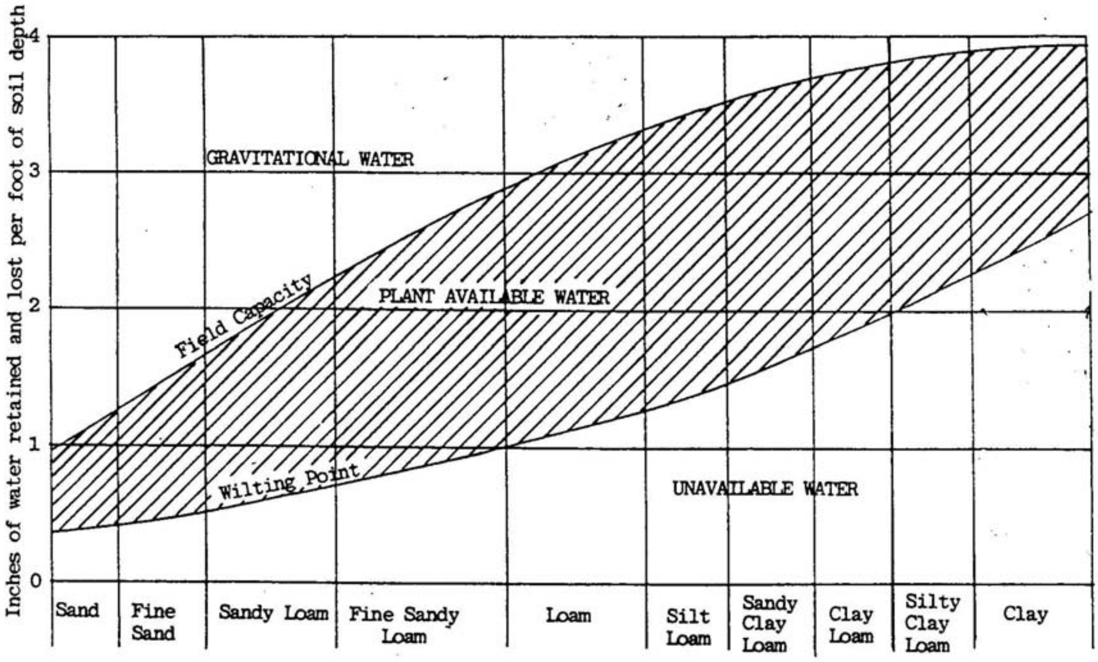

Figure 2.2-Effect of texture on a soil's ability to hold water
(Source: U.S.D.A., Forest Service, Handbook on Soils)

3. QUANTITATIVE GROUNDWATER MANAGEMENT

The basic laws governing the flow of water in saturated or unsaturated porous media are the generalized Darcy's law and the equation of continuity. Introducing the flow equation into the balance equation leads to a set of partial differential equations governing three-dimensional, transient, saturated-unsaturated groundwater flow.
Fluid density and viscosity depend on the actual load temperature. Saturation and relative permeability are function of the pressure distribution.
Figure 3.1 gives a classification of different flow situations and resulting model type which further depends on the actual problem and the questions to be answered (Walter Pelka, 1985.).
The assumption and simplifications introduced to the basic equation depend mainly on the following assumption: related to the flow conditions (saturation, viscosity, density), related to time behaviour (quasy-steady state), and to space dimensions.
The decision which assumption can be made is in accordance with actual flow situation and given problem, without violating essential characteristics of the flow field and the aquifer, without endangering the prognostic abilities of the model and the reliability of the model results, and without demanding economicly enfeseable efforts.

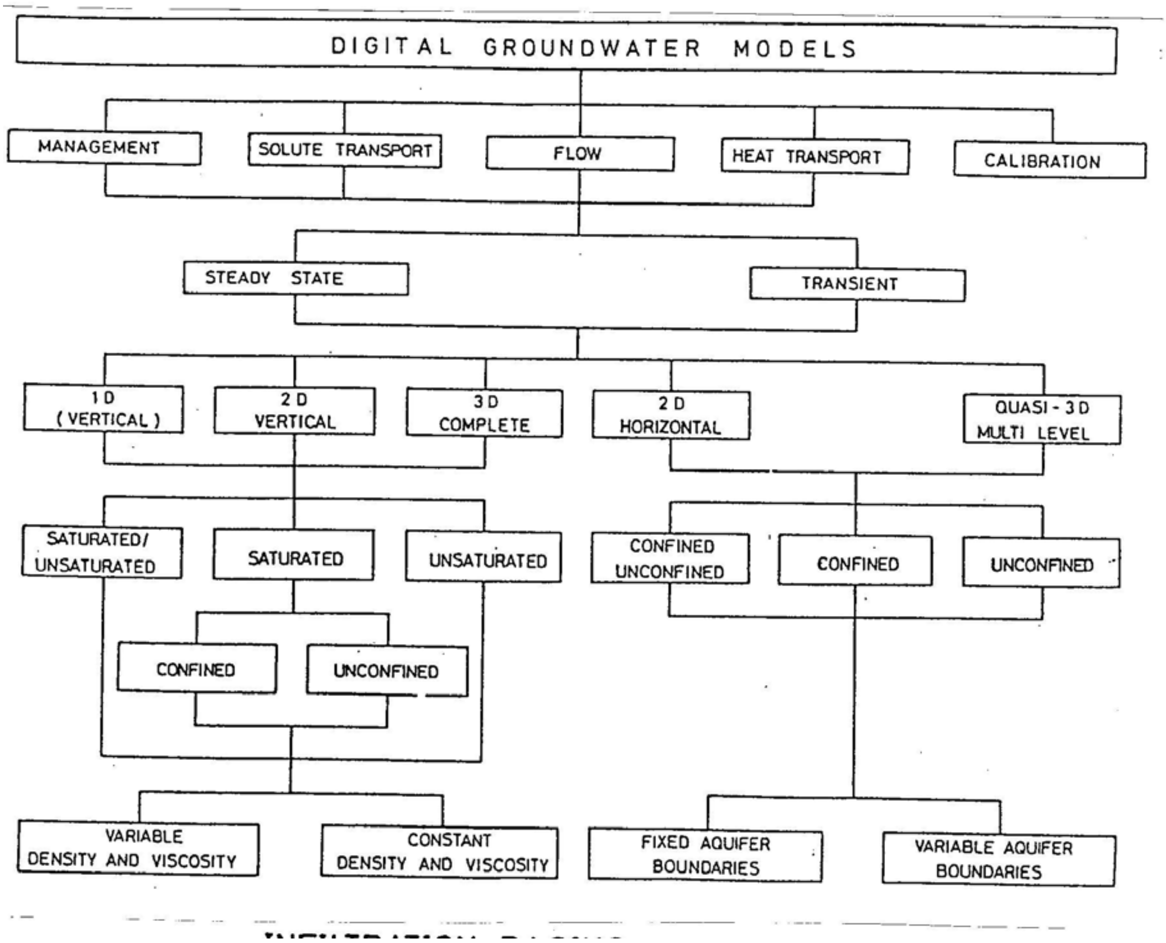

Figure 3.1-Classification of Groundwater Models (After W. Pelka, 1985)

It can be seen from the complexity of the physical problem and the resulting mathematical description, that, in order to conserve the achievements of the powerfull mathematical model during the process of solution, it becomes necessary to choose a method of high efficiency, i.e. numerical approaches have to be applied. The closeness of the finite differences method to the classical differential equation made it the natural choice in early numerical modelling work. Considering groundwater flow, the finite element method seems to be the more convenient approach in comparison with differential forms because of the adaptability to complex and arbitrary geometrical, geological and hydrological properties of the area under consideration, the possibility of condensing the spatial discretization in regions of special interest and higher demands in accuracy without effecting other regions, the convenient handling of all types of boundary conditions, and the convenient handling of arbitrary tensorial quantities. The chief drawback of the finite difference method, however, is that the discretization is designed to evaluate normal gradients only. Applying a finite element approach, arbitrary gradients, and by this, fluxes and velocities can readily be evaluated (W. Pelka, 1985).

4. SOURCES OF GROUNDWATER POLLUTION

Besides the quantity problem the quality problem has become one of the most limiting factors in water resources development. One of the typical sources of groundwater pollution are old or wild landfills for domestic or industrial wastes. Solutes are moving with the infiltrating rainwater from the landfill to the saturated groundwater flow, where they are transported further downstream. A similar process is the infiltration of polluted surface water into the aquifer. This may happen because of a permeable natural riverbed, or intentionally, for artificial groundwater recharge, as shown Figure 4.1.

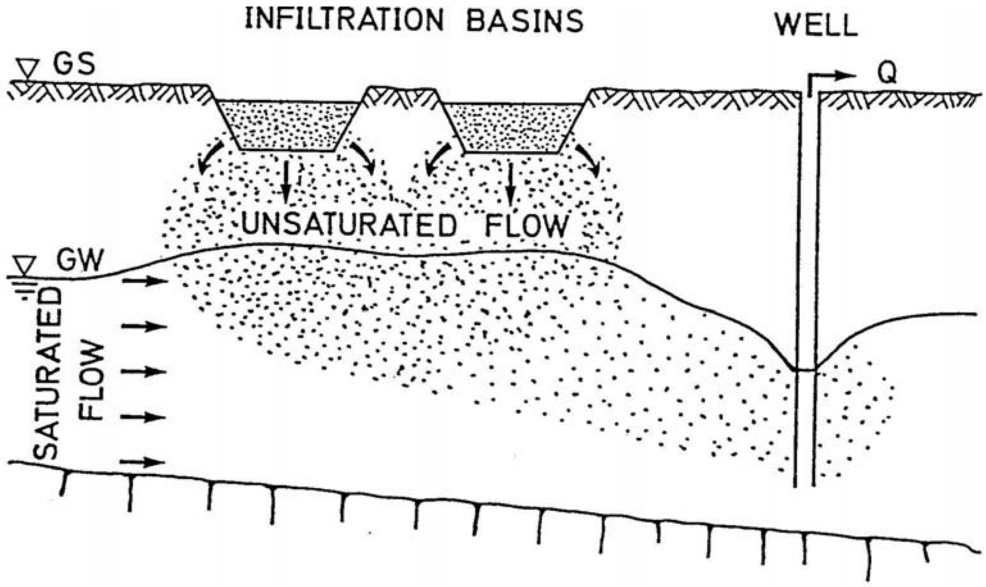

Figure 4.1-Infiltration from Polluted Surface Waters (After W. Pelka, 1985)

Except agricultural waste deposits, agricultural pollution is in general a spatial distributed source. Fertilizers, salt, nitrates and pesticides are transported with the infiltrating rain or irrigation water downwards to the water level. In this content it should also be noted that any air pollution may cause a long-term groundwater pollution.
In most of the cases mentioned above answers must be found to one or more of the following questions: Is there any danger of pollution for well fields and freshwater supply, when and at which concentration the pollution will reach the wells, how long will the pollution last, are there any measures which can be taken against this pollution, how will the efficiency be concerning these measures, when do they become effective, and where is the source of the pollution.

5. SOIL RECONSTRUCTION AND MANAGEMENT

Reconstruction of suitable soil profiles after mining involves placement of soil and overburden material in such a way as to establish physical and chemical characteristics at, and just below, the surface of the landscape that insure restoration or even improvement of the productive potential of the mined land (Figure 5.1). The best method of topsoil handling is thus to avoid stockpiling by immediately reapplying topsoil in an area contemporaneously under reclamation.

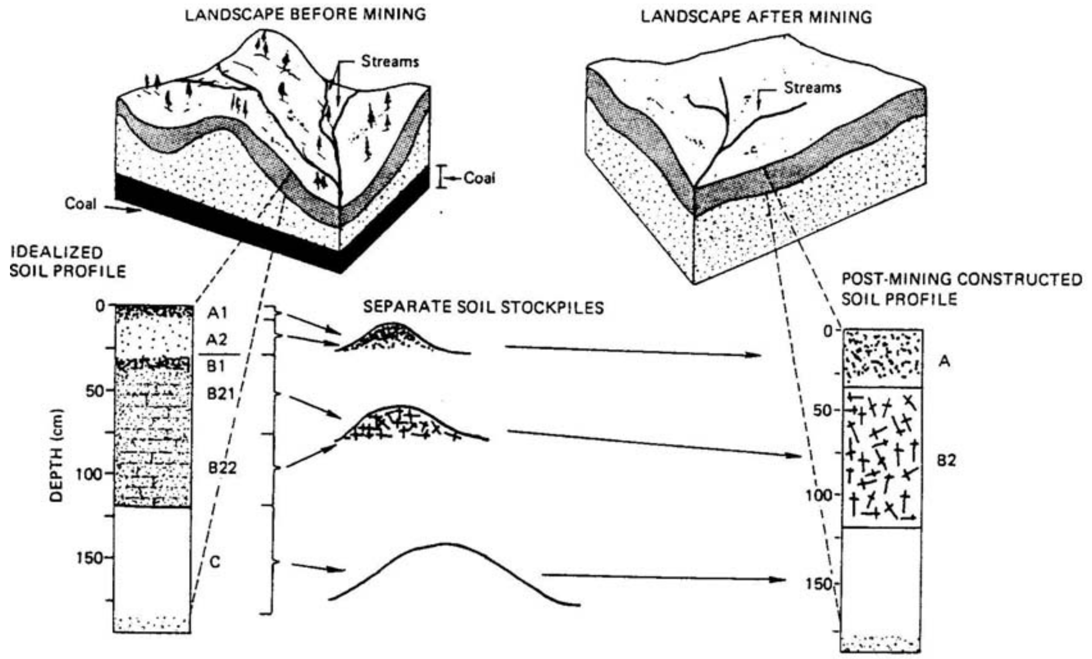

Figure 5.1-Idealized soil-reconstruction scheme. The stockpiling of soil horizons and the emplacement in reconstructed soil of mixed horizons can be managed so as to enhance plant growth. The two A horizons are mixed to form the top layer of the new soil. The two B horizons are mixed to provide new subsoil that has better overall qualities than did either of the original subdivisions of the B horizon.

6. CONCLUSION

Water resources engineering and management are issues of ever increasing importance in the densely populated and highly industrialized regions in Croatia and Macedonia. This includes both the quantitative problems of water availability for the needs of society and the problems of water quality. Groundwater is the most important drinking water resources in two countries. This means, that management and preventive protection of the groundwater resources both quantitatively and qualitatively are of central importance, because prevention is not only better but also much cheaper than remediation and repair.
Numerical models have become invaluable tools in environmental engineering for the quantitative and qualitative development and management of water resources. No other approach available is by far that powerful, predictive, and reliable. Different methods are well documented in the literature and in many cases models or parts of models are commercially available in form of programs or program systems. One should know that the basic assumption of the different models, their restrictions and their features in the levels of physical model concept, mathematical Formulation and numerical solution. After a general introduction to the subject and its place in water resource engineering the following issues are addressed: from concepts to quantification (experiments), from processes to technologies (interactions and scales), from processes to system (numerical models), and from technology to application (strategies).
It is also recognized that reclamation, or interested in reclamation, is not formal training in all phases of plant and soil sciences. Surface mine reclamation is often thought of in terms of revegetation, and this is not unreasonable. Planned revegetation of a minesoil controls erosion, puts the soil back into useful production and creates an aesthetically pleasing terrain.

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Nonlinear Model of Deep Water Random Wave Load on Horizontal Members of Offshore Structures

Živko Vuković & Pejo Brica

Faculty of Civil Engineering, University of Zagreb,
Kačićeva 26, 10000 Zagreb, Croatia
E-mail: vukovic@master.grad.hr
pejo@master.grad.hr

Abstract

In this paper the authors present a developed analytical procedure for predicting the wave load on horizontal cylindrical members of offshore structures to the action of deep water random waves, representing the nonlinear drag force by cubic approximation. Using Morison's equation, the spectra of the horizontal and vertical component of wave force on horizontal members are defined by the spectral analysis method. A numerical example is included to demonstrate the potential of the analytical model and to quantify the effects of nonlinear drag force on the structural load.

1 Introduction

The in-line wave forces acting on a slender member of offshore structure consist of two parts; drag and inertia. In the case when the wave length is much larger than the characteristic structural diameters, the drag forces can be significant. Since the drag forces, as normally defined through Morison's equation, are nonlinear, it is difficult to obtain an analytical solution, particularly when sea state is random.

With some assumptions and additional statistical approximations, one can easily study the above problem. The same is applied in this paper, which deals with the nonlinear drag force along the lines given by Borgman[1,2]. In fact, an attempt is made to exemplify and quantify the effects of nonlinear drag force on the second-order statistics of structural wave loading.

2 Assumptions

(a) In the following analysis the waves are considered in deep water. The linear (Airy) wave theory is employed to relate surface wave parameters and wave motions. The sea surface elevation is assumed to be an ergodic, normally (Gaussian) distributed random process of zero mean and uni-directional.

(b) It is assumed that on the analysed section the structure member is laid horizontally; that it is of a constant outside diameter; rigid; fixed; slender (in the sense that its characteristic dimension - diameter is relatively small in relation to wave length); that it is located in the horizontal co-ordinate $x = 0$; and that waves arrive perpendicularly to its axis.

(c) For the evaluation of wave force, Morison's equation[3] is used in this study. That is, wave force, $F(z,t)$, is considered to be consisting of two parts; the drag component, nonlinearly related to water particle velocity; and the inertia component, linearly proportional to water particle acceleration.

Horizontal component, $F_x(z,t)$, and vertical component, $F_z(z,t)$, of this force per unit length on the horizontal member of the marine structure, Fig. 1, can be expressed as:

$$F_x(z,t) = F_{Dx}(z,t) + F_{Ix}(z,t) = u_D v_x(z,t)\left|v_x(z,t)\right| + u_I a_x(z,t) \quad (1)$$

$$F_z(z,t) = F_{Dz}(z,t) + F_{Iz}(z,t) = u_D v_z(z,t)\left|v_z(z,t)\right| + u_I a_z(z,t) \quad (2)$$

where:

$$u_D = 0.50 C_D \rho D \quad (3)$$

$$u_I = 0.25 C_I \rho \pi D^2 \quad (4)$$

In the above equations $F_{Dx}(z,t)$ and $F_{Ix}(z,t)$ = horizontal components of drag and inertia force, respectively; $F_{Dz}(z,t)$ and $F_{Iz}(z,t)$ = vertical components of drag and inertia force, respectively; C_D and C_I = drag and inertia coefficients, respectively; ρ = water density; D = outside structure member diameter; x and z = horizontal and vertical co-ordinates, i.e. horizontal and vertical axes, respectively; $v_x(z,t)$ and $v_z(z,t)$ = horizontal and vertical components of water particle velocity, respectively; $a_x(z,t)$ and $a_z(z,t)$ = horizontal and vertical components of water particle acceleration, respectively; and t = time.

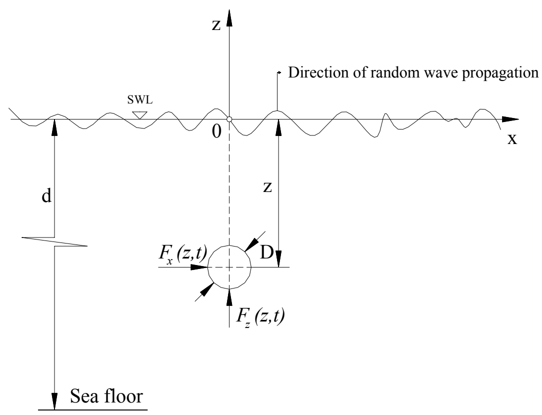


Fig. 1: Definition drawing of horizontal members wave load

3 Review of random wave load analysis

With the previous assumptions and for the case of deep water waves action, Borgman[1,2] obtained the theoretical autocorrelation function, $R_{FF}(z,\tau)$, for the horizontal force, $F(z,t)=F_x(z,t)$, on a rigid and vertical cylinder per unit length, as:

$$R_{FF}(z,\tau) = u_D{}^2 \sigma_{v_x}{}^4(z) G\left[\frac{R_{v_x v_x}(z,\tau)}{\sigma_{v_x}{}^2(z)}\right] + u_I{}^2 R_{a_x a_x}(z,\tau) \tag{5}$$

where:

$$G(r) = \frac{(4r^2+2)\,\text{arc}\ \sin r + 6r\sqrt{1-r^2}}{\pi} \tag{6}$$

$$r = \frac{R_{v_x v_x}(z,\tau)}{\sigma_{v_x}{}^2(z)} \tag{7}$$

$$\sigma_{v_x}(z) = \left[\int_0^\infty S_{v_x v_x}(z,\omega)\,d\omega\right]^{1/2} \tag{8}$$

$$S_{v_x v_x}(z,\omega) = \left(\omega \frac{\cosh\left[k\left(z+d\right)\right]}{\sinh kd}\right)^2 S_{\eta\eta}(\omega) \tag{9}$$

In the above equations $\sigma_{v_x}(z)$ = standard deviation of $v_x(z,t)$; $R_{v_x v_x}(z,\tau)$ and $R_{a_x a_x}(z,\tau)$ = autocorrelation functions of $v_x(z,t)$ and $a_x(z,t)$, respectively; τ = time lag in autocorrelation functions; $S_{v_x v_x}(z,\omega)$ = spectrum of $v_x(z,t)$; $S_{\eta\eta}(\omega)$ = deep water (uni-directional) wave spectrum; ω = wave frequency; k = wave number; and d = water depth.

The wave number is related to wave frequency by:

$$\omega^2 = gk \tanh kd \tag{10}$$

where g = acceleration of gravity.

The function $G(r)$ can be expanded in a power series in r as follows:

$$G(r) = \frac{1}{\pi}\left(8r + \frac{4r^3}{3} + \frac{r^5}{15} + \frac{r^7}{70} + \frac{5r^9}{1008} + \ldots\right) \tag{11}$$

This series converges quite rapidly for $0 \le r \ge 1$.

At $r = 1$, the linear approximation:

$$G_1(r) = \frac{8r}{\pi} \tag{12}$$

differs from $G(r)$ by 15 %.

The cubic approximation:

$$G_3(r) = \frac{1}{\pi}\left(8r + \frac{4r^3}{3}\right) \tag{13}$$

differs from *G(r)* at *r* = 1 by only 1.1 %. That means, if the drag contribution is approximated by the two first terms in the series, the maximum error is only of the order of 1%.

Hence, Eq. (5) may be recast as follows with good accuracy for most engineering problems:

$$R_{FF}(z,\tau) = \frac{u_D{}^2\sigma_{v_x}{}^4(z)}{\pi}\left[\frac{8R_{v_xv_x}(z,\tau)}{\sigma_{v_x}{}^2(z)} + \frac{4R_{v_xv_x}{}^3(z,\tau)}{3\sigma_{v_x}{}^6(z)}\right] + u_I{}^2R_{a_xa_x}(z,\tau) \tag{14}$$

The Fourier transform of $R_{FF}\,(z,\tau)$ gives the spectral density of *F(z,t)* as:

$$S_{FF}(z,\omega) = \frac{u_D{}^2\sigma_{v_x}{}^4(z)}{\pi}\left\{\frac{8S_{v_xv_x}(z,\omega)}{\sigma_{v_x}{}^2(z)} + \frac{4\left[S_{v_xv_x}(z,\omega)\right]^{*3}}{3\sigma_{v_x}{}^6(z)}\right\} + u_I{}^2S_{a_xa_x}(z,\omega) \tag{15}$$

where the three-hold convolution of $S_{v_xv_x}(z,\omega)$ with itself is given as:

$$\begin{aligned}&\left[S_{v_xv_x}(z,\omega)\right]^{*3} = S_{v_xv_x}(z,\omega) * S_{v_xv_x}(z,\omega) * S_{v_xv_x}(z,\omega) = \\ &\int_{-\infty}^{\infty}\int_{-\infty}^{\infty}\left[S_{v_xv_x}(z,\omega'')S_{v_xv_x}(z,\omega'-\omega'')d\omega''\right] S_{v_xv_x}(z,\omega-\omega')d\omega'\end{aligned} \tag{16}$$

and the deep water spectrum of $a_x(z,t)$ is defined as:

$$S_{a_xa_x}(z,\omega) = \left(\omega^2\frac{\cosh\left[k\left(z+d\right)\right]}{\sinh kd}\right)^2 S_{\eta\eta}(\omega) \tag{17}$$

4 Nonlinear model of random wave load on horizontal members

As shown by Vuković & Kuspilić[4,5,6] and by analogy with Eqs. (1), (2) and (15), it follows that the load spectra on a horizontal cylindrical member of offshore structure due to the horizontal component, $S_{F_xF_x}(z,\omega)$, and vertical component, $S_{F_zF_z}(z,\omega)$, of random wave force per unit length are defined by the equations:

$$S_{F_xF_x}(z,\omega) = \frac{u_D{}^2\sigma_{v_x}{}^4(z)}{\pi}\left\{\frac{8S_{v_xv_x}(z,\omega)}{\sigma_{v_x}{}^2(z)} + \frac{4\left[S_{v_xv_x}(z,\omega)\right]^{*3}}{3\sigma_{v_x}{}^6(z)}\right\} + u_I{}^2S_{a_xa_x}(z,\omega) \tag{18}$$

$$S_{F_zF_z}(z,\omega)=\frac{u_D{}^2\sigma_{v_z}{}^4(z)}{\pi}\left\{\frac{8S_{v_zv_z}(z,\omega)}{\sigma_{v_z}{}^2(z)}+\frac{4\left[S_{v_zv_z}(z,\omega)\right]^{*3}}{3\sigma_{v_z}{}^6(z)}\right\}+u_I{}^2S_{a_za_z}(z,\omega) \tag{19}$$

where:

$$\sigma_{v_z}(z)=\left[\int_0^\infty S_{v_zv_z}(z,\omega)d\omega\right]^{1/2} \tag{20}$$

$$S_{v_zv_z}(z,\omega)=\left(\omega^2\frac{\sinh\left[k\left(z+d\right)\right]}{\sinh kd}\right)^2 S_{\eta\eta}(\omega) \tag{21}$$

$$\begin{aligned}&\left[S_{v_zv_z}(z,\omega)\right]^{*3}=S_{v_zv_z}(z,\omega)*S_{v_zv_z}(z,\omega)*S_{v_zv_z}(z,\omega)=\\&\int_{-\infty}^{\infty}\int_{-\infty}^{\infty}\left[S_{v_zv_z}(z,\omega'')S_{v_zv_z}(z,\omega'-\omega'')d\omega''\right]S_{v_zv_z}(z,\omega-\omega')d\omega'\end{aligned} \tag{22}$$

$$S_{a_za_z}(z,\omega)=\left(\omega^2\frac{\sinh\left[k\left(z+d\right)\right]}{\sinh kd}\right)^2 S_{\eta\eta}(\omega) \tag{23}$$

In the above equations $\sigma_{v_z}(z)=$ standard deviation of $v_z(z,t)$; $S_{v_zv_z}(z,\omega)$ and $S_{a_za_z}(z,\omega)=$ spectra of $v_z(z,t)$ and $a_z(z,t)$, respectively; and $\left[S_{v_zv_z}(z,\omega)\right]^{*3}=$ three-hold convolution of $S_{v_zv_z}(z,\omega)$ with itself.

Substituting the velocity and acceleration spectrum functions from Eqs. (9), (17), (21) and (23) into Eqs. (18) and (19), it follows that:

$$\begin{aligned}S_{F_xF_x}(z,\omega)=&\frac{8}{\pi}\left\{\omega u_D\sigma_{v_x}(z)\frac{\cosh\left[k\left(z+d\right)\right]}{\sinh kd}\right\}^2 S_{\eta\eta}(\omega)+\frac{4u_D{}^2\left[S_{v_xv_x}(z,\omega)\right]^{*3}}{3\pi\sigma_{v_x}{}^2(z)}+\\&+\left\{\omega^2u_I\frac{\cosh\left[k\left(z+d\right)\right]}{\sinh kd}\right\}^2 S_{\eta\eta}(\omega)\end{aligned} \tag{24}$$

$$\begin{aligned}S_{F_zF_z}(z,\omega)=&\frac{8}{\pi}\left\{\omega u_D\sigma_{v_z}(z)\frac{\sinh\left[k\left(z+d\right)\right]}{\sinh kd}\right\}^2 S_{\eta\eta}(\omega)+\frac{4u_D{}^2\left[S_{v_zv_z}(z,\omega)\right]^{*3}}{3\pi\sigma_{v_z}{}^2(z)}+\\&+\left\{\omega^2u_I\frac{\sinh\left[k\left(z+d\right)\right]}{\sinh kd}\right\}^2 S_{\eta\eta}(\omega)\end{aligned} \tag{25}$$

On of the main difficulties in practical application of Eqs. (24) and (25) lies in the evaluation of the convolution integrals, Eqs. (16) and (22), which is a rather time consuming numerical operation.

5 Numerical example

In order to illustrate the previous analysis, a numerical example of random wave load on a horizontal cylindrical member of a offshore structure is presented in the text below.

It is assumed that the water depth at the member location, d = 75.0 m; water density, ρ = 1025.0 kg/m^3; acceleration of gravity g = 9.81 m/s^2; vertical co-ordinates of member horizontal axis, z = - 10.0, - 20.0 and - 30.0 m (Fig. 1); member outside diameter, D = 0.8 m; and the values of drag and inertia coefficients, C_D = 1.0 and C_I = 2.0, respectively.

For the deep water wave spectrum we will use Tabain's[7] wave spectrum applicable for the Adriatic Sea conditions, for the significant wave hight, H_s = 6.0 m.

The so selected random wave climate parameters and the diameter of a hypothetical member grants the drag loading regime.

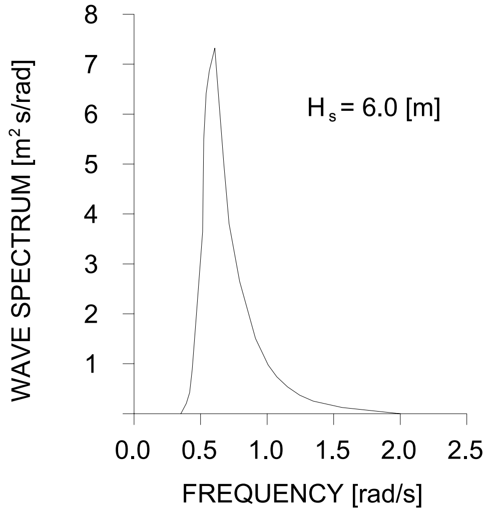
For the purpose of evaluation of the convolution integrals given by Eqs. (16) and (22), an effective computation algorithm is developed by means of the fast Fourier transformation technique.

The calculation results are shown in Fig. 2 (a) to 2 (c). Fig. 2 (a) represents Tabain's wave spectrum. The horizontal force spectra, $S_{F_xF_x}(z,\omega)$ from Fig 2 (b), and vertical force spectra, $S_{F_zF_z}(z,\omega)$, from Fig. 2 (c), are calculated for different member depths according to Eqs. (24) and (25), respectively.

It can be clearly seen from Fig. 2 (b) and 2 (c) that the influence of the drag force nonlinear term is considerable, both for the increase of the spectra peak values and for the increase of the total area below the spectra. Besides, the occurrence of spectra secondary peaks is evident in the region of higher frequencies, which can in the case of a certain type of structures cause a significantly higher displacement response that cannot be defined by the application of linear model.

This points to the need of taking into account the drag force nonlinear term while calculating the response of offshore structures located in the drag loading regime.

(a)



Tabain's wave spectrum

$$S_{\eta\eta}(\omega) = 0.862\frac{0.0135g^2}{\omega^5}\exp\left[-\frac{5.186}{\omega^4 H_s^{\,2}}\right]1.63^p$$

$$p = \exp\left[-\frac{(\omega-\omega_m)^2}{2\sigma^2\omega_m^{\,2}}\right];\sigma = \begin{Bmatrix} 0.08 \;\; for\; \omega \le \omega_m \\ 0.10 \;\; for\; \omega > \omega_m \end{Bmatrix};$$

$$\omega_m = 0.32 + \frac{1.80}{H_s^{\,2} + 0.60}$$

(b) (c)

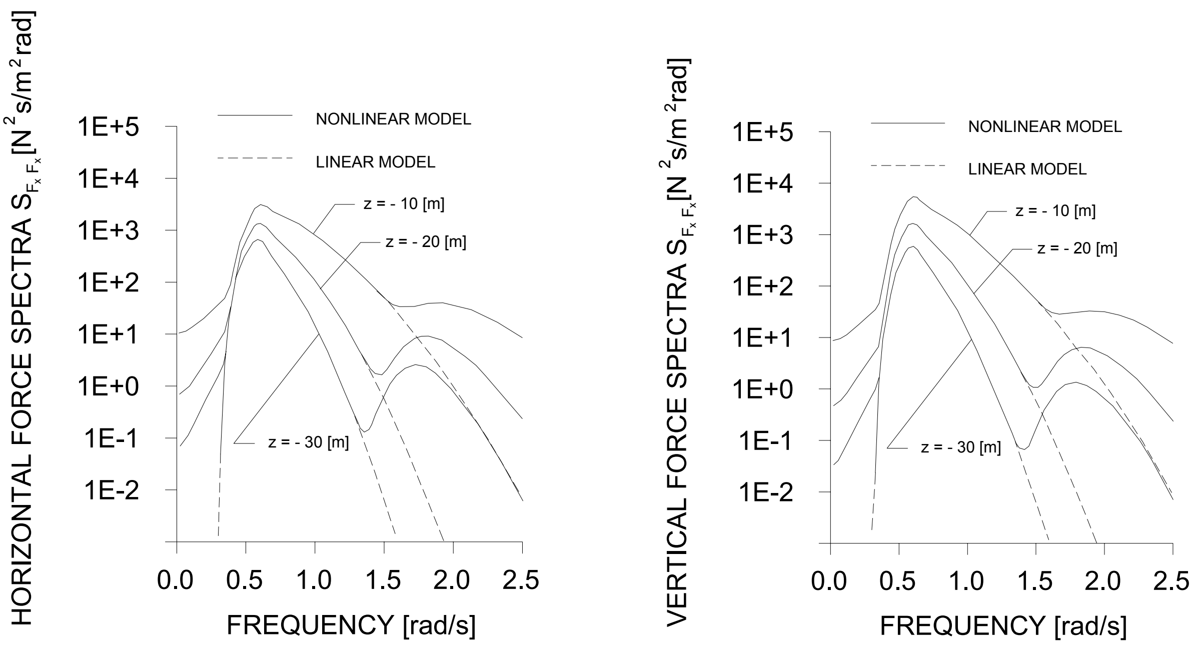


Fig. 2: Wave spectrum and random wave load spectra for a hypothetical horizontal member of offshore structures

6 Conclusion

A nonlinear random wave load effects on horizontal cylindrical members of offshore structures are presented by applying the spectral analysis method. The performed analyses and the results of the numerical example point to the considerable influence of the drag force nonlinear term in the calculation of the wave load of horizontal members located in the drag loading regime. It is therefore justified to use in engineering problems for such cases a nonlinear model of random wave load.

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Planning, Design, Construction and Operation Phases of Irrigation Systems in Macedonia

Zvonimir Vukelic(1), Ordan Cukaliev(2), Marija Vukelic-Sutoska(3), Lidija Trajanoska(4)

Abstract
Emphasis is placed in the design of new schemes, notwithstanding the fact that actually a major part of the work of the modern irrigation engineer will consist of the rehabilitation and modernization of existing schemes, which are, however, more difficult to plan and design because the existing hydraulic infrastructure, even when not the optimal one, has to be taken into account. Irrigation systems are constructed and operated to create favourable conditions for crop growth. Improving the welfare or social well being, and rehabilitation agriculture farmer involves several factors, in addition to a well operated and maintained irrigation water delivery system, organization viability, funding operation and mainetenace,, charges, establishing water charges, payment methods, organization management, system operation and system maintenance. This paper presents some topics of water management and water control, water management for agriculture under changing conditions in Macedonia, planning, design, construction and operation phases, data for technical planning and design, technical aspects of water control for irrigation, some elements of the irrigation system, secondary units, the commanded area, site characteristics, available information, preliminary site investigations, field surveys and sedimentation.

1 Introduction

Development of agriculture in Macedonia is followed by the water management plans with respect to water protection and influence of water management only. As a result of using agricultural land for other purposes, mainly for construction, mining, creating new forests and water reservoirs the area of agricultural land is decreased. The development after 1990 has been characterised by the process of **transformation of agriculture, privatization and restitution of the property.** The second important factor concerning agricultural influence on water management **was the concentration and use of irrigation systems. The third important influence of the agriculture on water management is agricultural pollution.** 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

(1) Prof. dr. sc., University St Cyril and Methodius, Faculty of Civil Engineering, Skopje, Macedonia, vukelic@gf.ukim.edu.mk
(2) Prof. dr. sc. University St Cyril and Methodius, Faculty of Agriculture, Skopje, Macedonia, Ordan.Cukaliev@zf.ukim.edu.mk
(3) Assist. dr. sc. University St Cyril and Methodius, Faculty of Agriculture, Skopje, Macedonia, marijavs@zf.ukim.edu.mk
(4) Grad. meteorol. eng., Republic Hydrometeorological Institute, Skopje, Macedonia, ltrajanoska@yahoo.com

frozen soil surface and others. Modernization of a scheme is needed when there is a change in objectives, be it that the original objectives are not anymore valid or that those objectives were wrong on unrealistic in the first place. Modernization could be needed in some cases immediately after the commissioning of the scheme.

2 Water Management and Water Control

Water control is a technical aspect of **water management. irrigation and drainage systems** form only a small part of the technical measures that are needed for proper water management in agriculture. The action required in water management is summarized as follows (Cunha, 1977):

a. Planning: national (50 years time horizon), river basin (20-25 time years horizon), project (varying time horizons),
b. Balance availability and demand, proper criteria, daya collection, storage, retrieval, and analysis,
c. Preparation, regulation and implementation of laws, definition of possible uses and users, quantities for each purpose, maximum admissible pollution levels, conditions for licensing water use, charge system, fines and penalties, responsible authorities, public and private entities,
d. Design, construction, operation and maintenance of works,
e. Economic incentives and financial support for rational water use,
f. Training, skilled personnel at various levels,
g. Research into the problems related with the foreseen development, these problems include social and political problems in water use,
h. Documentation and information at national and basin level for engineers and politicians, also public relations to be included here, and
i. International cooperation, especially in international river basins.

It is necessary to always be ware of the constraints decision makers are subjected to, O'Riordan and More (1969) summarized these constraints as follows:

a. Physical-there is a physical limit to possible water development,
b. Economic-development will be limited to the total amount of money available and other demands for that money,
c. Policy-certain water demands (such as for domestic purpose) must not be allowed to suffer regardless of what happens to other uses,
d. Legal-court decision and legislation are often the result of compromise between the demands of different groups with biased interests,
e. Administartive-willing cooperation between different local or state authorities is needed to achieve an optimum water resources management programme,
f. Ownership-private land ownership, if opposed to water development plans, may limit such water development,
g. Quantification-inability to quantity the benefits and costs, especially those relating to social or aesthetic factors, places greater emphasis on the previous six constraints, and
h. Perception-limited understanding of the range of options available of the rate of alternate uses by the decision maker may place some constraint on water resources development

The are the following aspects of water control for irrigation:

1. Sources of water	
1.1 Rainfall	Engineering hydrology, Statistics
1.2 Surface of water	-same, Hydraulics, Hydrometry, Pumping stations
1.3 Ground water	-same, Groundwater flow, Tubewells
2. Irrigation system	
2.1 Main and secondary	Hydraulics, Sediment transportation, Irrigation and Drainage systems, Canals and related structures, Operation and Maintenance, mathematical modeling, (Bank and bottom protection)
2.2 Tertiary	Hydraulics, Tertiary unit design
2.3 Field	Field irrigation and drainage, Modern irrigation methods
3. Water use and application Consumptive and non-consumptive use	Field irrigation and drainage, Principles of agronomy, Tropical crops, Agricultural land reclamation, Land evaluation, Physics of soil moisture, (Land leveling, Land-subsidence)
4. Drainage system	
4.1 Field	Field irrigation and drainage
4.2 Tertiary	Hydraulics, Tertiary unit design
4.3 Main and secondary	Hydraulics, Sediment transportation, Irrigation and drainage and systems, Canals and related structures, Operation and maintenance, (Mathematical modeling, bank and bottom protection)
4.4 Cross drainage	Engineering hydrology, Field irrigation and draianage, Irrigation and drainage systems
5. Sinks of water	
5.1 Evaporation	Engineering Hydrology
5.2 Surface water	-same, Rivers, Drainage sluices
5.3 groundwater	-same, Physics of soil moisture, Groundwater flow

In Figure (1) are shown	Data base, processing of raw data, water district level, network level, impact assessment and evaluation of measures.
In Figure (2) is shown	Flow-sheet for the planning of an irrigation or reclamation scheme.
In Figure (3) is shown	Framework for analysis in water resources planning studies

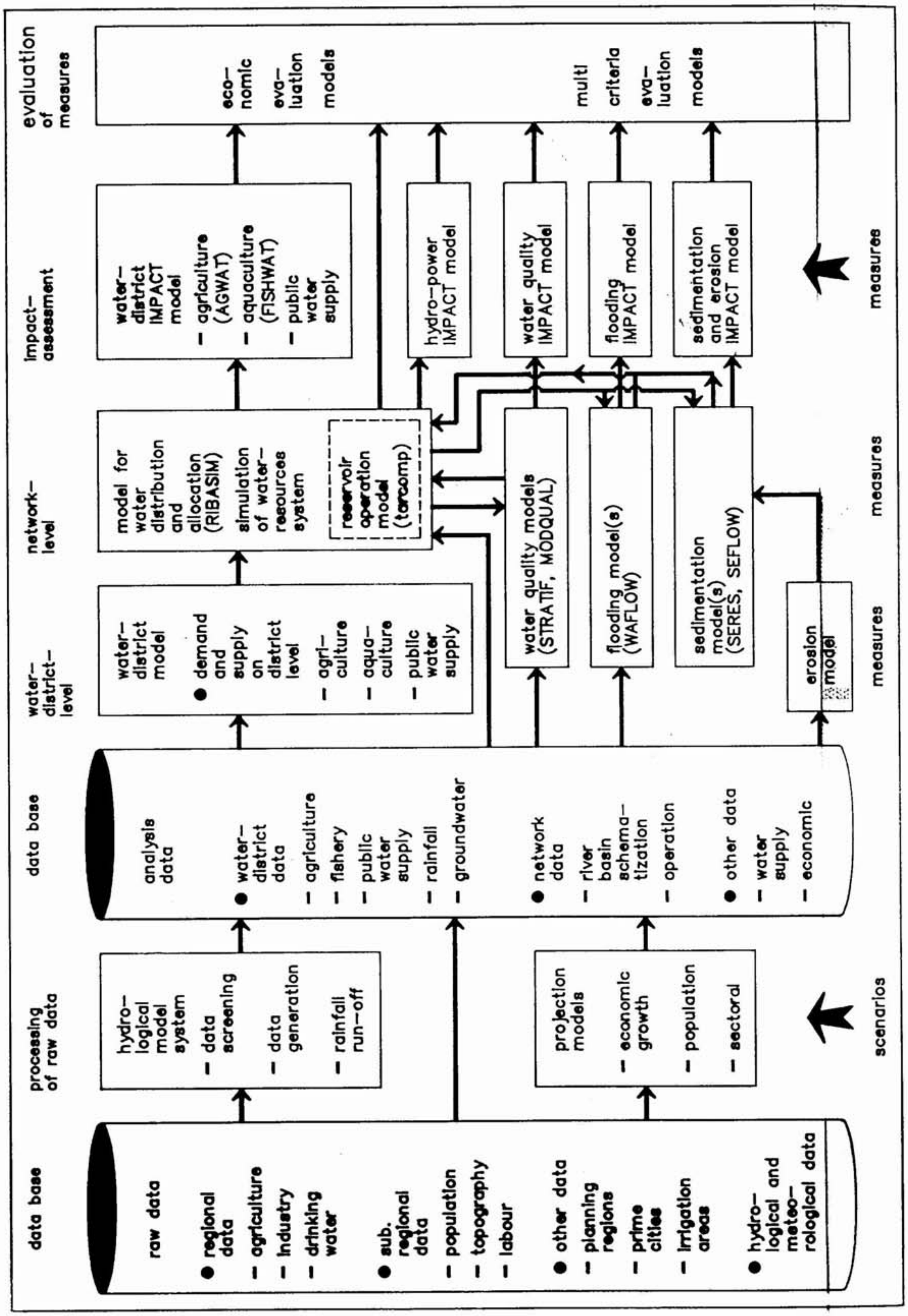
data base
raw data
● regional data
- agriculture
- industry
- drinking water
● sub. regional data
- population
- topography
- labour
● other data
- planning regions
- prime cities
- irrigation areas
● hydro-logical and meteo-rological data
processing of raw data
hydro-logical model system
- data screening
- data generation
- rainfall run-off
projection models
- economic growth
- population
- sectoral
scenarios
data base
analysis data
● water-district data
- agriculture
- fishery
- public water supply
- rainfall
- groundwater
● network data
- river basin schema-tization
- operation
● other data
- water supply
- economic
water-district-level
water-district model
● demand and supply on district level
- agri-culture
- aqua-culture
- public water supply
measures
erosion model
network-level
model for water distribution and allocation (RIBASIM)
simulation of water-resources system
reservoir operation model (tarcomp)
water quality models (STRATIF, MODQUAL)
flooding model(s) (WAFLOW)
sedimentation model(s) (SERES, SEFLOW)
measures
impact-assessment
water-district IMPACT model
- agriculture (AGWAT)
- aquaculture (FISHWAT)
- public water supply
hydro-power IMPACT model
water quality IMPACT model
flooding IMPACT model
sedimentation and erosion IMPACT model
measures
evaluation of measures
eco-nomic eva-luation models
multi criteria eva-luation models

Fig. 1

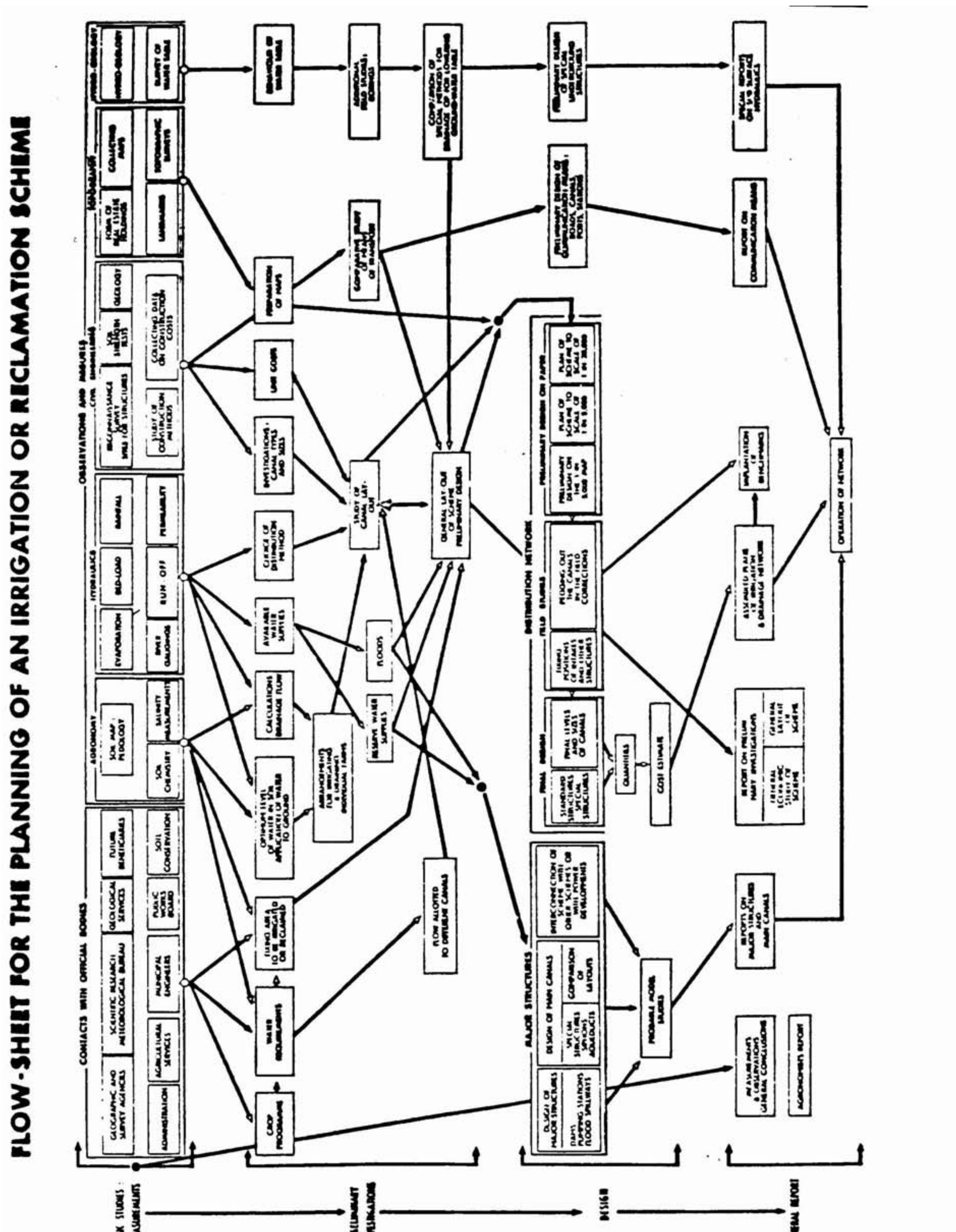
FLOW-SHEET FOR THE PLANNING OF AN IRRIGATION OR RECLAMATION SCHEME
CONTACTS WITH OFFICIAL BODIES
OBSERVATIONS AND MEASURES
AGRONOMY
HYDRAULICS
MAJOR STRUCTURES
DISTRIBUTION NETWORK
OPERATION OF NETWORK
BASIC STUDIES: MEASUREMENTS
PRELIMINARY INVESTIGATIONS
DESIGN
GENERAL REPORT

Fig. 2

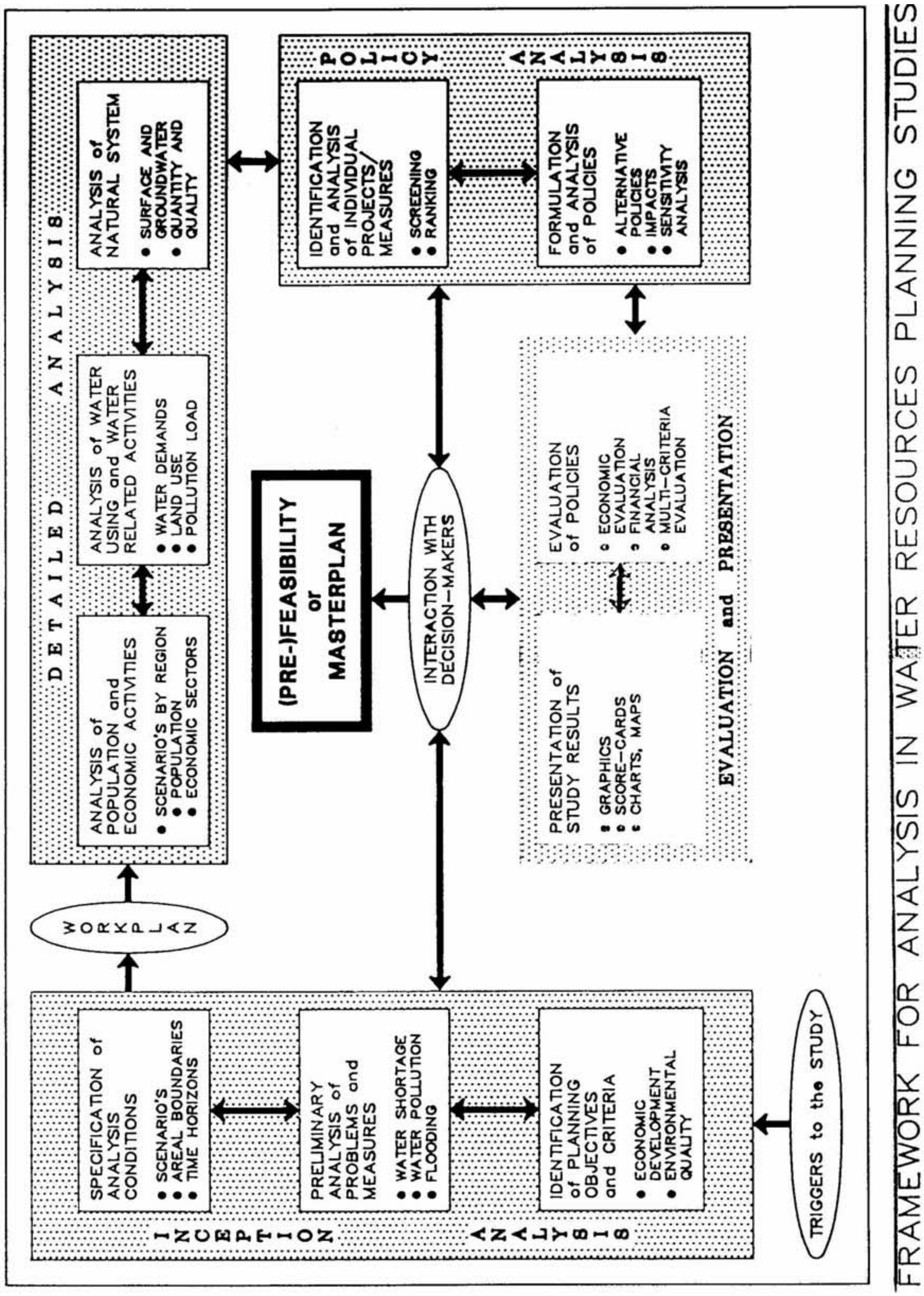
FRAMEWORK FOR ANALYSIS IN WATER RESOURCES PLANNING STUDIES
INCEPTION ANALYSIS
SPECIFICATION of ANALYSIS CONDITIONS
• SCENARIO'S
• AREAL BOUNDARIES
• TIME HORIZONS
PRELIMINARY ANALYSIS of PROBLEMS and MEASURES
• WATER SHORTAGE
• WATER POLLUTION
• FLOODING
IDENTIFICATION of PLANNING OBJECTIVES and CRITERIA
• ECONOMIC DEVELOPMENT
• ENVIRONMENTAL QUALITY
TRIGGERS to the STUDY
WORKPLAN
DETAILED ANALYSIS
ANALYSIS of POPULATION and ECONOMIC ACTIVITIES
• SCENARIO'S BY REGION
• POPULATION
• ECONOMIC SECTORS
ANALYSIS of WATER USING and WATER RELATED ACTIVITIES
• WATER DEMANDS
• LAND USE
• POLLUTION LOAD
ANALYSIS of NATURAL SYSTEM
• SURFACE AND GROUNDWATER
• QUANTITY AND QUALITY
(PRE-)FEASIBILITY or MASTERPLAN
INTERACTION WITH DECISION-MAKERS
POLICY ANALYSIS
IDENTIFICATION and ANALYSIS of INDIVIDUAL PROJECTS/ MEASURES
• SCREENING
• RANKING
FORMULATION and ANALYSIS of POLICIES
• ALTERNATIVE POLICIES
• IMPACTS
• SENSITIVITY ANALYSIS
EVALUATION and PRESENTATION
PRESENTATION of STUDY RESULTS
• GRAPHICS
• SCORE-CARDS
• CHARTS, MAPS
EVALUATION of POLICIES
• ECONOMIC EVALUATION
• FINANCIAL ANALYSIS
• MULTI-CRITERIA EVALUATION

Fig. 3

3 Site Characteristics, Available Information, Preliminary Site Investigations and Field Sutveys

The design of a system for a particular site should be begin with an evaluation of the site characteristics. A potential site for a system development will have a variety of system-related characteristics which are initially unknown and unquantified. The overall objective of the initial site evaluation is to observe and evaluate the characteristics of, and opportunities presented by, the site, as well as, conditions both on and offsite which affect the overall development. It is assumed for this discussion that the usual land study has already been conducted by the developer and that the following characteristics have already been evaluated: area of property and legal boundaries, zoning, availability of utilities, transportation, existing easements and rights-of-way, proximity of public facilities, soils and erosion potential and permit requirements.

3.1 Fixed Characteristics

3.1,1 Topography and Drainage

The overall topography of a site, which includes its shoreline as well as the large-scale rises and depressions, is the principal determinant in the irrigation and drainage of the site and limits the location of the system network. As part of the preliminary site survey, a working topographic map for the system design should be drawn up by the system designer. Information drawn on this map should include at least: property boundaries, vertical contour intervals, major physical features such existing systems, streams, unusual trees or vegetation, paths or roads, depressions, and extent and type of vegetation.

3.1,2 Hydrology and Water Resources

The system designer must make decisions that affect the supply, movement and quality of surface and groundwater on the site, and the interaction of these waters with water at neighboring sites on the boundaries of the property. It is therefore interested in locating, mapping and quantifying the amount and flow of surface water and groundwater resources.

3.1,3 Vegetation

The vegetation of an area invariably provides an indication of its soil characteristics and natural drainage conditions.

3.1,4 Soils

The system designer engineer needs to know the distribution of soil types on the site, because their widely-varying characteristics affect both drainage and construction decision.

3.2 Alterable Characteristics

They are drainage and pollution sources

3.3 Preliminary Site Investigations, Field Surveys and Instrumentation

Once the initial evaluation of site sharacteristics has been completed, the design objectives and constraints have been evaluated by the system designer, and there is an agreement between the owner and the designer to proceed, the next step in the design process is a preliminary site investigation-objectives, the monitoring and sampling problem and measurement requirements.

4 Sedimentation

There are many methods and techniques for trapping sediment. Smaller amounts of sediment are often trapped by slowing water flow with dams made of bales of straw, stone dams, sandbag dams, ditches, and small excavated pits. These are excellent methods of holding sediment close to points where sediment is likely to originate. In Fig. (4) is shown one type of sedimentation.

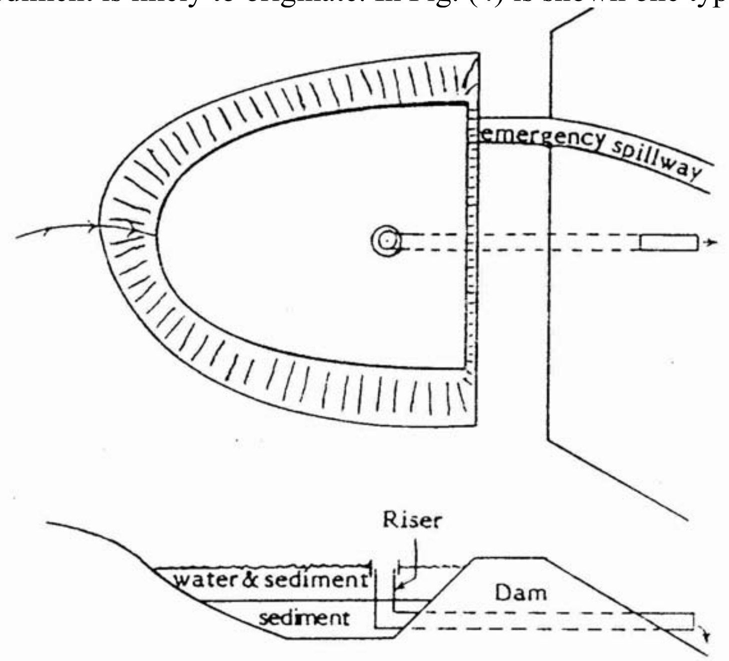

Fig 4-Top and side views of a pond

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Sulphate Reducing Bacteria in Groundwater Intakes in Gdansk Region

A. Wargin*, K. Olańczuk-Neyman*

INTRODUCTION

In the Gdańsk region some of the richest groundwater resources among hydrogeological units in Poland are found. This fact is own to superposition of aquifers present in well-developed and widely spread Cretaceous and Quaternary formations, morphological diversity of alimentation area and influence of the sea which is the basic draining reservoir of all aquifers, establishing flow directions. It is estimated that groundwater take off in the Three-city area amounts to about 400 000 m^3/d, out of which 12 000 m^3/d are Cretaceous waters.

Intensive take off as well as urbanisation and industrialisation of the area are the major reasons for substantial transformations of hydrogeological regime and deterioration of all aquifers quality.

Bacteriological control of groundwater quality includes only evaluation of pollution with faecal bacteria while specific groups of bacteria, characteristic of groundwater environment and in many cases responsible for deterioration of physical and chemical parameters of water are not examined. The following such groups of bacteria can be found in groundwater: denitrifying bacteria, manganese-oxidising bacteria, iron-binding bacteria and sulphate reducing bacteria.

In the study occurrence of sulphate reducing bacteria (SRB) in groundwater from Cretaceous and Quaternary formations in Gdańsk region was investigated. Specific requirements of SRB make groundwater in aquifer an adequate environment for their growth. Unfortunately, sulphur hydrogen is formed in SRB metabolitic processes, which results in deterioration of organoleptic features of groundwater. [Olańczuk-Neyman K., 2001]

It has been proved that SRB are present in raw water from Cretaceous and Quaternary formations and in treated water.

Because the SRB have negative influence over water quality (unacceptable odour), the treated water should be constantly or periodically disinfected. UV radiation, as well as, membrane filtration are the effective methods of eliminating the SRB from water.

THE STUDY AREA

Investigations were carried out at eight Cretaceous water intakes located at: the Marine Terrace ("Czarny Dwór" and "Zaspa"), Żuławy ("Lipce", "Krakowiec", "Pruszcz Gdański", "Sobieszewo"), at the edge of Kaszubian Lake District Upland ("Chełm") and at Quaternary-Tertiary water intake ("Reda II") located in the central part of Kaszubian Proglacial Stream Valley. Location of investigated water intakes is presented in the Fig. 1.

Additionally, water outflowing from water treatment plant processing Quaternary-Tertiary groundwaters from the intake "Reda II" was analysed. In the treatment plant traditional technological processes of aeration and filtration (at vertical filters filled with

* GDAŃSK UNIVERSITY OF TECHNOLOGY
Faculty of Hydro and Environmental Engineering
G. Narutowicza Str.11/12
80-952 Gdańsk, POLAND
phone: (48-58)347-27-63, 347-27-43, fax: (48-58)347-24-13
e-mail: awar@pg.gda.pl, kola @pg.gda.pl

quartz sand) are conducted. Cretaceous waters do not undergo treatment and are mixed with treated Quaternary waters.

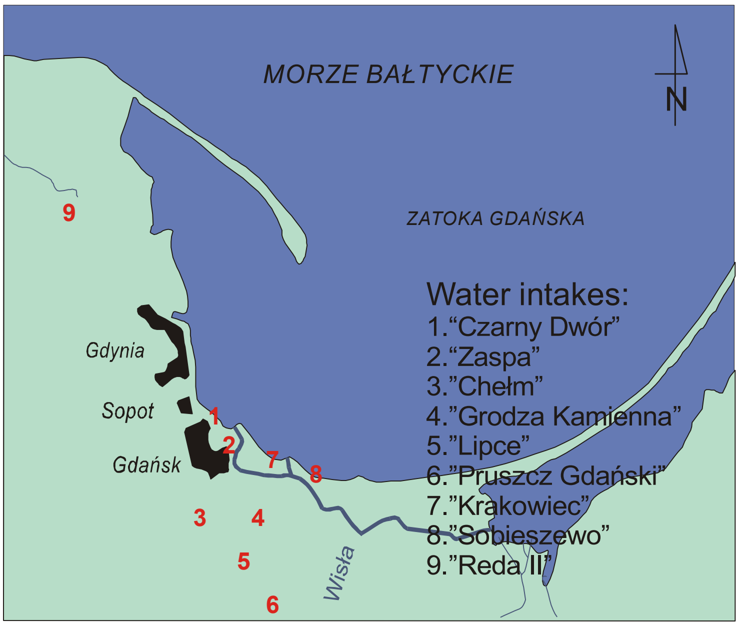

Fig. 1 Location of investigated groundwater intakes

Hydrogeological conditions

In the Gdańsk region the water-bearing system is formed by Cretaceous, sub-Tertiary and Quaternary aquifers. Schematic illustration of hydrogeological conditions is presented in the Fig.2. [Olańczuk-Neyman K., Pruszkowska M., Wargin A., 1998].

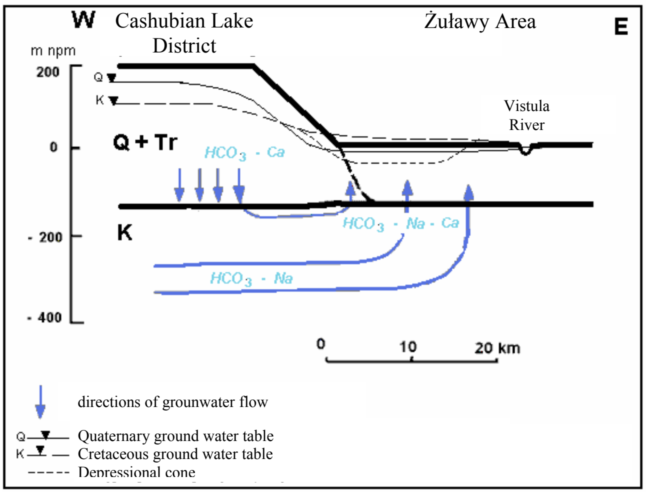

Fig.2 Schematic illustration of hydrogeological conditions

Alimentation of Gdańsk reservoir takes place in the central part of Kaszubian Lake District and it is of indirect type. Groundwaters flow down towards their draining areas - the Bay of Gdańsk and Żuławy. [Jesionek, Kwaterkiewicz, Sadurski, 1980].

In the Kaszubian Lake District waters of Quaternary and Tertiary waters participate in transitional flow. Some amount of water percolates to lying below Cretaceous aquifers, taking part in regional flow. Cretaceous multiaquifer formation plays fundamental role in regional circulation enabling migration of water from the Lake District to Żuławy and seaside

lowlands. Natural alimentation of Quaternary formation by means of ascension also takes place there.
Intensive take off resulted in considerable decrease of hydrostatic pressures in the 1990s. The static water table dropped even beneath the sea level. Moreover extensive regional depression cone was formed.
Tertiary aquifer's roof lies between 60.6 m below sea level (in the north-western side of "Reda II" water intake) to 69.2 m b.s.l. in the south-eastern side. Thickness of the aquifer varies from 5 to 25 m, decreasing in the south-eastern direction. The aquifer is alimented by lateral inflow from Kaszubian Lake District Upland.
The intake "Reda II" exploits waters of Quaternary origin. There is one Holocenic – Pleistocenic aquifer there, the thickness of which varies from 20 to 50 m. Only south-eastern part of the intake is located in the deep erosion valley where the thickness of fluvioglacial sandy sediments exceeds 100 m.
Groundwater table, free or confined by surface organic layers, lies from 0.5 to 2.0 m below ground surface. Groundwaters are alimented by direct seepage from the surface, waters flowing from the uplands and ascension from deeper aquifers.
Natural alimentation of Quaternary aquifer was changed due to intensive exploitation of the intake "Reda II". Formation of extensive depression cone caused draining off processes of subsurface and Holocenic waters. Therefore near the wells groundwater from deep aquifers is mixed with shallow waters from peats and silts, containing high concentrations of iron. [Wargin A., 2002].

METHODS

Samples of Cretaceous groundwater from 8 intakes were collected for two years from 1995 to 1996. Eighty five samples form the following wells: K1 at "Chełm", K1 at "Czarny Dwór", K2 at "Grodza Kamienna", K4 at "Krakowiec", K2 at "Lipce", K1 at "Pruszcz Gdański", K2 at "Sobieszewo" and K2 at "Zaspa" were examined. Samples of Quaternary groundwater were collected from March 2000 to July 2000 from 16 wells at the intake "Reda II" (no. A-1A,1B, P-2A, 2A, 3B, 4A, 5A, 6B, 7B, 8B, 9B, 10B, 11B, 12C, 14C, 16B). Altogether eighty samples of Quaternary water were examined. Samples of treated water were collected from the pipeline at water treatment plant "Reda II". Ninety four samples of treated water were collected.
The following physical and chemical parameters were analysed: odour, pH, hardness, COD_{Mn}, concentrations of total iron, manganese and sulphates. Odour was identified directly after the sample was collected.
Bacteriological analyses included determination of the most probable number (MPN) of sulphate reducing bacteria in 100 cm^3 of water on Starkey's liquid nutrient medium. Incubation at 22-25^{0}C lasted for 4 weeks.
Survival rate of SRB under UV radiation was investigated at water treatment plant "Reda II" using E-2 device made by Wedeco, equipped with 4 low-pressure Spectrotherm radiators. Effectiveness of radiation doses in the range from 13 mJ/cm^3 to 88 mJ/cm^3 was analysed.
Investigations of effectiveness of SRB removal from water by means of filtration were carried out at the pilot station type "Zee Weed" using hydrophilous membranes with pore size from 0.08 to 0.1 μm. Fifteen series of investigations, including raw water, treated water and concentrate were conducted.

RESULTS

Physical and chemical properties of groundwater

➢ **Cretaceous waters**

Groundwaters from the intakes: Pruszcz Gdański, Grodza Kamienna and Lipce are of hydrogen carbonate-calcium-sodium character. Waters from the other five intakes (Chełm, Czarny Dwór, Krakowiec, Sobieszewo and Zaspa) are hydrogen carbonate-sodium waters. These waters contain very low concentrations of oxygen and no free carbon dioxide. [Olańczuk-Neyman, Pruszkowska, Wargin, 1998]

Results of physical and chemical analyses of groundwaters are presented in Table 1.

Table 1. Results of physical and chemical analyses of raw Cretaceous groundwaters

	Parameter						
	Odour		Total hardness	COD_{Mn}	Iron	Manga-nese	Sulpha-tes
water intake	[-]	pH	[$mgCaCO_3/dm^3$]	[mgO_2/dm^3]	[$mgFe/dm^3$]	[$mgMn/dm^3$]	[$mgSO_4/dm^3$]
MAC*	Acceptable	9,5			0,2	0,05	250
Lipce K2	periodically unacceptable	7,4-8,4	105-215	0,5-3,3	0-0,70	0-0,1	8,3-24,1
Sobieszewo K2		8,1-8,5	30-55	1,0-2,3	0,03-0,70	0,005-0,09	20,2-35,9
Czarny Dwór K1		8,0-8,9	5-25	1,2-5,5	0-0,26	0-0,04	15,0-27,8
Krakowiec K4		8,0-8,6	15-70	1,6-3,9	0-0,28	0-0,04	19,0-32,8
Zaspa K2		7,1-8,8	13-160	0,8-3,1	0-0,32	0-0,05	11,0-43,1
Grodza Kamienna K2		7,1-8,6	65-135	0,8-2,9	0-0,32	0-0,05	11,0-23,9
Pruszcz Gdański K1		7,6-8,0	150-208	1,0-1,4	0-0,11	0-0,02	15,6-104,1
Chełm K1		7,6-8,5	100-175	1,2-6,5	0,05-2,50	0-0,22	10,2-14,4

*Maximum Admissible Concentration according to Polish Standards and European Drinking Water Directive (EWDW) 1980

Unacceptable odour of sulphur hydrogen was detected in 8-55% of all water samples (Chełm – 55%, Lipce – 39%, Czarny Dwór – 33%, Pruszcz – 33%, Zaspa – 32%, Sobieszewo – 30%, Krakowiec – 10%, Grodza Kamienna – 8%).

Only the concentrations of iron in three water intakes (“Lipce”, “Chełm”, “Grodza Kamienna”) did not meet acceptable value for drinking purposes (according to Polish Standards and EWDW) (tab.1).

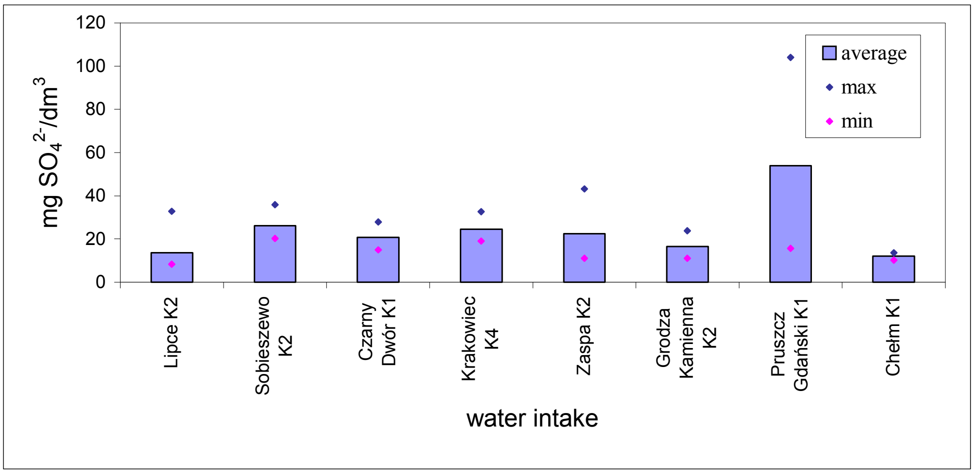

Fig.3 Average, minimal and maximal concentrations of sulphates in Cretaceous waters

The admissible concentration of sulphates in water (according to Polish Standards and European Drinking Water Directive 1980, Maximum Admissible Concentration MAC=250 mg SO_4/l) was not exceeded in any of investigated water intakes. Average, minimal and maximal concentrations of sulphates in Cretaceous waters are presented in the Fig. 3.

- **Quaternary and treated waters from water treatment plant "Reda II"**

During ten years of water exploitation (1989-1999) unacceptable odour of sulphur hydrogen was detected in waters from most Quaternary wells.

Results of physical and chemical analyses are presented in Table 2.

Table 2. Results of physical and chemical analyses of raw Quaternary waters

well	Parameter					
	Odour [-]	pH	COD_{Mn} [mgO_2/dm^3]	Iron [$mgFe/dm^3$]	Manganese [$mgMn/dm^3$]	Sulphates [$mgSO_4/dm^3$]
MAC*	Acceptable	9,5	5,0	0,2	0,05	250
A-1A	periodically unacceptable	7,4-7,7	2,5-3,8	0,88-2,0	0,20-0,3	19,3-79,8
1B		7,3-7,6	1,4-2,7	0,88-1,8	0,08-0,43	30,6-80,0
P-2A		7,1-7,7	0,2-3,2	0,66-2,0	0,09-0,22	22,2-72,1
2A		7,3-7,8	1,1-5,7	0,80-3,0	0,15-0,3	48,5-108,6
3B		7,0-7,7	1,8-6,5	0,80-1,8	0,10-0,5	32,5-98,3
4A		7,2-7,7	2,0-2,5	1,12-2,6	0,09-0,2	71,6-112,3
5A		7,0-7,8	1,4-4,2	0,87-2,0	0,10-0,5	25,5-249,0
6B		7,1-7,9	2,0-3,7	0,67-3,3	0,15-0,32	34,5-239,1
7B		6,9-7,7	2,4-2,9	0,68-1,8	0,15-0,29	36,5
8B		6,9-7,7	2,0-5,0	0,60-5,0	0,10-0,35	37,2-150,2
10B		7,0-7,7	2,2-3,7	0,78-2,2	0,09-0,32	94,2-130,9
11B		7,1-7,7	2,6-4,7	0,80-1,8	0,15-0,33	39,9-102,4
12C		7,1-7,3	2,1-3,4	1,60-2,6	0,20-0,25	99,0-119,6
14C		7,0-7,7	1,2-4,2	0,52-2,0	0,10-0,8	34,3-134,7
16B		6,9-7,7	1,3-3,5	0,58-1,8	0,11-0,25	16,1-72,6

*MAC - Maximum Admissible Concentration according to Polish Standards and European Drinking Water Directive 1980

Because concentrations of iron and manganese were to high for drinking purposes, water was treated using quartz sand filters.

The average concentrations of iron and manganese in Quaternary and treated waters are presented in the Fig. 4.

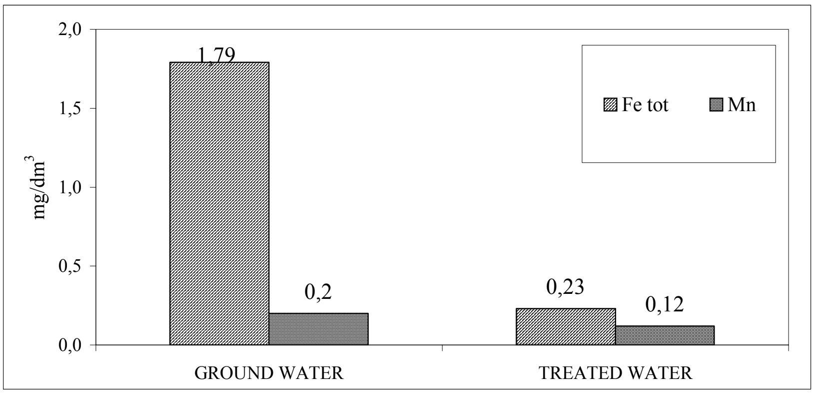

Fig. 4. The average concentrations of iron and manganese in raw Quaternary water and in mixed treated water from Reda II intake.

Sulphate reducing bacteria in waters

- **Cretaceous water**

Sulphate reducing bacteria were present in all of 8 investigated Cretaceous groundwater intakes (Fig. 5). The highest amount of samples containing sulphate reducing bacteria (67%) were collected at the intake Chełm (well K1). The maximal MPN of these bacteria was 18. In four intakes: Czarny Dwór (well K1), Zaspa (well K2), Grodza Kamienna (well K2) and Pruszcz Gdański (well K1) from 38 to 66% of water samples contained SRB and maximal MPN was from 6 to 16. In case of other intakes – Lipce (well K2), Sobieszewo (well K2) and Krakowiec (well K4) SRB were detected in 25-38% of analysed samples and MPN was from 2 to 3.

In all analysed water samples presence of SRB was accompanied with unacceptable sulphur hydrogen odour.

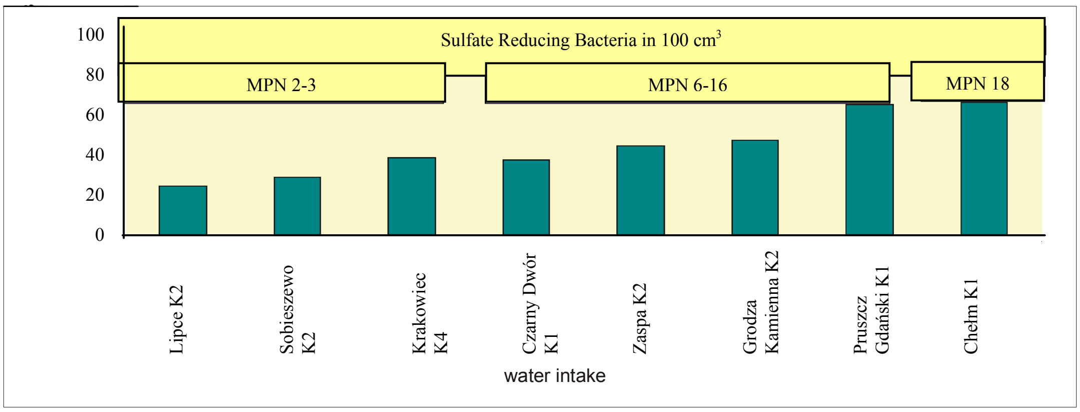

Fig. 5. Percentage share of Cretaceous water samples containing sulphate reducing bacteria (SRB)

- **Quaternary waters**

Sulphate reducing bacteria were detected in all of 16 investigated Quaternary wells at the intake "Reda II" (Fig. 6).

SRB were present in 100% of samples from three wells: no. P-2A, no. 3B and no. 6B; in 80% of samples from the wells no. A-1A and 4A and in 60% of samples from the wells no. 1B, 2A, 5A, 7B and 16B. In the case of six other investigated wells (no. 8B, 9B, 10B, 11B, 12C and 14C) SRB were detected in 50% of analysed samples or less.

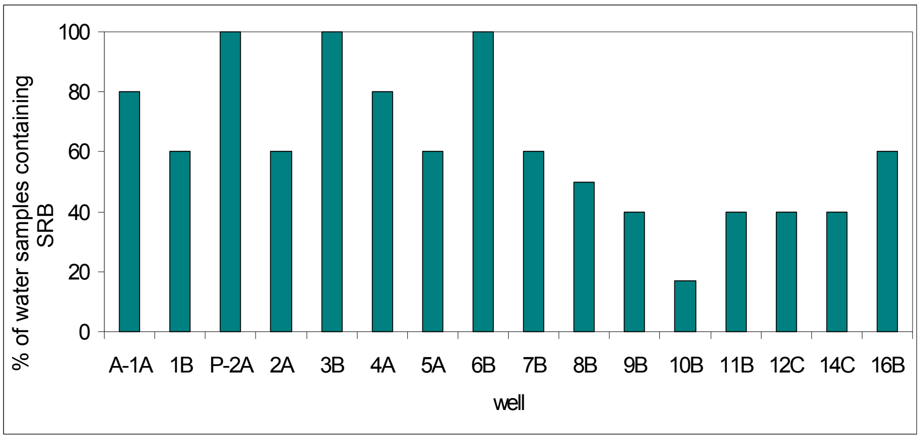

Fig. 6. Percentage share of Quaternary water samples containing sulphate reducing bacteria

- **Treated waters from water treatment plant "Reda II"**

In 73 (86%) samples of treated water from water treatment plant "Reda II" sulphate reducing bacteria were detected. This result indicates that SRB were almost constantly present in treated water. Since the number of these bacteria is relatively low (1/100 cm^3), there are some periods when they are undetectable.

Effectiveness of water desinfection with UV radiation

Results of investigations of elimination of SRB from treated Quaternary groundwaters by means of UV desinfection are presented in the Fig. 7.

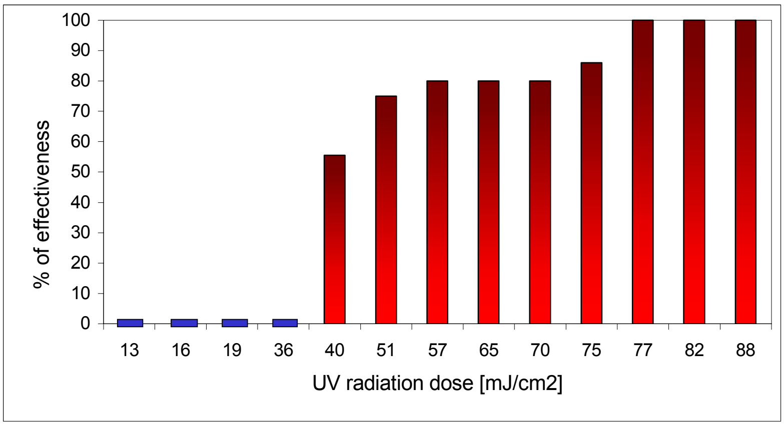

Fig. 7 Effectiveness of SRB elimination with various doses of UV radiation

Effectiveness of UV doses in the range from 13 to 88 mJ/cm^2 was analysed. It was found out that doses below 40 mJ/cm^2 were ineffective in SRB elimination. In doses range from 40 to 75 mJ/cm^2 effectiveness of SRB removal varied from 55 to 86%. The highest removal effectiveness (about 100%) was observed when UV doses above 77 mJ/cm^2 were applied.

Effectiveness of microfiltration

Results of investigations of SRB elimination from treated Quaternary groundwaters by means of membrane filtration are presented in the Fig. 8.

SRB were present in 66.7% of raw water samples and in 93.3% of concentrate samples. The samples of water after microfiltration (permeat) did not contain SRB.

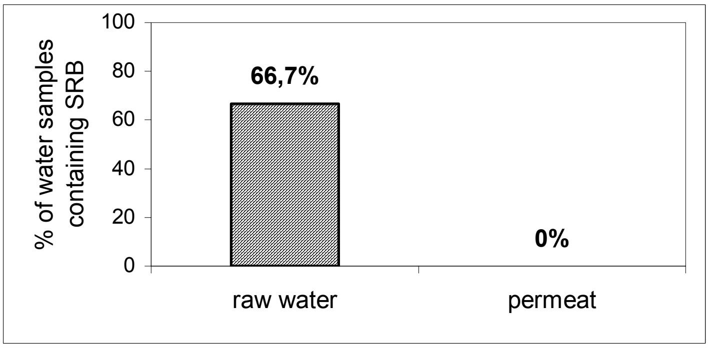

Fig. 8 Occurrence of SRB in groundwater before and after membrane filtration

CONCLUSIONS

The objective of bacteriological investigations conducted during the study was evaluation of the number and frequency of sulphate reducing bacteria (SRB) occurrence in waters originating from both multiaquifer formations. It was indicated that SRB are commonly found in raw water from either Cretaceous or Quaternary aquifers. The number of SRB did not exceed twenty in 100 ml. Usually presence of SRB was accompanied with sulphur hydrogen odour.

Formation of sulphur hydrogen in groundwater is characteristic for anaerobic environments where adequate conditions for growth of sulphate reducing bacteria occur. SRB growth in the aquifer results in negative changes of water quality - deterioration of organoleptic properties due to production of sulphur compounds (sulphur hydrogen).

The changes of quality of groundwater in the Gdansk region, which begun in 1980s, most probably result from intensive water exploitation and periodical activation of SRB.

Moreover SRB are present in treated waters after aeration and filtration on quartz sand beds. Since SRB cause deterioration of distributed water quality and can negatively affect water-pipe network, treated waters need to be constantly or periodically desinfected.

In the study it was proved that UV radiation is an effective method of SRB elimination if the UV dose is higher than 77 mJ/cm^2. This dose is almost twice as high as recommended for conventional drinking water desinfection. High effectiveness of membrane filtration in SRB elimination was also indicated.

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Water Hammer Analysis in Pipe Networks by the Method of Characteristics (MOC)

Roman Wichowski

Faculty of Civil and Environmental Engineering, Technical University of Gdańsk,
ul. Narutowicz`a 11/12, 80-952 Gdańsk, Poland, rwich@pg.gda.pl

Abstract

The paper presents method of analyzing of water hammer phenomenon in pipe networks. A mathematical model is presented by a set of partial differential equations of hyperbolic type, which have been transformed by the method of characteristics into total differential equations, i.e. compatibility equations on appropriate characteristics. Subsequently, the above-mentioned equations have been transformed into approximate difference equations in presentation of so-called predictor-corrector method.

The main part of the paper is the application of appropriate equations in a difference form describing the unsteady flow phenomenon in complex pipeline systems, i.e. equations for nodes with branching or connecting pipelines. Such complex nodes can be found in various types of water supply systems, and also in district heating or industrial networks.

One calculation example is given related to the complex water-pipe network consisting of 17 real loops, 47 pipelines and 33 nodes, supplied by two independent sources. Water-hammer throughout the whole pipeline network were caused by closing the gate valve at midpoint of one selected pipe. The results of the numerical calculations are presented in a graphic form with respect to the final cross-sections of pipes.

1 Introduction

The problem of *unsteady*, or *instationary flows* in closed pipelines under pressure belongs to one of the more complex phenomena which occur in hydromechanics. One of the most common reasons of the occurrence of this phenomenon is a sudden closing or opening of liquid flowing in a pipeline by means of various kinds of valves as well as planned or accidental starting or stopping of pumps in water supply systems.

The main purpose of this paper is to put forward the method of analysis of transients flows in water supply networks. This subject-matter has been dealt with only by several foreign scientific papers, mainly in the USA and Canada, cf. Streeter (1967), Karney and McInnis (1992), McInnis and Karney (1995). Particularly interesting is the paper of McInnis and Karney [6], where computerized transient-flow models have been used in the analysis of water-hammer events in topologically simple pipeline systems, and the performance of these models is well documented. The result of field test conducted on one of the city of Calgary major transmission and distribution subsystems is presented.

The paper includes equations to be applied to more complex pipeline systems, i.e. water-pipe network with distributing and connecting conduits. So-called internal boundary conditions, that is junctions or nodes, where several pipes of positive and several pipes of negative flow direction converge, are discussed. Open water supply systems (radial network) and closed systems (looped network) consist of such partial junctions. One computational example related to the water-pipe network is given in the paper. The example refers to the analysis of a medium size water-pipe network consisting of 17 real loops, 48 pipelines and 33 junctions (nodes). The unsteady flows in this network were caused by closing the gate valve at midpoint of pipe no. 9.

Notation

a - velocity of pressure wave propagation, m/s;
A - cross-sectional area of a pipe, m^2;
D - internal diameter of a pipe, m;
E - modulus of elasticity of pipe material, Pa;
E_c - bulk modulus of fluid, Pa;
g - acceleration of gravity, m/s^2;
H - pressure head, m;
k - absolute roughness of a pipe, m;
L - length of a pipe, m;
m - discharge exponent in formula for friction losses;
N - number of calculation sections of a pipe;
Q - volume discharge of a liquid, m^3/s;
R_o - elementary pipe resistance coefficient, s^2/m^6;
R - coefficient of total loss of head in a pipeline, s^2/m^5;
Re - Reynolds number;
s - thickness of pipe wall, m;
t - time, s;
T_c - time of the complete closure of the valve or gate valve, s;
w - mean flow velocity, m/s;
x - abscissa along the pipeline axis, m;
Z - characteristic pipeline impedance, s/m^2;
λ - coefficient of linear friction resistance;
ρ - density of the liquid, kg/m^3.

2 Solution of unsteady flows by the method of characteristics

The mathematical model of the unsteady flows of compressible liquid in elastic pipes is expressed by ***the set of two partial differential equations of the first order of hyperbolic type, i.e. a momentum equation (1), and a relation of mass conservation (2).*** To solve these equations advantage is commonly taken ***the method of characteristics.***

2.1 Equations of characteristics

For further analysis let us take down the equations describing the unsteady flow phenomenon, which have the following form, cf. Chaudhry (1979), Evangelisti (1969), Fox (1977), Streeter (1967, 1972), Szymkiewicz (1975), Wichowski (1999, 2002):

$$\frac{\partial H}{\partial x} + \frac{1}{gA}\frac{\partial Q}{\partial t} + R_0 |Q|^m \ \mathrm{sgn}\ Q = 0 \qquad (1)$$

$$\frac{\partial H}{\partial t} + \frac{a^2}{gA}\frac{\partial Q}{\partial x} = 0 \qquad (2)$$

The first equation is the momentum equation stating the dynamical equilibrium of the liquid particles in the cross-section of pipe, the second is the continuity equation derived from the mass conservation of the elastic fluid particles during their flow through an elastic pipe.

The paper presents a solution of the set of equations (1) and (2) by taking advantage of the characteristics method. At first it is necessary to transform them into suitable ordinary differential equations, referred to as compatibility equations on appropriate characteristics:

- compatibility equation on C_+ characteristic:

$$dH + \frac{a}{gA}\, dQ + R_0 |Q|^m \, sgn\, Q\, dx = 0 \qquad (3)$$

- compatibility equation on C_- characteristic:

$$dH - \frac{a}{gA} dQ + R_0 |Q|^m \, \mathrm{sgn}\ Q\, dx = 0 \qquad (4)$$

For the numerical calculation an iterative procedure, known as ***predictor-corrector method*** has been applied (Evangelisti, 1969, Streeter, 1972, Wichowski, 1999, 2002). Equations (3) and (4) are ordinary differential equations and they will be substituted by ***approximated difference equations***, where finite increments Δx and Δt of independent variables x and t are used.

2.2 Equations for the internal points of the grid of characteristics

In ***the predictor-corrector method*** the corrected values of the unknows, i.e. the pressure head H(x,t), and the volume discharge of the liquid Q(x,t), or velocity w(x,t) are calculated on the basis of the mean resistance of an elementary pipeline section related to the flow or velocity value of the proceeding and the following time steps. Making use of compatibility equations (3) and (4) we can write down the approximate difference equations for the internal points of the grid of characteristics (Fig. 1):

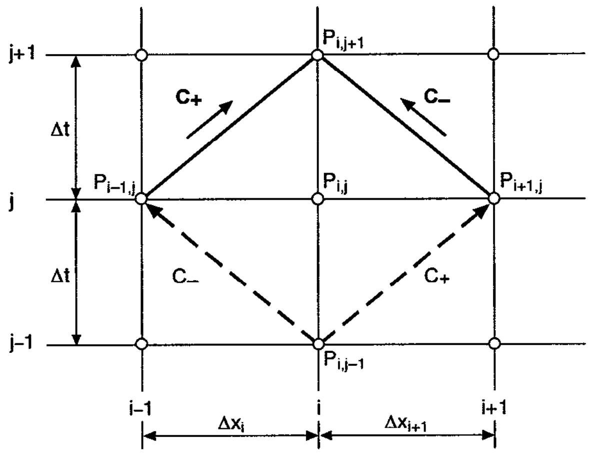

Fig. 1. Elementary mesh of characteristics grid for internal points

Predictor

C_+ compatibility equation:

$$H^{p}_{i,j+1} - H_{i-1,j} + Z_{i,j}\left(Q^{p}_{i,j+1} - Q_{i-1,j}\right) + R_{i,j}\left|Q_{i-1,j}\right|^{m} \operatorname{sgn} Q_{i-1,j} = 0 \tag{5}$$

C₋ compatibility equation:

$$H^{p}_{i,j+1} - H_{i+1,j} - Z_{i+1,j}\left(Q^{p}_{i,j+1} - Q_{i+1,j}\right) - R_{i+1,j}\left|Q_{i+1,j}\right|^{m} \operatorname{sgn} Q_{i+1,j} = 0 \tag{6}$$

Corrector

C_+ compatibility equation:

$$\begin{aligned} &H_{i,j+1} - H_{i-1,j} + Z_{i,j}\left(Q_{i,j+1} - Q_{i-1,j}\right) + \\ &+ 0.5\, R_{i,j}\left[\left|Q_{i-1,j}\right|^{m} \operatorname{sgn} Q_{i-1,j} + \left|Q^{p}_{i,j+1}\right|^{m} \operatorname{sgn} Q^{p}_{i,j+1}\right] = 0 \end{aligned} \tag{7}$$

C_- compatibility equation:

$$\begin{aligned} &H_{i,j+1} - H_{i+1,j} - Z_{i+1,j}\left(Q_{i,j+1} - Q_{i+1,j}\right) + \\ &- 0.5\, R_{i+1,j}\left[\left|Q_{i+1,j}\right|^{m} \operatorname{sgn} Q_{i+1,j} + \left|Q^{p}_{i,j+1}\right| \operatorname{sgn} Q^{p}_{i,j+1}\right] = 0 \end{aligned} \tag{8}$$

In the above equations R is ***the hydraulic resistance*** of the pipe or its section while value Z is the so-called ***characteristic impedance*** of pipeline expressed by the formula:

$$Z = \frac{a}{gA} \tag{9}$$

Having the approximated flow value at internal point $P_{i,j+1}$, it is now possible to find the corrected value of flow:

$$Q_{i,j+1} = Q^{P}_{i,j+1} + \Delta Q \tag{10}$$

where: ΔQ is the value of the correction, derived on the basis of Eqs. (5)-(8).

2.3 Equations for boundary points of the grid of characteristics

Examination of the grid of characteristics shows that the boundary points of the system being influencing the interior points after the first time step Δt. Therefore, in order to complete the solution to any desired time, it is necessary to include the boundary conditions.

At either end of a single pipe only one of the compatibility equations is available. For the left end (Fig.2a) equation (4) along the C_- characteristic is valid, and for the right end (Fig.2b) equation (4) along the C_+ characteristic is valid.

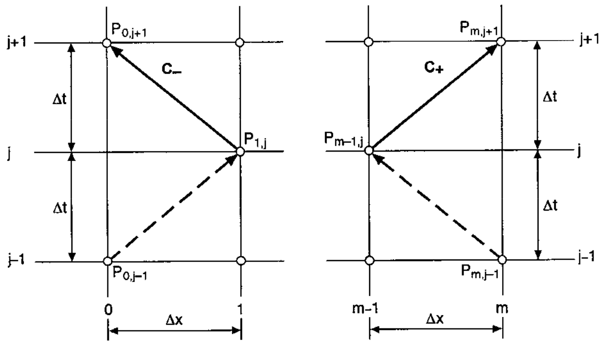

Fig. 2. Elementary meshes of grid of characteristics for boundary points:
a) left end; b) right end

Left end (upstream end) (Fig. 2a):

Predictor

C_- compatibility equation:

$$H^p_{0,j+1} - H_{1,j} - Z_{1,j}\,(Q^p_{0,j+1} - Q_{1,j}) - R_{1,j}\,\left|Q_{1,j}\right|^m \operatorname{sgn} Q_{1,j} = 0 \qquad (11)$$

where: $H^p_{0,j+1} = H(x,t)$ or $Q^p_{0,j+1} = Q(x,t)$.

Corrector

C_- compatibility equation:

$$\begin{aligned} &H_{0,j+1} - H_{1,j} - Z_{1,j}\left(Q_{0,j+1} - Q_{1,j}\right) + \\ &- 0.5R_{1,j}\left(\left|Q_{1,j}\right|^m \operatorname{sgn} Q_{1,j} + \left|Q^p_{0,j+1}\right|^m \operatorname{sgn} Q^P_{0,j+1}\right) = 0 \end{aligned} \qquad (12)$$

where: $H_{0,j+1} = H(x,t)$ or $Q_{0,j+1} = Q(x,t)$.

Right end (downstream end) (Fig. 2b):

Predictor

C_+ compatibility equation:

$$H^p_{m,j+1} - H_{m-1,j} + Z_{m,j}\,(Q^p_{m,j+1} - Q_{m-1,j}) + R_{m,j}\,\left|Q_{m-1,j}\right|^m \operatorname{sgn} Q_{m-1,j} = 0 \qquad (13)$$

where: $H^p_{m,j+1} = H(x,t)$ or $Q^p_{m,j+1} = Q(x,t)$.

Corrector

C_+ compatibility equation:

$$\begin{aligned} &H_{m,j+1} - H_{m-1,j} + Z_{m,j}\left(Q_{m,j+1} - Q_{m-1,j}\right) + \\ &+ 0.5\,R_{m,j}\left(\left|Q_{m-1,j}\right|^m \operatorname{sgn} Q_{m-1,j} + \left|Q^p_{m,j+1}\right|^m \operatorname{sgn} Q^p_{m,j+1}\right) = 0 \end{aligned} \qquad (14)$$

where: $H_{m,j+1} = H(x,t)$ or $Q_{m,j+1} = Q(x,t)$.

3 Analysis of unsteady flows in water-pipe networks

In the water supply systems, as well as in district heating networks and industrial complex pipelines systems are used generally branching pipes, i.e. forking and connecting pipes. Systems of this kind occur both in distributed and looped pipe networks, which consist of single transit pipelines, water mains, and distributing pipes.

The junction points or nodes in systems of branching pipes are dealt with like appropriate internal boundary points. Thus, each junction point or node W (Fig. 3) is right end boundary with respect to an elementary section or sections of pipelines on its, or their, left side and simultaneously a left end boundary for an elementary section or sections of pipelines on the right side of that point.

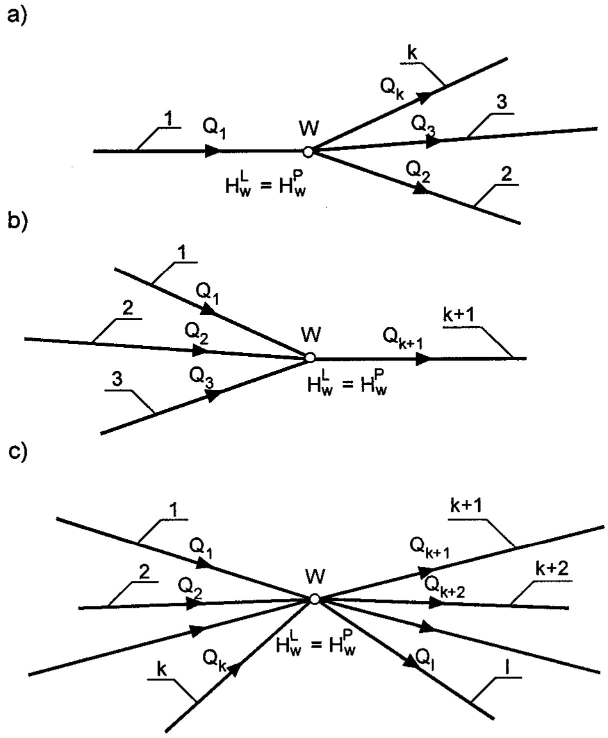

Fig. 3. Examples of pipelines with different nodes: a) node with forking pipelines; b) node with connecting pipes; c) complex node

In all cases the continuity equation, i.e. equilibrium condition of the sums of inflows to the junction and the sums of outflows from the junction, is utilized as the boundary condition:

$$\sum Q_{\text{inflows}} = \sum Q_{\text{outflows}} \tag{15}$$

It is additionally assumed that the pressure heads in the neighbourhood of point W are equal:

$$H_w^l = H_w^r \tag{16}$$

Supplementary equations are the compatibility equations with respect to appropriate characteristics depending on the case under consideration. Adequate equations for commonly applied branching junctions in complex pipeline systems will be presented below.

For the junctions of forking pipelines (Fig. 3a) the following compatibility equations can be written:

$$\begin{aligned}
&H_{m1,j+1}^{(1)} - H_{m1-1,j}^{(1)} + Z_{m1,j}^{(1)} \left(Q_{m1,j+1}^{(1)} - Q_{m1-1,j}^{(1)}\right) + R_{m1,j}^{(1)} \left|Q_{m1-1,j}^{(1)}\right|^m \operatorname{sgn} Q_{m1-1,j}^{(1)} = 0 \\
&H_{0,j+1}^{(2)} - H_{1,j}^{(2)} - Z_{1,j}^{(2)} \left(Q_{0,j+1}^{(2)} - Q_{1,j}^{(2)}\right) - R_{1,j}^{(2)} \left|Q_{1,j}^{(2)}\right|^m \operatorname{sgn} Q_{1,j}^{(2)} = 0 \\
&\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots \\
&H_{0,j+1}^{(k)} - H_{1,j}^{(k)} - Z_{1,j}^{(k)} \left(Q_{0,j+1}^{(k)} - Q_{1,j}^{(k)}\right) - R_{1,j}^{(k)} \left|Q_{1,j}^{(k)}\right|^m \operatorname{sgn} Q_{1,j}^{(k)} = 0
\end{aligned} \tag{17}$$

where: $H^{(1)}_{m1,j+1} = H^{(2)}_{0,j+1} = H^{(3)}_{0,j+1} = \cdots = H^{(k)}_{0,j+1} = H_{j+1}$

and the continuity equation:

$$Q^{(1)}_{m1,j+1} = Q^{(2)}_{0,j+1} + Q^{(3)}_{0,j+1} + \cdots + Q^{(k)}_{0,j+1} \qquad (18)$$

where superscripts (1), (2), ..., (k) denote the successive number of the pipeline.

Now let us take the following denotations:

$$\begin{aligned}
S^{(1)} &= -H^{(1)}_{m1-1,j} - Z^{(1)}_{m1,j}\, Q^{(1)}_{m1-1,j} + R^{(1)}_{m1,j} \left|Q^{(1)}_{m1-1,j}\right|^m sgn\, Q^{(1)}_{m1-1,j} \\
S^{(2)} &= -H^{(2)}_{1,j} + Z^{(2)}_{1,j}\, Q^{(2)}_{1,j} - R^{(2)}_{1,j} \left|Q^{(2)}_{1,j}\right|^m sgn\, Q^{(2)}_{1,j} \\
&\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots \qquad (19) \\
S^{(k)} &= -H^{(k)}_{1,j} + Z^{(k)}_{1,j}\, Q^{(k)}_{1,j} - R^{(k)}_{1,j} \left|Q^{(k)}_{1,j}\right|^m sgn\, Q^{(k)}_{1,j}
\end{aligned}$$

Making use of the denotations and equations (17) and (18), the following equations we obtained:

$$\begin{aligned}
&H_{j+1} + Z^{(1)}_{m1,j}\, Q^{(1)}_{m1,j+1} + S^{(1)} = 0 \\
&H_{j+1} - Z^{(2)}_{1,j}\, Q^{(2)}_{0,j+1} + S^{(2)} = 0 \\
&\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots \qquad (20) \\
&H_{j+1} - Z^{(k)}_{1,j}\, Q^{(k)}_{0,j+1} + S^{(k)} = 0.
\end{aligned}$$

Taking into consideration the continuity equation (18), as well as the compatibility equation (17), the following set of (k+1) equations with (k+1) unknown values is derived:

$$H_{j+1} = \frac{\dfrac{S^{(1)}}{Z^{(1)}_{m1,j}} + \dfrac{S^{(2)}}{Z^{(2)}_{1,j}} + \dfrac{S^{(3)}}{Z^{(3)}_{1,j}} + \cdots + \dfrac{S^{(k)}}{Z^{(k)}_{1,j}}}{\dfrac{1}{Z^{(1)}_{m1,j}} + \dfrac{1}{Z^{(2)}_{1,j}} + \dfrac{1}{Z^{(3)}_{1,j}} + \cdots + \dfrac{1}{Z^{(k)}_{1,j}}} \qquad (21)$$

$$\begin{aligned}
Q^{(1)}_{j+1} &= -\frac{H_{j+1} + S^{(1)}}{Z^{(1)}_{m1,j}} \\
Q^{(2)}_{j+1} &= \frac{H_{j+1} + S^{(2)}}{Z^{(2)}_{1,j}} \qquad (22) \\
&\cdots\cdots\cdots\cdots \\
Q^{(k)}_{j+1} &= \frac{H_{j+1} + S^{(k)}}{Z^{(1)}_{1,j}}
\end{aligned}$$

In the same way we can write appropriate equations for junctions with connecting pipelines and for complex junctions where several pipelines of positive and several pipelines of negative flow direction meet in the node.

4 Spatial distribution of pressure in water supply looped networks

Spatial distribution of pressure in water supply looped networks is illustrated by a case related to the analysis of a middle-sized water supply network in unsteady conditions. The network consists of 17 real loops, 47 pipelines and 33 nodes, supplied by two independent sources, i.e. from the upper water reservoirs (Fig. 4).

When designing the water supply networks it is necessary to take into account of the occurrence of some significantly high pressures that can be dangerous to the pipelines and appurtenances, e.g. hydrants, valves and fittings. The pressures are caused by instantaneous closure of the fluid flow at the end of any pipes, or due to failure of power to pump.

The calculations of water supply network in steady state conditions were carried out using of EPANET 2 software elaborated by a group of research workers headed by Lewis Rossman from the Water Supply and Water Resources Division, National Risk Management Research Laboratory, U. S. Environmental Protection Agency, Cincinnati (Rossman, 2000).

In the calculations of the looped network in steady states advantage was taken of input data as presented by the scheme of pipe network presented in Figure 4. The data include the number of the pipe, the length of the pipe section, and the nominal diameter, as well as information related to the nodes, as for instance, the node number, the external flow from the node, and the elevation of the node. It was assumed that the looped network would be made of cast iron flange pipes compatible with the Polish Standard PN-68/H-74101.

The calculations of the looped network in unsteady conditions were made using the WHAMMER computer program written in FORTRAN language based on the calculation algorithm presented by the author in the paper [12]. The unsteady flows in the network of Fig. 4 were caused by closing of the ball valve in time T_c = 5 s, installed in the midpoint of pipe no. 9. The pipe of nominal diameter DN 400 and the total length L = 540 m is located in the vicinity of the upper reservoir supplying node no. 8 with water. The calculation results are presented in Figs. 5 to 7.

The analysis of the obtained results indicates that the largest pressure increments occur in the middle of the pipe before the ball valve on the side of node no. 8 (Fig. 4). The maximum pressure head will reach 207.4 m of H_2O, while the minimum one will be 4.6 m of H_2O (Fig. 5). For this reason the higher pressure will be followed by pressure drop of 202.8 m, which is a magnitude twice bigger than the permissible working pressure for cast iron flange pipes of LA type according to Polish Standard PN-68/H-74101. The maximal value of the high pressure is in the permissible pressure range which in the case of cast iron flange pipes amounts to 2.5 MPa, that is about 250 m of H_2O.

The increment of pressure to the maximal value will occur in this case in pipe 9 after the valve has been completely closed, i.e. after 5 s. Further pressure oscillations are characterized by an evident reduction of the extreme values and in fact after 30 s the pressure heads will be close to the steady state value being approximately 70 m of H_2O. The maximum pressure head at the end of pipe no. 8 connecting reservoir 32 with the network is 156 m of H_2O, and the minimum value of the reduced pressure is 18.7 m of H_2O. However, the pressure oscillations around the steady state pressure amounting to approximately 60 m of H_2O undergo a relatively fast damping and after 12 s have no significance taking the pipeline strength into consideration.

An examination of the extreme pressure heads in the remaining pipes outside pipe no. 9 where the flow disturbance occurred, has indicated that in this case we are dealing with remarkably large pressures in phases of higher values exceeding the permissible ones for cast iron pipes of ordinary wall thickness in 3 extra pipes, i.e. pipe no. 7 (node 7), pipe no. 46 (node 29), and in pipe no. 41 (node 27).

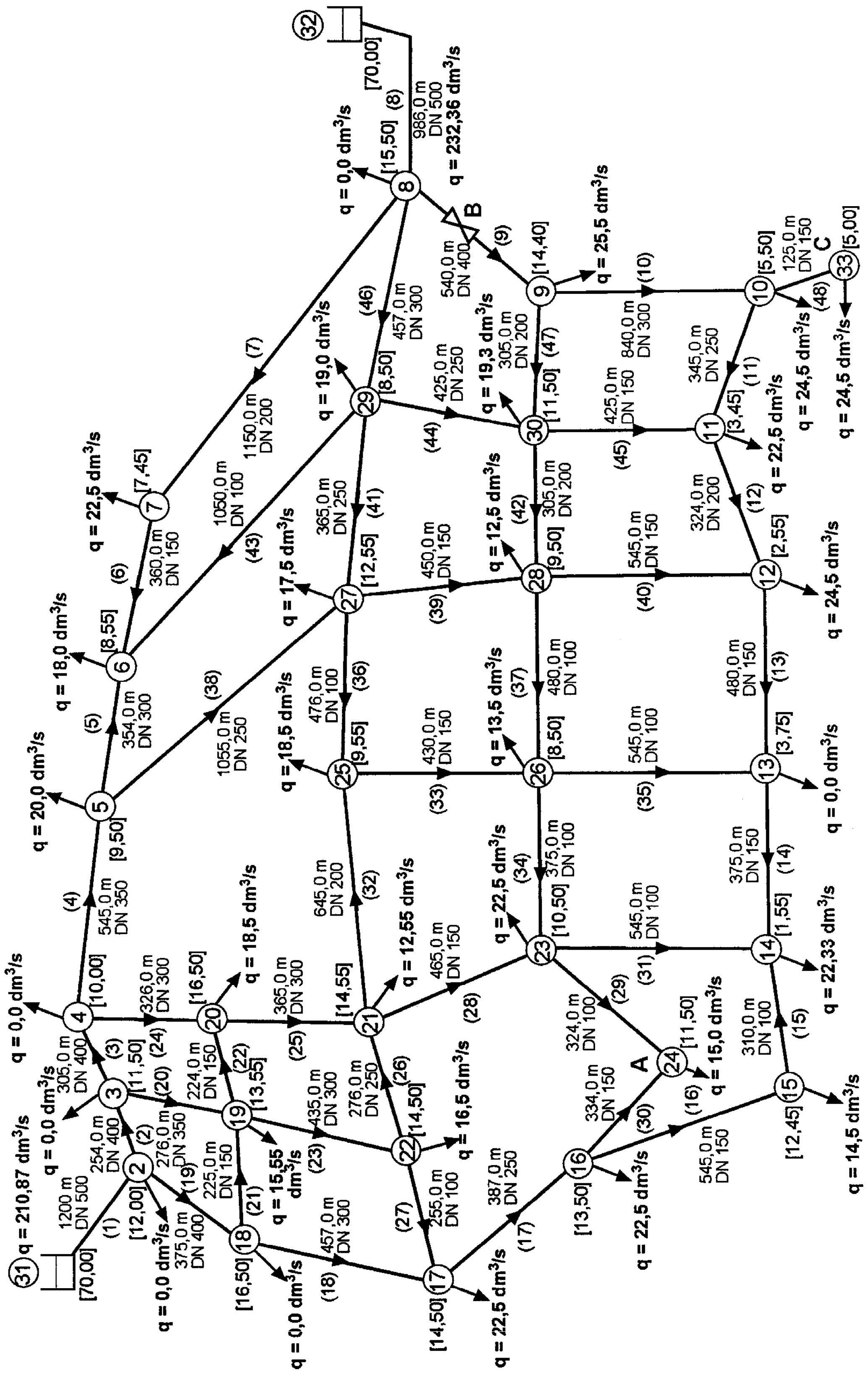

Fig. 4. Scheme of water-pipe network for the Example: 47 pipes, 17 real loops, 33 nodes

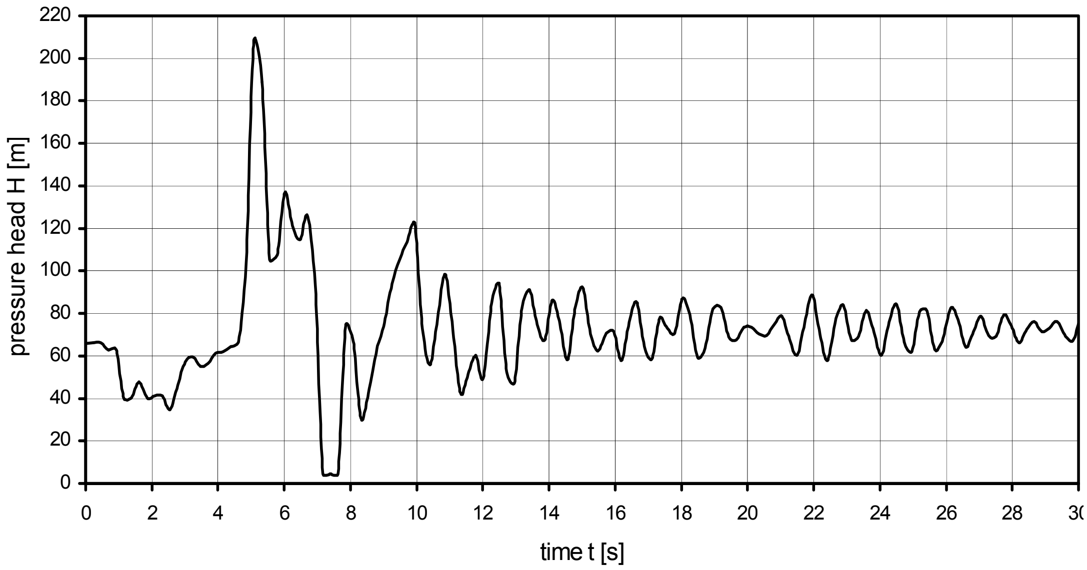

Fig. 5. Pressure head oscillations at midpoint of pipeline no. 9 (node no. 1)

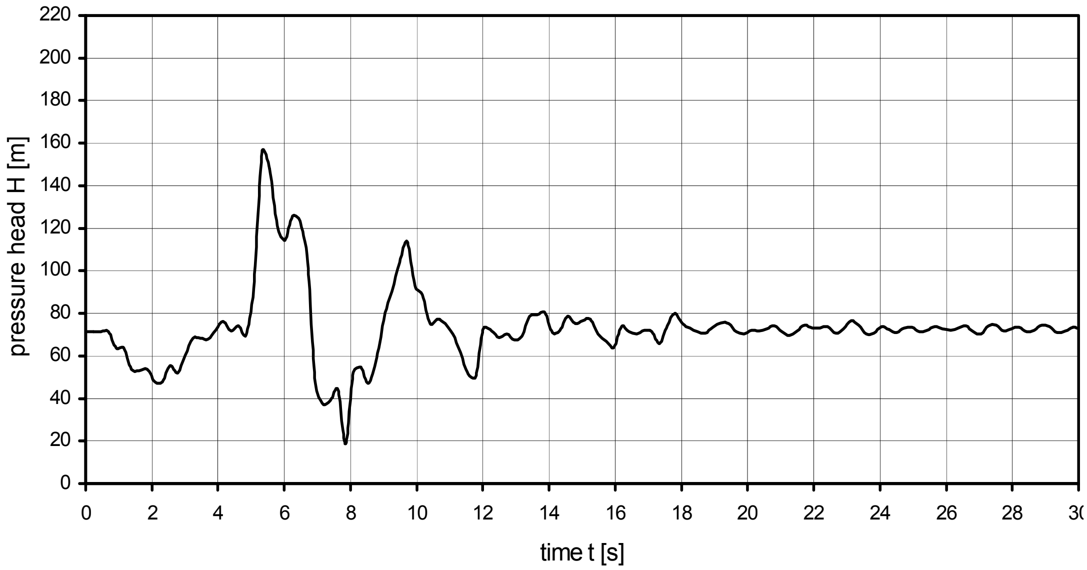

Fig. 6. Pressure head oscillations at the final cross-section of pipeline no. 8 (node no. 8)

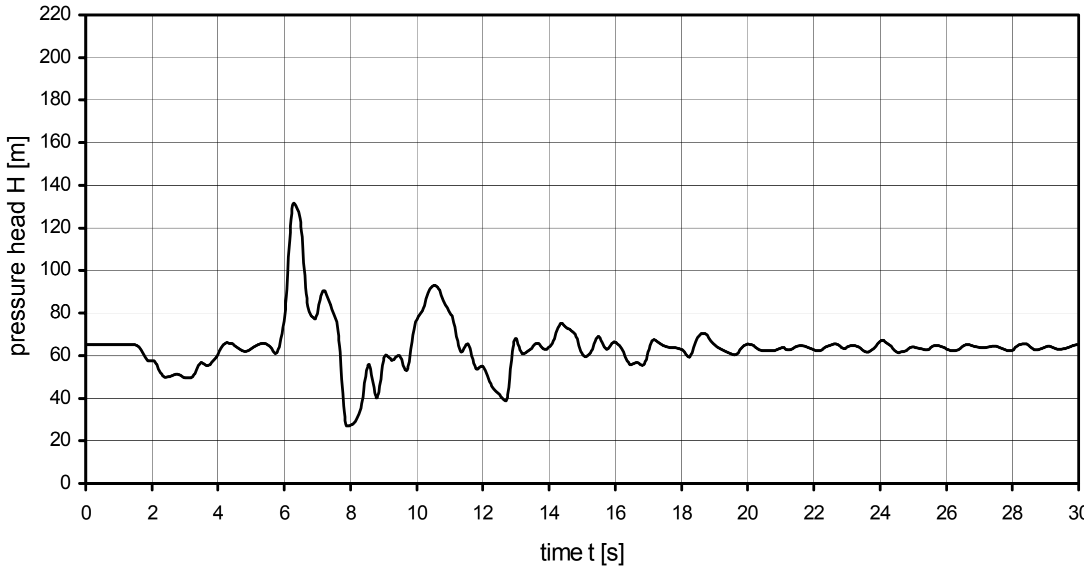

Fig. 7. Pressure head oscillations at the final cross-section of pipeline no. 7 (node no. 7)

As can be seen in Fig. 4 pipe no. 7 and 46 have a direct connection with node 8, and pipe 41 is situated farther off pipe no. 29, but the flow intensity in this pipe is significantly great reaching 37.69 dm^3/s. The maximum and minimum pressure heads at some selected nodes are as follows:

- H_{max} = 130,6 m; H_{min} = 28,0 m of H_2O at the end of pipe no. 7 (node 7);
- H_{max} = 114,1 m; H_{min} = 22,4 m of H_2O at the end of pipe no. 46 (node 29);
- H_{max} = 90.80 m; H_{min} = 18,5 m of H_2O at the end of pipe no. 47 (node 30);
- H_{max} = 89,30 m; H_{min} = 41,9 m of H_2O at the end of pipe no. 43 (node 6);
- H_{max} = 104,1 m; H_{min} = 20,7 m of H_2O at the end of pipe no. 41 (node 27);
- H_{max} = 88,50 m; H_{min} = 42,8 m of H_2O at the end of pipe no. 38 (node 5);
- H_{max} = 54,10 m; H_{min} = 30,8 m of H_2O at the end of pipe no. 14 (node 14);
- H_{max} = 83,30 m; H_{min} = 50,2 m of H_2O at the end of pipe no. 26 (node 22).

This illustrative example proves that the extreme pressure heads in unsteady state in pipes situated much farther away from the disturbance source, and particularly in the case of pipe no. 14, are insignificant since the pressure head increment in the phase of higher pressure attains21.1 m of H_2O, and the maximum pressure head reduction reaches only 2.2 m of H_2O with regard to stationary conditions. The analysis of an arbitrary water supply looped network should therefore refer to pipes situated as close to the source of flow disturbance as possible.

5 Summary and final conclusions

The main purpose of the paper was to present an adequate method for the analysis of transients flows in water pipe looped networks. To solve a set of partial differential equations describing the unsteady flow phenomenon in closed pipelines under pressure, there was applied *the method of characteristics*, where the partial differential equations of hyperbolic type are transformed into ordinary differential equations, which have been solved by *the finite-difference method.*

In the numerical calculations advantage has been taken of the iterative process referred to as *the predictor-corrector method* which can be used in the cases when frictional effects are very important. In this method the corrected values of the unknows H(x,t) and Q(x,t) are calculated on the basis of the mean resistance of elementary pipe sections related to the flow value of the preceding and following time steps. Appropriate equations are used in the predictor-corrector method for internal nodes of the grid of characteristics, as well as for the boundary nodes. The main part of the paper is connected with presentation of suitable equations describing the unsteady flow phenomenon occurring in complex pipeline systems.

In the case of water pipe looped network, one computational example is given, in which the unsteady flows throughout the entire network were caused by a sudden closing the gate valve in the midpoint of pipeline no. 9. Numerical calculations were made with respect to the above specified system, and some results were presented in the paper in a graphic form, indicating the pressure heads under unsteady conditions in the selected final cross-sections.

The closing procedure of the gate valve installed at the midpoint of pipeline 9 with the time of closure T_c = 5 s will be accompanied by minimal pressure head 4,6 m H_2O and the maximal pressure head 207,4 m H_2O. The permissible pressure for cast iron flanged, or flared pipes of normal wall thickness is 1.0 MPa, i.e. approximately 100 m of H_2O.

The correctness of the applied procedure for analyzing the unsteady flows with regard to single pipes, has been verified by the use of our own unsteady-flow experiments. The investigations were carried out on a model situated at the Faculty of Environmental

Engineering of the Warsaw University of Technology, and concentrated on unsteady-flows caused by a sudden closure of fluid flow in straight section of steel pipeline. Advantage was also taken of the investigation results made available by Pezzinga from the Institute of Hydraulics, Hydrology and Water Management, University of Catania (Italy), obtained from model tests carried out at the Hydraulic Laboratory. The calculation results have indicated a good compatibility with the measurements not only with respect to our own investigations but also other research data. Due to limited space of the paper it was not possible to include the investigation results in the paper.

For the reason that the method proposed in the paper has turned out to be an effective and useful instrument for the analysis of unsteady flows through the single pipeline, use of it has also been made for analyzing unsteady flows in complex water supply systems, e.g. in the looped networks. In fact the course of the phenomenon in complex pipeline systems is similar to single pipes. The main difference lies in some additional boundary conditions.

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Comparison of Methods for Determination of Potential Evapotranspiration (PET)

Elzbieta Woloszyn

Gdansk University of Technology, Fac. of Hydro- and Environmental Engineering,
ul. Narutowicza 11/12, 80-952 Gdansk, Poland, ewol@pg.gda.pl

Abstract

A potential evapotranspiration (PET) is an index of the maximum possible evapotranspiration when the available energy is the only limiting factor. This value is the most frequently used as an input data in hydrologic models especially in the models of the water-balance in the drainage basin and in the models for estimations of the water needs of the vegetation cover (irrigation needs). In the paper the comparison of the four the most often used methods for determination of potential evapotranspiration are presented. The results of calculations according to Konstantinow,Turc, Thornthwaite and Penman are compared.
The calculations were based on the meteorological data from the station Bielnik located near Gdansk.

1 Introduction

Potential evapotranspiration (PET) is defined as the maximum rate of evapotranspiration that would occur from a large area completely and uniformly covered with growing vegetation which has access to an unlimited supply of soil water and without advection or heat storage effect. Thornthwaite introduced this concept in 1948 [1, 9] and it depends essentially on climate but is independent of the surface characteristics.
However, we know that the several characteristics of vegetative surface have a strong influence on evapotranspiration rate: the albedo of the surface, which determines the net radiation, the maximum leaf conductance, which is largely determined by vegetation height and presence or absence of intercepted water. Strictly speaking, it is an abstract concept, which is very rarely reflected in reality. However, it is often used as an index of possible evapotranspiration in the atmosphere: "the drying power". Potential evapotranspiration (PET) also plays an important role in the modelling of the water balance of the drainage basin and in the evaluation of the water needs of irrigated areas.
The existing research concerning potential evaporation [1, 2, 6, 7, 8, 9, 10] is quite rich. There are various methods of calculating its value. However, it is difficult to evaluate and compare these methods, because there is no consistent empirical model.
Next to water supply (precipitation), evaporation is the most important process in the hydrologic cycle on land. In the prevailing hydrologic conditions in Poland, on the average approx. 75% of the annual precipitation undergoes evaporation.
In Poland are no generally required methods or guidelines for calculating watershed actual evaporation. In practice, various methods are used, depending on the goals of the research, the accessibility of data and the experience of the authors.
The practical methods of calculating potential evaporation as well as actual watershed evaporation can be classified as follows:

— based on water balance,
— based on heat balance,
— based on equations of turbulent diffusion,
— empirical,
— semiempirical,
— complex (...i.e., a combined method).

In the present study, we compare methods proposed by four researches: Konstantinow [4, 10], Turc [8, 10], Thornthwaite [1, 7], Penman [2, 6, 10].

There have been many studies comparing various models and methods of calculation, but to date there has been no study specifically comparing these four methods. These methods have been selected for comparison because they are used very frequently. Although the methods of Turc and Thornthwaite are not used quite so frequently in Poland, their advantage is that, they are simple and require the knowledge of only a few parameters.

Konstantinow method elaborated in former U.S.S.R in 1956, as reported by [4, 10], gives quite good results, and it has moreover been used in the Hydrologic Atlas of Poland [4], in which the monthly values of watershed potential evapotranspiration (PET) were calculated in 17 meteorological stations for the period 1951–1970.

Of the four methods compared, the most modern method is that of Penman [6, 8, 10] which makes possible the calculation of potential evapotranspiration (PET) during a small lapse of time (24 hours). However, it requires a large amount of data and appropriate software. This model is chiefly used in the mathematical modelling of the drainage basin, which takes into account water balance.

2 The methods of calculation of PET

2.1 The calculation of potential evapotranspiration (PET) using the method of Konstantinow

The basis of Konstantinow method, which uses standard meteorological data, is the following equation of turbulent diffusion proposed by Konstantinow in 1956 [4, 8]

$$\mathrm{PET} = -k_w \cdot \rho \cdot \frac{\partial q}{\partial z} \tag{1}$$

PET – potential evapotranspiration,
k_w – coefficient of the turbulent exchange depending on the wind velocity roughness of the surface and Richardson's number,
ρ – air density,
q – specific humidity,
z – elevation.

Konstantinow calculated the empirical dependence between, the value of vertical gradients of air temperature and actual pressure of water vapour in the air, and the value of the temperature and pressure of water vapour at the height of z meters. Next, using the equation of the turbulent diffusion he calculated the relationship between the value of potential evaporation [PET, mm] and the value of air temperature [t°C] and the pressure of water vapour in the air [hPa] at the height of z [m].

This relationship is presented in the form of a chart. Since during the period of winter-spring transition the sun first heats the surface of the Earth and the air is then heated by the earth, the air temperature close to the earth's surface is greater than at the height of z m. In order to take

this into account, during the months of January to June a positive corrective value is added to the calculation, while during the months of July to December a negative corrective value is added (since the earth cools faster than the air).
Research conducted by Polish hydrologists during 1956–1971 introduced some modifications to Konstantinow's nomogram, and also confirmed the utility of this method of calculation for the hydrologic conditions prevalent in Poland [4]. It was estimated that the margin of error relative to the average values over a multi-year period is less than 20%.

2.2 The method of Turc

The French hydrologist Turc [8, 10] devised empirical formulas for potential evapotranspiration, which for the regions of moderate climate with humidity greater than 50% take the forms:
for decade

$$\mathrm{PET}_d = 0.15\ \frac{t}{t+15}\ (J_g + 50) \tag{2}$$

for month

$$\mathrm{PET}_m = 0.4\ \frac{t}{t+15}\ (J_g + 50) \tag{3}$$

where: t – mean temperature for the period [°C],
J_g – mean net radiation energy available at surface [cal · cm^{-2} · day^{-1}],

$$J_g = J_{gA}\left(0.18 + 0.62\frac{n}{N}\right) \tag{4}$$

J_{gA} – total daily theoretical radiation energy [cal · cm^{-2} · day^{-1}],
N – maximum possible duration of bright sunlight,
n – actual duration of bright sunlight.

2.3 Method of Thornthwaite

A widely used method for estimating the potential evaporation (PET) is given by Thornthwaite [1, 9]. The Thornthwaite equation is based on an exponential relationship between mean monthly temperature and mean monthly evapotranspiration (consumptive use). This relationship is based largely on experience in the central and eastern United States.

The Thornthwaite equation for 30 day month is:

$$\mathrm{PET} = 1.62 \cdot \mathrm{b}\left(\frac{10 \cdot t_m}{J}\right)^a \quad [\mathrm{cm}] \tag{5}$$

in which: b – represents day-time hours expressed in units of 12 hrs,
J – the temperature-efficiency (heat index) is a sum of 12 monthly values of the heat index i:

$$J = \sum_{i=1}^{12} i, \quad i = \left(\frac{t_m}{5}\right)^{1.514} \tag{5a}$$

where: t_m is the mean monthly temperature [°C],
a – is obtained from a cubic function of the heat index J as:

$$a = 6.75 \cdot 10^{-7} \cdot J^3 - 7.71 \cdot 10^{-5} \cdot J^2 + 1.792 \cdot 10^{-2} \cdot J + 0.49239 \tag{5b}$$

Since the number of days in a month varies from 28 to 31 and since the number of hours in the day between the onset of evapotranspiration in the morning and its termination in the evening varies with the reason and with latitude, it becomes necessary to reduce or increase the unadjusted rates by a factor which varies with the month and the latitude (b).

2.4 The Penman equation

The Penman equation is based on the combination of two different approaches: mass transfer and energy balance in one equation. Penman's approach has the advantage that only standard climatic data together with some empirical coefficients are used in a physically meaningful equation. The simplified form of Penman equation given by McCulloch [6, 9, 10] is as follow:

$$\begin{aligned}\mathrm{PET} = {} & \frac{\Delta}{\Delta+\gamma} \cdot R_a\left(a_1 + a_2 \cdot \frac{n}{N}\right) \cdot (1-r) - \frac{\Delta}{\Delta+\gamma} \cdot \sigma \cdot T^4 \cdot \left(a_3 - a_4\sqrt{e_d}\right) \cdot \left(a_5 + a_6 \cdot \frac{n}{N}\right) + \\ & + \frac{\Delta}{\gamma+\Delta} \cdot a_7 \cdot (a_8 + a_9 \cdot h) \cdot (a_{10} + a_{11} \cdot u) \cdot (e_a - e_d) \cdot\end{aligned} \tag{6}$$

where: Δ – slope of the saturation vapour pressure curve at mean air temp. [mb/°C],
γ – constant of the wet and dry bulb psychometer equation [mb/°C],
R_a – theoretical incoming short wave radiation at the limits of the earth's outer atmosphere [mm of evap.],
r – reflection coefficient or albedo,
n – actual hours of sunshine [h],
N – theoretical duration of sunshine [h],
σT_a^4 – black body radiation at mean air temperature [mm of evap.],
e_d – mean vapour pressure [hPa],
e_a – saturation vapour pressure at mean air temp. [hPa],
u – run of wind (km/day) at 2 m,
T_a – air temperature [K], at elevation 2 m,
a_1 to a_{11} – empirical constants, dependent of the units used and the location of the station,
h – elevation [m].

Using the appropriate values of coefficients in this case the equation may be written in the form:

$$\begin{aligned}\mathrm{PET} = {} & \frac{\Delta}{\Delta+\gamma} \cdot R_a\left(0.209 + 0.565 \cdot \frac{n}{N}\right) \cdot (1-r) - \frac{\Delta}{\Delta+\gamma} \cdot \sigma \cdot T^4 \cdot \\ & \cdot \left(0.56 - 0.08\sqrt{e_d}\right) \cdot \left(0.1 + 0.9 \cdot \frac{n}{N}\right) + \frac{\Delta}{\gamma+\Delta} \cdot 0.26 \cdot (1.0 + 0.00015 \cdot h) \cdot \\ & \cdot (1.0 + 0.54 \cdot u) \cdot (e_a - e_d) \cdot \quad \text{[mm/day]}\end{aligned} \tag{7}$$

3 The results of calculations of the potential evapotranspiration (PET)

The comparison of the four methods of calculation of the potential evapotranspiration (PET) was made possible by the complete data available from the Bielnik station near Gdansk. The data used were from 1988. Special computer programs in Pascal were elaborated for the calculation.

Depending on the method of calculation, the time lapse was taken to be 24 hours (Penman), 10 days (Turc, Konstantinow) and 1 month (Thornthwaite).

The results of the calculations are shown in Tables 1 to 6 and in Fig. 1,2, 3. Both, the monthly and the seasonal PET sums were compared.

Table 1

Decade, monthly and annual sums of PET [mm] estimated by Konstantinow method, Bielnik 1988

	I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII	Year
I*)	7.70	6.00	6.20	15.20	25.00	32.00	38.20	27.50	20.00	11.00	2.70	2.20	
II*)	4.10	6.80	6.50	15.70	25.00	26.50	33.00	31.00	11.50	12.00	4.00	3.70	
III*)	3.63	5.04	12.10	14.30	37.40	33.00	39.27	24.20	15.00	5.28	2.20	7.15	
I + II + III	15.43	17.84	24.80	45.20	87.40	91.50	110.5	82.70	46.50	28.28	8.90	13.05	572.1
Month	14.57	17.40	24.18	45.00	89.28	99.60	112.2	85.56	43.50	21.70	8.40	12.40	573.8

*) decade

Table 2

Decade, monthly and annual sums of PET [mm] estimated by Turc method, Bielnik 1988

	I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII	Year
I*)	3.08	1.76	0.06	11.25	31.98	35.60	38.12	36.82	19.32	13.03	0.83	0.00	
II*)	0.00	1.22	0.85	19.24	35.28	42.53	30.10	32.64	17.66	12.91	2.31	0.06	
III*)	0.00	0.30	6.43	18.34	31.42	22.56	33.47	26.39	14.08	4.92	0.00	2.54	
I + II + III	3.08	3.28	7.34	48.83	98.68	100.7	101.7	95.85	51.06	30.86	3.14	2.60	547.1
Month	1.93	3.67	7.52	50.07	101.6	104.2	104.2	97.51	52.32	31.20	3.28	2.56	560.0

*) decade

Table 3

Decade, monthly and annual sums of PET [mm] estimated by Penman method, Bielnik 1988

	I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII	Year
I*)	1.15	1.64	4.15	11.57	27.29	34.37	36.31	30.94	15.52	6.82	1.17	0.56	
II*)	0.62	2.05	5.71	17.41	30.87	38.26	29.08	27.72	12.04	3.52	0.83	0.48	
III*)	0.18	3.37	9.43	19.28	34.19	23.63	34.79	23.27	9.84	1.72	0.89	1.17	
Month I + II + III	3.77	7.06	19.28	48.25	92.36	96.34	100.3	81.91	37.38	12.06	2.89	2.21	503.7

*) decade

Table 4

Monthly and annual sums of PET [mm] estimated by different methods, Bielnik 1988 with precipitation [mm] and mean temperature [°C]

	I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII	Year
Konstantinow	14.57	17.40	24.18	45.00	89.28	96.60	112.22	85.56	43.50	21.70	8.40	12.40	573.8
Turc	1.93	3.67	7.52	50.07	101.62	104.18	104.16	97.51	52.32	31.20	3.28	2.56	560.0
Thornthwaite	2.46	3.32	6.23	38.74	102.36	116.50	139.86	115.48	76.76	38.37	3.71	3.65	647.4
Penman	3.77	7.06	19.28	48.25	92.36	96.33	100.20	81.91	37.38	12.06	2.89	2.21	503.7
Precipitation	35.30	31.20	45.40	18.90	54.80	87.70	115.30	54.40	37.30	15.20	68.80	51.50	615.8
Mean temperature	0.82	1.08	1.40	7.00	14.93	16.90	19.40	17.40	14.40	8.70	1.21	1.26	8.7

Table 1 and 2 show a comparison of the calculation of monthly PET sums obtained from the decade PET sums with the calculation of monthly PET sums performed on the basis of the average monthly values of meteorological parameter. The differences in these results are very small especially for the sums for the seasons.

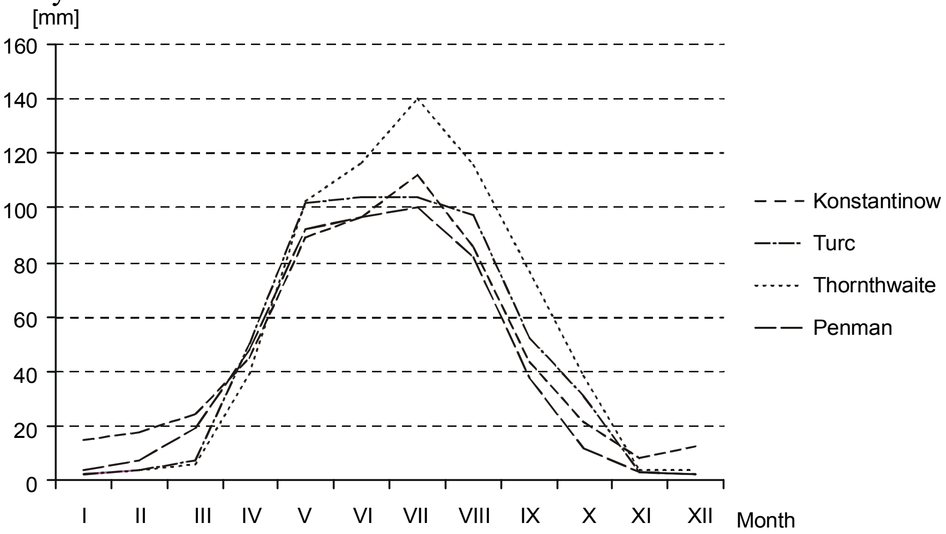

Fig. 1: Seasonal distribution of potential evapotranspiration (PET) calculated by various methods, Station Bielnik 1988

The results of monthly sums of PET obtained using the Turc, Konstantinow and Penman methods were very close to each other especially for the growth season, while the results using the Thornthwaite methods were significantly higher (approx. ~30% for the summer) – Tab. 4, 5, 6, Fig. 1, 2, 3.
In Table 5 the results for seasonal sums of PET are presented together with the ratio of each value of PET to value obtained by Penman method. For the winter season, the highest values gives the Konstantinow method (more 46%) while for the warm period and for all year the highest values gives Thornthwaite method (from 24 to 40% higher).

Table 5

Seasonal and annual sums of PET [mm] estimated by various methods, Bielnik 1988

	Winter		Summer		Growth season		Year XI-X	
	XI-IV		V-X		IV-IX		IIX-X	
	[mm]	*)	[mm]	*)	[mm]	*)	[mm]	*)
Konstantinow	122.0	1.46	451.9	1.08	519.4	1.06	573.8	1.14
Turc	69.0	0.83	491.0	1.17	509.9	1.04	560.0	1.11
Thornthwaite	58.1	0.70	589.3	1.40	609.1	1.24	647.4	1.29
Penman	83.5	1.00	420.2	1.00	491.6	1.00	503.7	1.00
Precipitation	232.2	2.82	383.6	0.91	368.4	0.75	615.8	1.22

*) ratio: value [mm]/Penman [mm]

Table 6

Seasonal summsof PET [mm] estimated by various methods, Bielnik 1988

	XI-III	IV-V	VI-VIII	IX-X	XI-X
Konstantinow	77.0	134.3	297.4	65.2	573.8
Turc	19.0	151.7	305.9	83.5	560.0
Thornthwaite	19.4	141.1	371.8	115.1	647.5
Penman	35.2	140.6	278.4	49.4	503.7

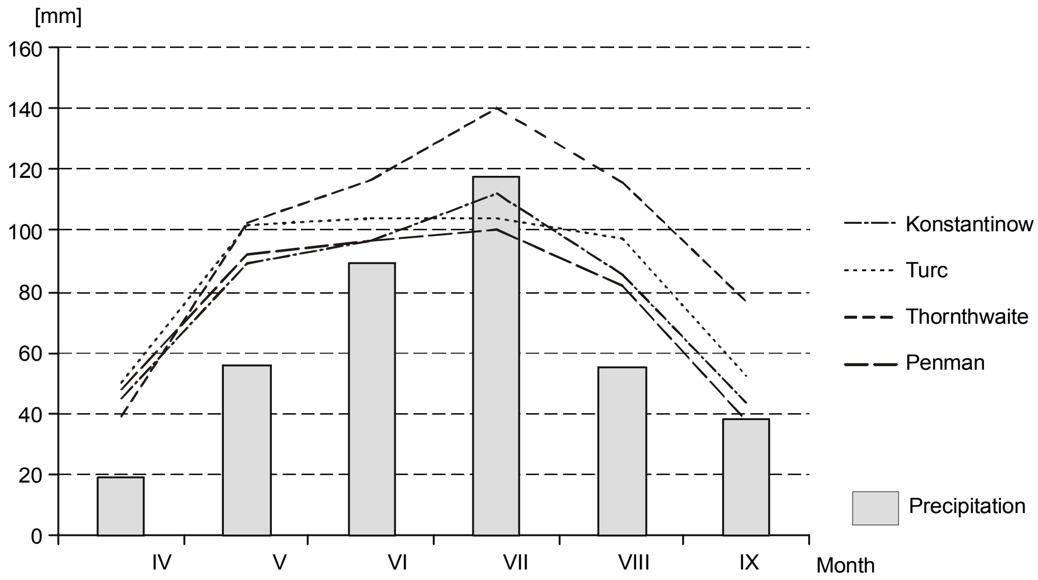

Fig. 2: The monthly sums of PET with precipitation, for the growth season, Bielnik 1988

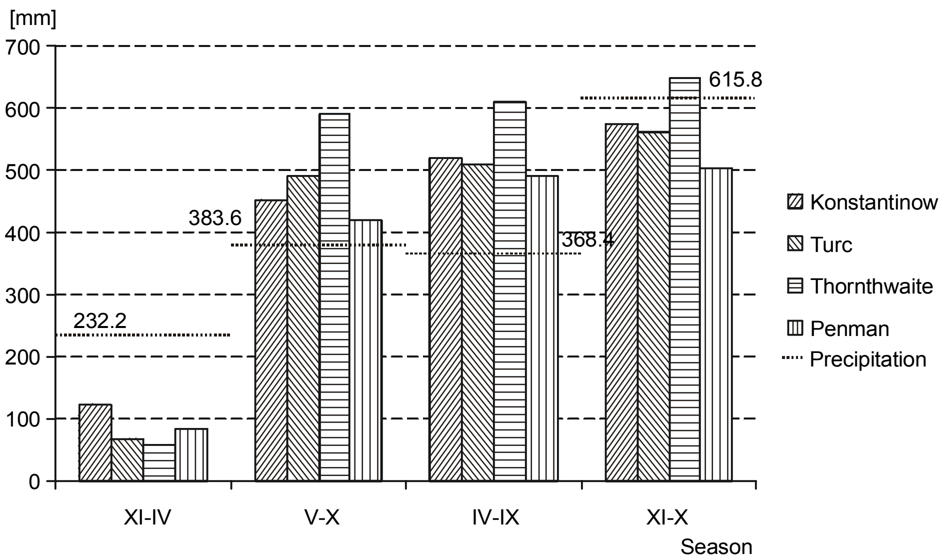

Fig. 3: Comparison of the seasonal sums of PET determined by various methods, Bielnik 1988

4. CONCLUSION

Taking the most modern Penman method as a frame of reference, we can state that under Polish hydrologic and climate condition (like Bielnik) it is possible to use for estimating the potential evapotranspiration (PET) for long time period (10 days, one month), very simple Turc method because the closeness of the results (from 3% to 17%) with model of Penman. The good agreement for warm season with Penman equation were also for Konstantinow methods, but this method is rather complicated to calculation (chart). The Thornthwaite method gives the results significantly higher for warm period and for all the year (24% to 40%), while the amounts of PET are slightly lower for winter (30%). However, for the estimation of the daily value of PET the only appropriate method is the method of Penman.

In the research of Sarnacka and all [5] for the station of Swojec near Wroclaw for the growth season and period of 1965 – 1969, the methods of Penman, Thornthwaite and Turc were also compared. The results of PET calculation for method of Thornthwaite were in average 1.07 higher (1.12 to 0.99 depending a year) than for Penman methods, whereas for Turc method the value of PET were lower in average 0.89 (0.85 to 0.95 of Penman PET). The Konstantinow method was not considered. In the case of station Bielnik for year 1988, the ratio to Penman method for growth season was 1.24 for Thornthwaite method, 1.04 for Turc and 1.06 for Konstantinow method.

The problem of the elaboration the practical and simple method of estimation the potential evapotranspiration (PET) needs the continuation of the research. It is necessary to consider another periods as well as various regions and also to investigate the comparison with the measurements of evapotranspiration in the catchment.

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The Comparative Study of Daily Loads of Polish Pumped-Storage Plants in relation to the Curves of Daily Power Generation in the National Power System

Jan Wróblewski

Technical University of Gdańsk, Department of Hydraulic Structures and Water Management, PL 80-952 Gdańsk Wrzeszcz, ul. Narutowicza 11/12, Poland, Fax: ++48 58 3472413, jwro@pg.gda.pl

Abstract

The paper provides an analysis of daily loads of the pumped-storage plants Żarnowiec and Porąbka-Żar in relation to the curves of daily power generation in the national power system in the years 2000-2001. The studies are made for one selected working day of each month, characteristic Saturdays and Sundays, as well as for all Polish national holidays. The investigations are based on daily power balances of the power system made in the National Power Dispatching Centre. The studies are carried out to facilitate the prognosis of load of both investigated power stations for any day of the year.

1 Introduction

The analysis carried out in the paper concerns the two largest pumped-storage plants in Poland: Żarnowiec (4 units 179 MW each) and Porąbka - Żar (4 units 125 MW each). The presented distributions of load in these power stations are considered in relation to the distribution of load in the national power system in the years 2000-2001 [1].

The investigations extended on characteristic working days, Saturdays and Sundays of each month. All Polish national holidays were also scrutinised: Easter holidays (2 days), Christmas holidays (Christmas Day and Boxing Day, also including the working day of Christmas Eve), the period of May 1 to 3, Corpus Christi Day, August 15, All Saints Day, Independence Day – November 11 and New Year Day with New Year's Eve. Some working days neighbouring to national holidays, termed as untypical by the National Power Dispatching Centre, for example May 2, Christmas Eve, New Year's Eve, as well as other working days neighbouring with holidays and weekend days are the most difficult to cope for the dispatching centre. Working days and weekend days were selected from the middle of the month. The same days were chosen for the year 2001 as for the year 2000. The investigations were carried out to facilitate the prognosis of daily load of the two power stations for any day of the year. The investigations based on reports of power balance from the National Power Dispatching Centre of the National Power Networks (stock company).

The time period after September 2001, when the energy balance market at the energy stock exchange was introduced, was analysed in detail. The change of load of pumped-storage plants in this period was compared to that of September to December 2000. It turned out that introduction of the energy balance market forced the National Power Dispatching Centre in Warsaw to keep an additional intervention reserve in reservoirs of the two pumped-storage plants both for generation and pumping work. As a results the programme work of the power stations was reduced.

2 Description of investigation

Based on daily reports from the national power balance, data concerning operation of the two power stations and the national power system were gathered and the following quantities were calculated:

- generation power of the country,
- pumping power from all pumped-storage plants of the country,
- generation power of the country, with the power consumed for pumping subtracted,
- generation power of all hydro-power plants of the country,
- generation power of the country, not including power generation of all hydro-power plants of the country,
- normal demand for power (corresponding to frequency 50,000 Hz),
- gradient of change of normal demand with respect to time, as calculated from the following equation:

$$q = \frac{\Delta P}{\Delta t} \text{ [MW/min]}, \qquad (1)$$

where: ΔP - power change in time Δt [MW],

Δt - time period of 30 min.

- generation power of the power station Porąbka - Żar [MW],
- pumping power of the power station Porąbka - Żar [MW],
- generation power of the power station Żarnowiec [MW],
- pumping power of the power station Żarnowiec [MW],
- generation power of the power stations Porąbka - Żar and Żarnowiec together [MW],
- gradient of change of generation load in the power stations Porąbka - Żar and Żarnowiec together, as calculated from Equation (1),
- pumping power of the power stations Porąbka - Żar and Żarnowiec together [MW],
- gradient of change of pumping load in the power stations Porąbka - Żar and Żarnowiec together, as calculated from Equation (1),
- total generation power of the country, not including generation power of the power stations Porąbka - Żar and Żarnowiec [MW],
- total generation power of the country, not including pumping power of the power stations Porąbka - Żar and Żarnowiec [MW].

Taking into account the above data, two graphs were made for each analysed day. They are presented here, by way of example, for September 13 (Thursday), 2001. The first graph illustrates the effect of the two pumped-storage plants (Porąbka - Żar and Żarnowiec) on the curve of power generation in the national power system with the indicated power needed for pumping and the peak power generated by these power stations. 5 curves feature in this graph:

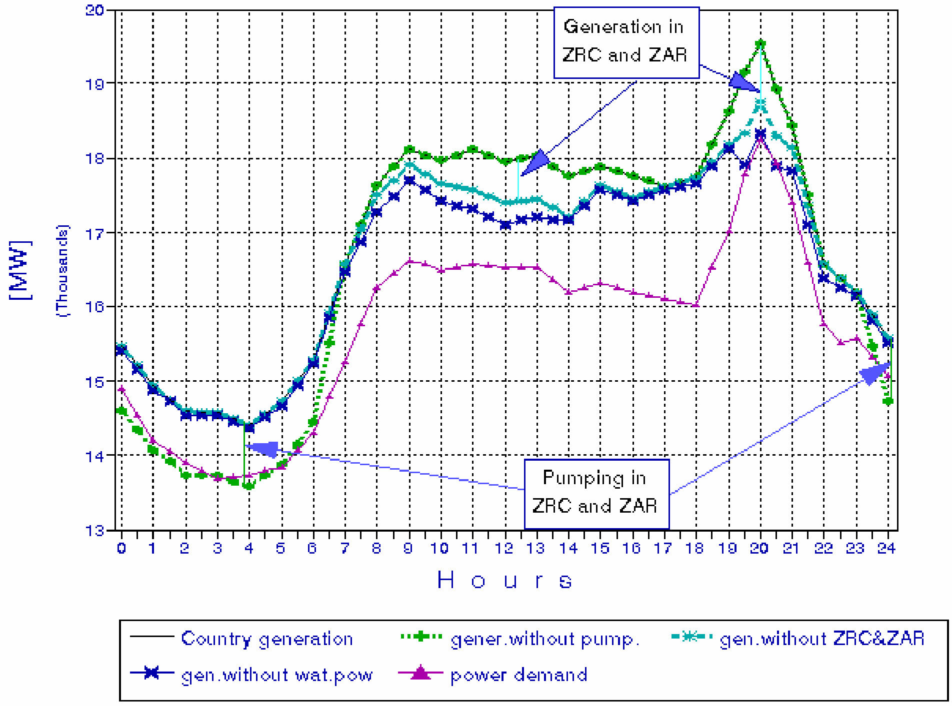

Fig. 1: The effect of power stations Żarnowiec (ZRC) and Porąbka-Żar (ZAR) on the generation curve in the national power system, Thursday, September 13, 2001.

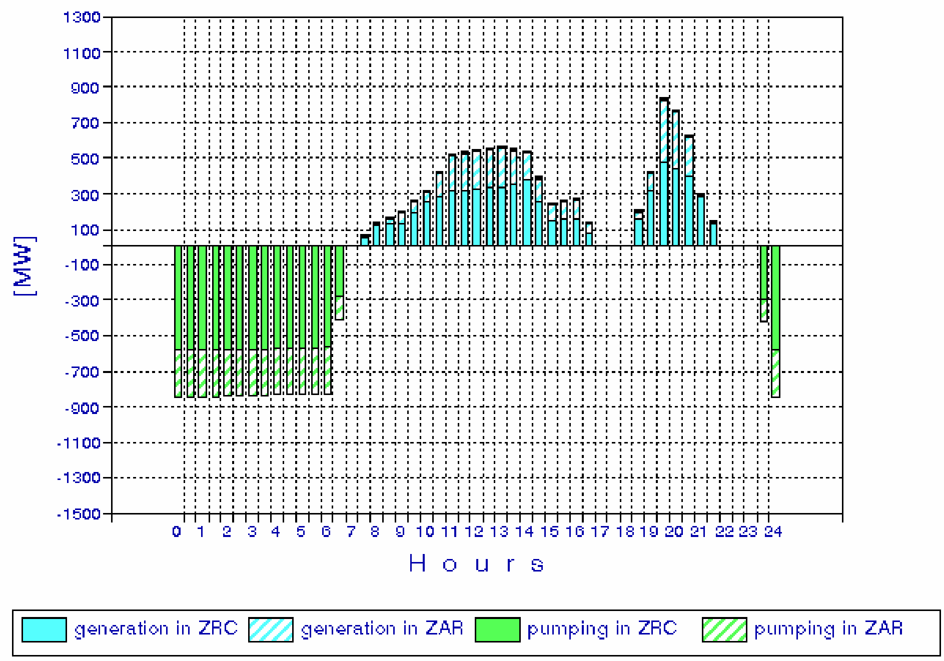

Fig. 2: Summary load of power stations Żarnowiec (ZRC) and Porąbka-Żar (ZAR), Thursday, September 13, 2001.

- generation of the country (upper curve),
- generation of the country, minus power consumption for pumping,
- generation of the country, minus generation from the two power stations,
- generation of the country, minus generation from large hydro-power stations,
- normal demand for power.

The second graph illustrates, in the form of a bar diagram, generation and pumping power in the power stations Porąbka - Żar and Żarnowiec – jointly and separately.

3 Results of investigations

The conducted investigations revealed a number of interesting results and led to a number of conclusions which will be formulated below.

1. A general principle of loading pumped-storage plants in the national power system follows from the programme 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.
2. The distribution of load changes between working days, Saturdays, Sundays and holidays, as well as some other days neighbouring 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 characterised by the largest demand for power of all week days. The peak power usually takes place on Tuesdays or Thursdays.
3. 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 week days.
4. 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 months values of peak demand during the evening peak are below those of the morning peak.
5. The maximum difference in power demand between the winter peak load and minimum summer peak is larger for the evening peak than for the morning peak. On working days in the year 2000 this difference was equal to about 7400 MW for the evening peak and 5650 MW for the morning peak. In the year 2001, the difference was 7925 MW for the evening peak and 5750 MW for the morning peak. Comparing the differences between the working days and Saturdays or Sundays, it seems that in general these differences are the

largest on Saturdays (about 8150 MW for the evening peak and 6750 MW for the morning peak observed in 2001), and the smallest on Sundays.

6. Maximum values of generation load in the analysed pumped-storage plants change in accordance with changes of maximum generation power of the national power system. Operation of each of the two power stations, from the point of view of the national power dispatching centre, is complementary to one another. It is very often that the number of power units operated at one time in one power station is not the same as in the other. Therefore, it was necessary to consider the load of the two power stations jointly, and to analyse the number of units operated in each of them. The maximum summary load in the evening peak of winter working days is of the order of 1000-1050 MW where all 4 units are operated in at least one of the considered power stations. On summer working days the load can decrease even down to 100 MW. In this case not more than one unit is operated. In the morning peak the situation is reversed and the largest generation load is observed in summer, exceeding 800 MW jointly in both power stations where at least 3 units are operated in one of them. In winter this load drops typically to 200-250 MW where at most one unit is operated.

   The maximum summary load of both power stations in the evening peak on Saturdays turns out to be larger, as compared to the working days. This refers both to winter months (November-February) and summer months (June-August), however in summer months the maximum summary load of the two power stations in the evening peak on Saturdays is on average by two times larger than on the working days (2-3 power units are then operated). On Sundays the maximum summary load of the two power stations in the evening peak is in principle similar to that on Saturdays, and also exceeds the load on working days. In general, the maximum summary load on holidays and untypical days is the largest of all week days of the months in which these days occur. Usually, 4 or 3 units in at least one power station are loaded then.

   In the morning peak on Saturdays the load is similar to that on working days, however some differences are conspicuous here. The largest load on Saturdays occurs in summer and reaches 600 MW in both power stations (whereas on working days exceeds 800 MW), where 3 units are operated (as on working days). The lowest load on Saturdays occurs in winter and is, in principle, comparable with that on working days – the average load is about 250 MW, with not more than one unit operated in each power station. In general, in the morning peak on Sundays, the load is lower than on Saturdays. The load on holidays is, in turn, the lowest of all days of the year.

7. The maximum pumping load in the night off-peak is less differentiated as compared to generation load at peak conditions. Anyway, the load in winter is larger than in summer, and there might be some variations of load during the months of spring and autumn. The maximum pumping load in both power stations jointly in the year 2000 occurred in January (1335 MW), in the year 2001 in February (1297 MW) where in both cases all 4 units in at least one power station were operated. The lowest pumping load of 12 months occurred in the year 2000 in November (798 MW), whereas in 2001 in September (852 MW). In both years a minimum of 3 units were operated in at least one power station. On Saturdays, the maximum pumping load in the two power stations jointly is of the same order of magnitude as on working days. On Sundays the pumping load is slightly lower. The maximum pumping load in the night off-peak on holidays is on average larger than on Sundays, but lower than on Saturdays. Usually, 4 or 3 pumping units are operated in at least one power station for all holidays in the night off-peak.

8. It is found that the minimum demand for power in the national power system during the afternoon off-peak on some Sundays and some holidays can be lower that during the night off-peak. This means that on Sundays the demand for power after the morning peak decreases again even below the night minimum.

9. The maximum contribution of power generated by the two power stations in summer in the morning peak as related to the power generated in all large hydro-power stations in Poland is observed on working days and, on average (as averaged from the two investigated years), amounts to 77% (on Saturdays - 75%, on Sundays - 48%, on holidays - 57%). In winter, in turn, this contribution on Saturdays amounts to 75% (whereas on working days is equal to 62%, on Sundays - 38%; on holidays such as: November 1, December 24-26 and New Year Day the contribution is equal to zero, as the pumped-storage plants are not loaded on these days in the morning peak). In the evening peak in summer months, the contribution of the two pumped–storage plants reaches the largest value on holidays - about 76% (on working days - 43%, on Saturdays - 66%, on Sundays – about 72%), whereas in winter this contribution is 85% on holidays (on Saturdays and Sundays - 83%, on working days - 76%).

10. The contribution of gradient of increase in power generated by the two pumped-storage plants in relation to the gradient of increase in demand for power in the national power system in winter during change from the afternoon off-peak to evening peak is the largest on working days and amounts to about 53% on average (on holidays is equal to 49% and remains of the same order of magnitude as on Saturdays and Sundays - 48%). In summer, this contribution on Sundays reaches 53% (on holidays is equal to 46%, on working days - 34%, on Saturdays - 40%).

11. The introduction of balance market for electric energy on September 1, 2001, resulted in certain changes in functioning of active power control within the system of secondary control. Managing the operation of the national power system has become significantly more difficult, and required on the one hand precise planning of load of generation units, and on the other hand securing the safety of operation of the system and quick prevention of possible failure disturbances. Therefore, certain additional limitations were introduced since September 2001 in operation of the two largest pumped-storage plants in Poland. To assure sufficient amount of reserve, certain water surplus is left in upper reservoirs of these power stations, corresponding to one hour of generation work of two units in each power station. Also a certain reserve if left for pumping to enable pumping work of two units in each power station for one hour. The introduction of this principle reduced the usage of energy from upper reservoirs in the period from September to December 2001, as compared to September - December 2000. This fact can indicate that after introduction of the balance market for electric energy, the intervention function of both power stations could become increased. Further investigations are needed to observe intervention work of the two power stations in the national power system.

12. The limitation of energy capacity of the reservoir for programme work led to a decreased duration of generation load in the evening peak, practically on all days of the week, by about 0.5 to 1.0 hour. The duration of pumping load in the night off-peak was also decreased on average by 0.5 to 1.5 hour. The duration of generation load in the morning peak practically did not change.

13. The maximum generation load in the evening peak and maximum pumping load in the night off-peak are in principle slightly larger on Saturdays in the year 2001, as compared to Saturdays in the period September-December 2000. The loads in the morning peak on

Saturdays are similar for the two investigated years. For other week days, no differences in load were observed in any part of the day. Also, the comparative analysis of load graphs and curves of demand for power in the national power system does not show significant differences between the years 2000 and 2001.

4 Summary

The presented results of investigations give a background for load planning in the pumped-storage plants of Porąbka - Żar and Żarnowiec practically for each day of the year. However, introduction of the energy stock exchange in July 2002 made the possibilities of load prognosis significantly more difficult. In order to plan loading of the power stations, taking the energy stock exchange into account, supplementary investigations need to be carried out for the period after introduction of the energy stock exchange, and the results need to be compared with those of the previous time period.

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