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CONTENT

SCOUR RESISTANCE OF DIKE MATERIALS ON DOWNSTREAM SLOPE DURING OVERTOPPING

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Abstract

Scour resistance of dike material is an essential phenomenon when assessing dike breaching due to overtopping. Usually the resistance of particular materials against scouring is determined via experimental modelling. In the literature sources many recommendations and also empirical formulae are introduced. Some of them are based also on experience from overtopping of real structures during floods. In this paper, summary of past experimental results is carried out. Individual approaches and formulae are compared and assessed from the viewpoint of their applicability on steep slopes. The resulting recommendations can be used as a basis when assessing threshold non-scouring velocities or critical shear stress as a part of simulation of dike breaching.

Keywords

Critical shear stress, Critical specific discharge, Froude number, Non-scouring velocity, Scour resistance

1 INTRODUCTION

Dike overtopping during flood events becomes more dangerous when dike materials at downstream slope initiate to be eroded.

Scour resistance of downstream materials against water erosion is one of the significant properties affecting dike safety during floods. Factors affecting the resistance and transport of dike materials have been the subject of investigation since the beginning of the 20th century.

Erosion and transport of soil particles starts when the water flow-induced shear stress exceeds the resistance of dike materials. This resistance is usually defined either by critical shear

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stress, critical velocity or critical flow (specific discharge usually) [1], [7], [9], [11]. Each critical variable (stress, velocity, flow rate) is defined as the maximum value which does not cause particle movement yet [6]. Several authors proposed empirical formulae derived based upon laboratory and field measurements to express those critical values.

The resistance for various materials against water flow is summarized within [2].

In the case that the crest and surface of the downstream slope are protected by riprap or grass layer, the erosion of the dike materials will start after the protection layer is damaged. The results of research regarding the resistance of grass protective layers were also published within [3], [4]. For rockfill dams or riprap protection, Knauss [7] published critical specific discharge as a function of the bottom slope and the effective diameter of stones in the range from 0.5 to 1.0 m. He also took into account the construction process, particularly whether the protection is formed by free laid stones or by hand-placed stones. Some technical measures used for the enhancement of dam protection against overtopping are summarized within [11].

In the traditional literature sources numerous relations expressing critical values (shear stress, non-scouring velocity, etc.) are related to open channels. In these cases relatively mild channels slopes were subject of the researches. Usually granular sediment materials were studied, the grain size only rarely exceeds 100 mm.

Empirical equations expressing the critical shear stress τ_{cr} for unlined surfaces were developed by authors. A simple formula of critical shear stress related to water density ρ and effective grain size d_e was developed by Krey. Kramer and U.S. waterway proposed the critical shear stress as a function of water density, effective grain size, density of sediment ρ_s, and homogeneity modulus M. Schoklitsch took into account - instead of homogeneity modulus - influence of the shape factor C_T. Shields expressed the critical shear stress by means of the so-called Shields parameter θ, which is a function of the Reynolds number of sediments Re_d.

The non-scouring velocity v_n for unlined surfaces were determined according to a number of empirical relations developed by several authors. Mavis developed a relation for non-scouring bed velocity as a function of water density, density of sediments and effective grain size. Levi derived equations for uniformly graded materials, for a gravel-sand mix and for fine-grained soils. Goncarov derived his relations from the condition of balance between the pressure force of static moment and the dead moment force of a particle for 0.1 mm < d_e < 1.5 mm and then for 1.5 mm < d_e < 20 mm. Neil derived his equation - relating as well to water depth h - for d_e > 3 mm and $0.5 > d_e/h > 0.01$. The frequently used equation related to water and sediments density, effective grain size and Chézy discharge coefficient was developed by Meyer-Peter. Samov developed a simple relation of non-scouring velocity related to effective grain size and water depth h.

Only limited research was focused on the resistance of materials on steeper slopes. Usually surfaces lined by the grass or granular materials were subjected to the studies. Results of such research were discussed in the following text.

2 ASSESSMENT OF THE SLOPE RESISTANCE

In general the resistance of materials on the downstream slope of the dike can be assessed by comparing following variables with their critical values:

- Shear stress with critical shear stress [9],
- Flow velocity with the non-scouring velocity [2], [5],
- Froude number with the critical value of Froude number [6], [11],
- Specific discharge with critical specific discharge [7].

Shear stress τ acting on the downstream slope may be expressed by means of the following relation:

$$\tau = \rho \cdot g \cdot h \cdot \sin\alpha , \quad (1)$$

where ρ is the water density, g is the acceleration of gravity, h is water depth at the downstream slope and α is angle of downstream slope.
Quantities (flow velocity, Froude number and specific discharge) for assessing the resistance of materials can be mutually recalculated as follows:

$$\tau = \rho \cdot g \cdot h \cdot \sin\alpha = \rho \cdot g \cdot \frac{v^2}{C^2} = \rho \cdot g \cdot \frac{q^2}{C^2 \cdot h^2} = \rho \cdot g^2 \cdot \frac{\mathrm{Fr} \cdot h}{C^2} , \quad (2)$$

where v is mean velocity, q is specific discharge, Fr is Froude number, C is the Chézy coefficient expressed as follows:

$$C = \frac{1}{n} \cdot h^{1/6} , \quad (3)$$

where n is Manning's roughness coefficient.

3 EXPERIMENTAL RESEARCH

Several experimental researches were carried out to develop empirical relations for the assessment of the resistance of downstream materials.
For dams or dikes with rock fill shoulders or lining, Knauss [7] gives limit specific discharges q_{cr} for the exposed downstream face slope J_d, for effective grain sizes d_e = 0.50 m, d_e = 0.75 m and d_e = 1.0 m (the approximate corresponding weights of grains are 1.75 kN, 5.85 kN and 13.85 kN), and for the combination of methods of their placing (for manual or natural packing). The results obtained from experimental research are given in Fig. 1.

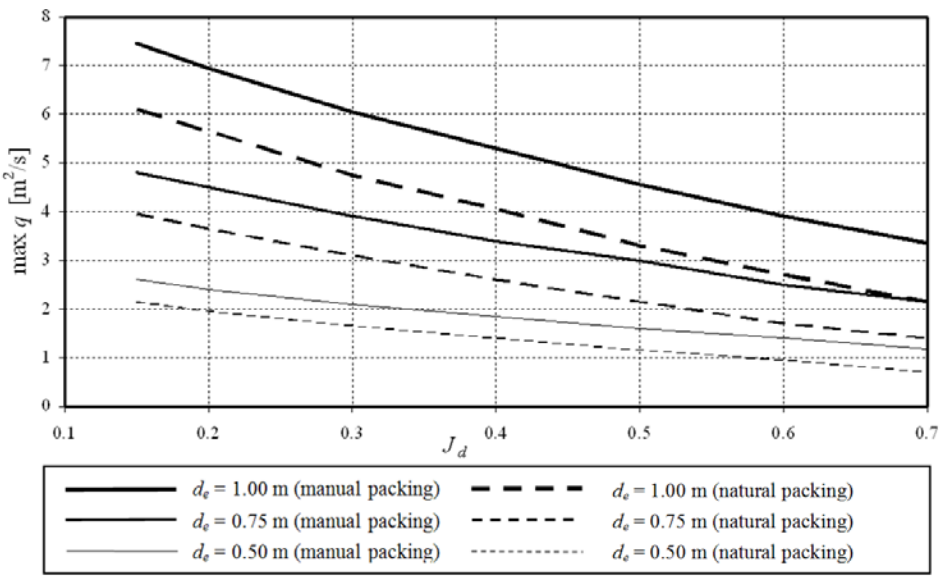
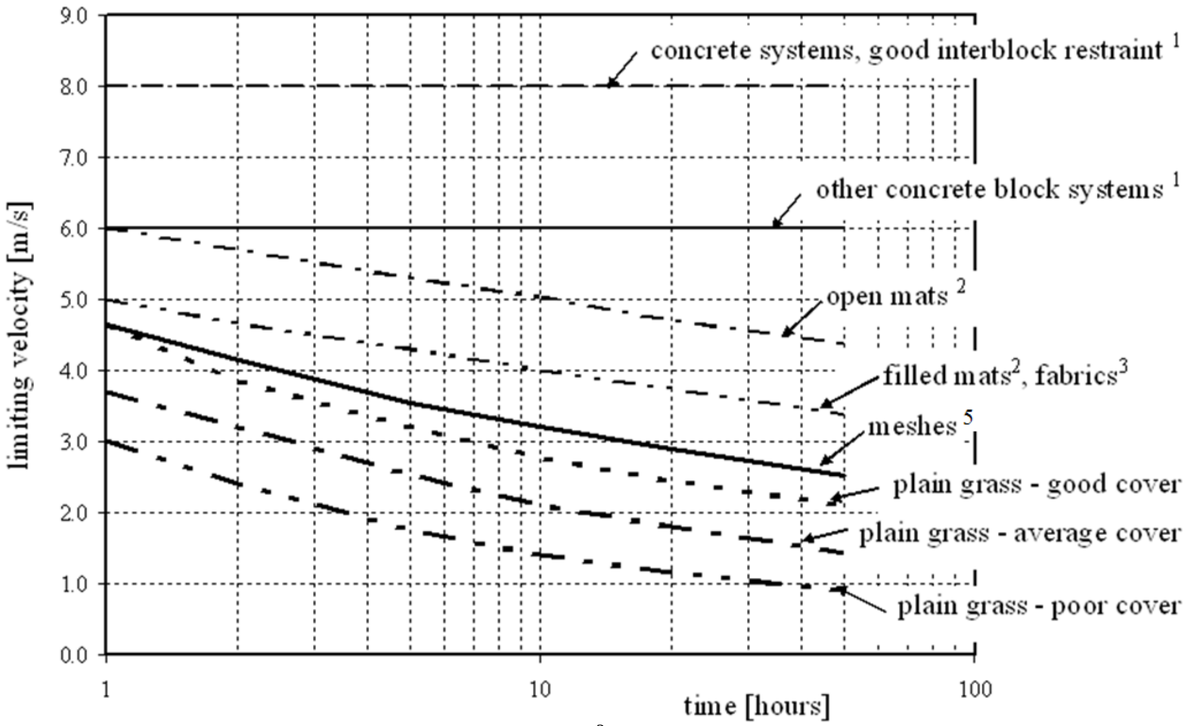


Fig. 1 Limit specific discharges for downstream rock fill lining [7]

For assessing the resistance to erosion of particular types of dam surface lining using the limit (non-scouring) cross-sectional velocity, the graphs in Fig. 2 can be used [2]. The limit cross-sectional velocity depends on the overflow duration (Fig. 2), namely in the case of non-rigid materials (grass, meshes, mats, etc.).



Notes: 1. minimum superficial mass 135 kg/m^2.
2. minimum nominal thickness 2 cm.
3. installed within 2 cm of the soil surface, or in conjunction with a surface mesh.
4. this graph should only be used for erosion resistance to unidirectional flow.
5. all reinforced (by meshes) grass values assume well-established good grass cover.

Fig. 2 Limit velocities for particular materials as a function of overflow duration [2]

Within the framework of the research achieved by Linford and Saunders [9], the critical shear stress τ_{cr} for the rock fill was determined as well:

$$\tau_{cr} = 0.0715 \cdot g \cdot (\rho_s - \rho) \cdot d_e \cdot \frac{1.2}{l}, \tag{4}$$

where ρ_s is the density of sediments and l is the packing (compaction) factor considered at a lower limit value of $l = 0.8$ for well compacted or manually packed rock fill, and an upper limit value of $l = 1.2$ for naturally packed rock fill.

Hartung and Scheuerlein [5] expressed flow velocity v as a function of the water depth, slope gradient, effective grain size and flow aeration. The calculated flow velocity is compared with the limit (non-scouring) velocity v_n determined from the relation:

$$v_n = \left(\frac{1.33 \cdot g \cdot (\rho_s - \rho)}{\sigma \cdot \rho} \cdot d_e \cdot \cos\alpha \cdot (\tan\varphi - \tan\alpha) \right)^{0.5}, \tag{5}$$

where φ is the angle of internal friction of the surface layer, α is the slope angle and σ is the degree of flow aeration expressed as follows:

$$\sigma = (1 - 1.3 \cdot \sin\alpha + 0.24 \cdot \frac{h}{d_e}) \qquad (6)$$

The results of extensive research into the resistance of grass cover were published within [3], [4]. Non-scouring velocities and critical shear stress τ_{cr} for grass covers of different quality are given in Table 1.

Tab. 1 Allowable shear stress τ_{cr} and non-scouring velocities v_n for grass cover [3], [4]

grass cover quality	critical shear stress τ_{cr} [Pa]	non-scouring velocity v_n [m/s]	
		erosion-resistant soils	easily eroded soils
dense, uniform, well developed and maintained grass carpet with well rooted turf	177	1.8	1.2
dense, uniform, well maintained grass carpet	101	1.5	0.9
grass mixture, less dense grass carpet *)	48	1.2	0.9
thin grass cover with an irregular surface **)	29	1.1	0.8
temporary grass cover, one year-old grass **)	18	1.1	0.8

*) Unsuitable for overflowed slopes of a slope gradient larger than 10 %.
**) Unsuitable for overflowed slopes of a slope gradient larger than 5 %.

At the laboratory of the Institute of Water Structures at Brno University of Technology, types of structural protection such as riprap and placed stones have been tested on physical model [6], [11]. Three downstream face slopes (1:2, 1:3 and 1:4) were tested in combination with three riprap stone sizes.
A view of the model is shown in Fig. 3.

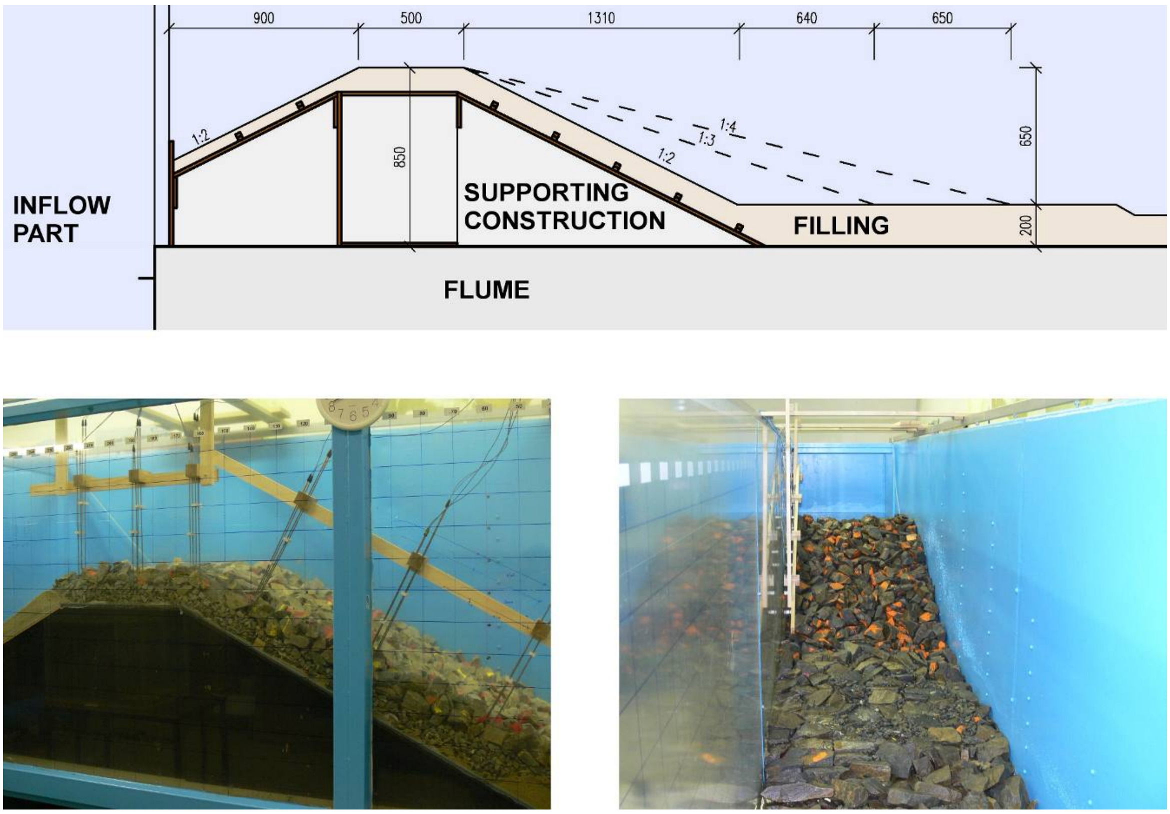


Fig.3 Hydraulic model of the dike [6]

A cross comparison of results was carried out via Froude number related to the effective grain size (the diameter of stones) as shown in Eq. 7 [6], [11].

$$\mathrm{Fr}_{\mathrm{cr}} = \frac{q_{cr}}{\left[g \left(\frac{\rho_s - \rho}{\rho} \right) d_e^3 \right]^{0,5}}, \tag{7}$$

where d_e is the effective grain size [m] and q_{cr} is the critical specific discharge [m^2/s]. The critical unit (specific) discharge q_{cr} is usually defined as a specific discharge at which total destruction of a slope occurs. However, sometimes the critical discharge may be defined in another way, for example as the first movement of stones [10], [11].

Jandora and Spano [6] indicated several types of stone lining movements like the first movements of individual stones, movement of the mass of stones and total destruction of the lining. Corresponding limits of the tested materials in terms of critical specific discharge for each limit were determined and expressed via critical Froude number corresponding to face slope, material and movement limit (Fig. 4). The results are shown for effective grain diameters 18, 58, 88 mm.

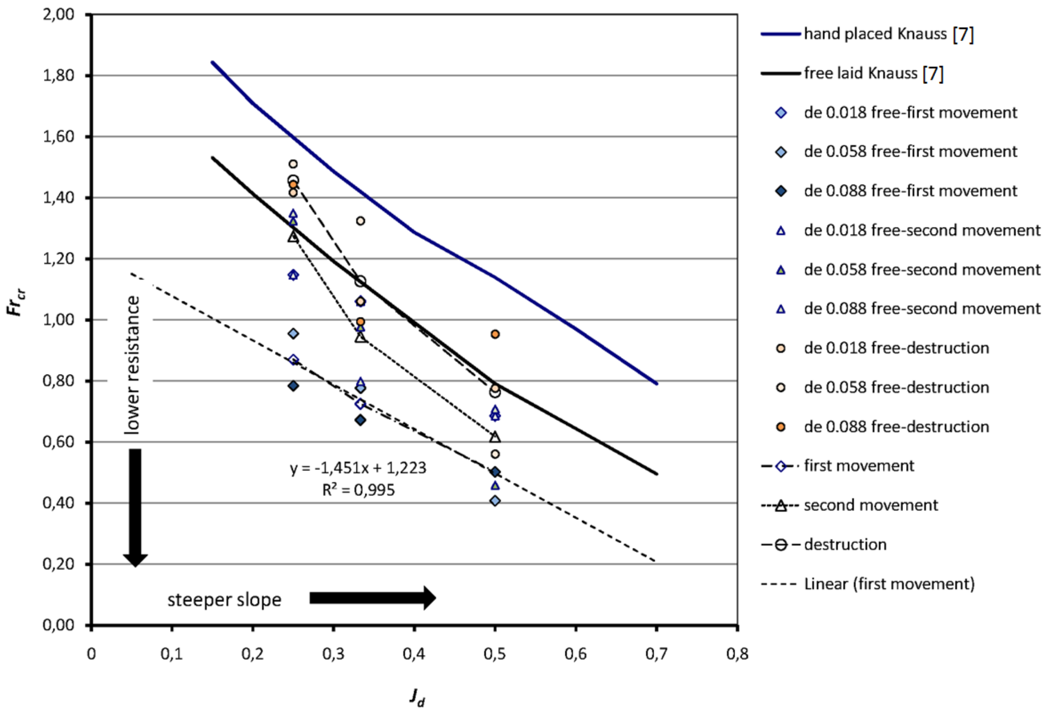


Fig. 4 Critical Froude number as a function of slope [6]

In Fig. 4 results achieved by Jandora and Spano [6] are compared with results achieved by Knauss [7]. The first movement limit was approximated by a simple linear regression line with estimated limits ($0.15 < J_d < 0.7$) as follows:

$$Fr_{cr} = -1.451 \cdot J_d + 1.223 \tag{8}$$

4 COMPARISON

In this chapter a comparison of results presented by authors mainly in terms of non-scouring velocity v_n and critical value of Froude number $\mathrm{Fr}_{\mathrm{cr}}$ is carried out. Non-scouring velocity is

used as criterion when assessing resistance of surface covered with grass, Froude number is used in case of surfaces with stone layers.
Values of limit specific discharge shown in Fig. 1 and those obtained from Eq. 7 were expressed in terms of critical Froude number and compared with values presented in Fig. 4. Results shown in Fig. 1 [7] were recalculated using Eq. 7 and displayed in Fig. 4 [6], where line of free laid stones [7] partly shows converged values of Froude number with the destruction line proposed by Jandora and Spano [6] (Fig. 4). Line of hand placed stones [7] (Fig. 4) shows a higher values of Fr_{cr}, this can be attributed to the manual packing which gives higher resistance of slope materials.
Similarly values of non-scouring velocities shown in Fig. 2 are compared with values presented in Tab. 1. Non-scouring velocities presented in Fig. 2 [2] for long-term period of dikes overtopping (50 hours) are 0.95, 1.45 and 2.1 m/s corresponding to poor, average and good grass cover respectively. These values are close to non-scouring velocity values shown in Tab. 1 corresponding to erosion-resistant soils (1.2, 1.5 and 1.8 m/s) with grass cover. Of course assessment of the grass cover quality is rather subjective.
Eq. 4 proposed by Linford and Saunders [9] takes into account the packing (compaction) factor and effective grain size to determine the critical shear stress for the rock fill. Equation proposed by Hartung and Scheuerlein [5] takes into account the angle of internal friction of the surface layer and the degree of flow aeration which are important in case of slopes covered by stones. These scour resistance factors are hard to be compared with the results of [6] and [7].

5 CONCLUSION

Resistance of downstream slope materials was assessed within several experimental studies performed by various authors. In this paper, a summary of some studies developed by authors of unlined surfaces was presented in chapter 1. However the study [10] concluded that the results of traditional older research carried out in hydraulic flumes can not be applied for steep slopes and grains of greater diameter.
Experimental researches for assessment resistance of lined surfaces on steep slopes carried out by Knauss [7], Jandora and Spano [6], Linford and Saunders [9] and Hartung and Scheuerlein [5] are presented in more details, and results taken from the guidelines [2], [3] and paper [4] are also presented in chapter 3.
In the chapter 4 a comparison of equations expressed the resistance of lined surfaces is carried out in terms of non-scouring velocity v_n and critical value of Froude number Fr_{cr}. The reasonable agreement was observed between the values achieved by individual authors and literature sources.

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RESEARCH OF GROUNDWATER FLOW IN FLOOD SITUATION

Ing. Tomas Andrassy[1]

Abstract

During passing of flood wave through river it occurs increasing of groundwater level and thus direct threat of property and especially the groundwater resources (water supply) which are located near the affected rivers. Since during the floods water is polluted the solution of this problem is more than necessary. By transition of flood waves occurs time retardation (increase of groundwater level can be after a few weeks) and lateral retardation (groundwater increases reach in a direction perpendicular to the flow) of spreading groundwater. To these conditions we must adapt also operating of groundwater sources located near rivers. As inputs in the research will be used specific data from those areas (geological, hydrogeological, hydrological data from the site) Based on the input data, the model will be calibrated, verified and then will be used to forecast groundwater flow during passing flood wave and for subsequent determination of impact to groundwater sources. Outputs from the model will be used for design of operational plan for groundwater sources during floods. Outputs more help understand groundwater flow in flood situations. One of the outcomes will be also threat maps of groundwater sources near rivers. Maps will be in digital and graphical form. On maps will be show the direction of groundwater flow - which direction and at what time it gets to water pollution

Keywords

Groundwater flow, flood situation, TRIWACO, modeling, Sihot, Sedláčkov island

1 INTRODUCTION

During passing of flood wave through river it occurs increasing of groundwater level and thus direct threat of property and especially the groundwater resources (water supply) which are located near the affected rivers. Since during the floods water is polluted the solution of this problem is more than necessary. By transition of flood waves occurs time retardation (increase of groundwater level can be after a few weeks) and lateral retardation (groundwater increases reach in a direction perpendicular to the flow) of spreading groundwater. To these conditions we must adapt also operating of groundwater sources located near rivers. As inputs in the research will be used specific data from those areas (geological, hydrogeological, hydrological data from the site) Based on the input data, the model will be calibrated, verified and then will be used to forecast groundwater flow during passing flood wave and for subsequent determination of impact to groundwater sources. Outputs from the model will be used for design of operational plan for groundwater sources during floods. Outputs more help understand groundwater flow in flood situations. One of the outcomes will be also threat maps of groundwater sources near rivers. Maps will be in digital and graphical form. On maps will be show the direction of groundwater flow - which direction and at what time it gets to water pollution [1]

2 MATERIAL AND METHODS

During research of groundwater flow in floods situation we have to count with problem of drainage (CD) and infiltration resistance (CI) which is related with interaction of ground water and surface waters and river bed colmatatation. Drainage or infiltration resistance is related with filtration coefficient and thickness of colmatated layer of riverbed. [1] It has to be a lot of combination of these two parameters for one value of CD/CI. During survey it is too difficult correctly define filtration coefficient and thickness of colmatated layer. In literature it isn't written which values (CD/CI) we could use but this parameter is calibration parameter in modeling. In my research (impact of drainage and infiltration parameter to groundwater flow) I came with these results. If the value of CD/CI is high impact of surface water to groundwater is smaller it means time delay in spreading of groundwater and also spread of groundwater level in direction perpendicular to flow retarding. With these knowledge's and with input data (geological, hydro geological, etc.) it will be design mathematic model which will be calibrated and after that verified by exist measurements of surface and groundwater and after that it will be used for groundwater modeling in floods situation on chosen hydrological significant areas – Sihot area (one of the groundwater source for Bratislava) or Sedlackov island which is also one of the groundwater source for Bratislava. These two islands are in Danube river basin and it will be god to know how fast responds groundwater to surface water changes, especially in dramatic increases of surface water which is caused by floods. Based on these results it will be possible more accurate design plan for groundwater pumping which means more effective use of water source.

3 MATHEMATIC MODEL

3.1 Methodic for groundwater flow modeling

Groundwater flow model is abstract model which simulate flow of groundwater in difficult hydro geological conditions [2]. Mathematics modeling is very important in water management. One of the option of modeling is modeling by mathematical model which is based on mathematical equations. By numerical model it is possible understand to flow of groundwater in time and space continuity. Groundwater flow model is imagination of flow in porous space which hydrogeologist use for prognoses. It is consists from two parts. Abstract model (model in mind, graphic imaginations, and expressions of hydro geological situation) and mathematic model which shows hydrogeological process at given area.

3.2 TRIWACO

Has been developed by Royal Haskoning.[3] Since the first version in 1980 it has developed to a modeling environment which is used to support policy development, research and planning in the field of groundwater, surface water and ecology

Triwaco offers an integrated modeling environment for unsaturated, saturated, drainage/infiltration and surface water flow. The modular and open structure assures easy linkage between different models, like SOBEK or ArcView/GIS.

Clear and structured modeling:

Triwaco users enjoy a working environment which allows them to define hydrological situations in physical terms while they can rely on a proper translation of this conceptual model into mathmatics of a numerical engine.

Triwaco is designed for an easy and structured way of moving from a conceptual model to a final calibrated model and further to scenario and postprocessing calculations.

The working environment is designed in such a way that it provides the user with real time information on modelling progress. The user can also readily see if revision of data sets or parameters are needed due to input change, i.e. parameter map modification or parameter allocation.

Since parameters are defined as GIS maps (i.e. independent from a grid) and are stored with the model it is possible to fully reproduce a model and, subsequently, it is easy to make a another model, based on the same initial data but with a different grid

Scenario management:

In most cases a model is used to predict the consequenses of changes made to the watersystem. These changes usually concern only a few model parameters. Therefore **triwaco** has introduced so-called scenario management.

For a particular scenario a data set is created with parameters linked to the final data set. Only parameters that need to be altered for that scenario have to be specified.

The advantage is that for each scenario it is immediately clear which parameters are altered for that particular scenario simulation. Secondly the calibrated model remains intact, thus enhancing the reproducibility of the results.

Modular structure:

One of the distinct advantages of triwaco is its modular structure and flexibility. Various modules are available for:

- grid generation
- surface water flow

- groundwater recharge and unsaturated zone
- groundwater flow in saturated zone

- automatic calibration
- reliability of modelled results
- water balance for random areas in space and time
- path lines and solute transport
- residence times and capture zones of sources and sinks
- automatic determination of groundwater systems

Additional modules calculate:

- (ground)water quality predictions
- surface water quality predictions
- land subsidence
- agricultural damage
- hydro-ecological forecasting (nature and agriculture)

The modular structure assures easy expansion of the capabilities by simply adding a module. Additional modules can be purchased and added at any time. These modules are either standard or may be custom designed for specific modelling requests.

Interaction with groundwater and surface water system

A unique feature of **triwaco** is the so called TOPSYSTEM, which handles the interaction between the groundwater system, unsaturated zone and the surface drainage/infiltration system. Four different drainage levels can be assigned depending on the selected type of ‘topsystem”, each with unique drainage-/ infiltration characteristics.

Watercourses, faults and tubes may be defined by line-elements in multiple layers. In combination with the TOPSYSTEM can simulate virtually any water system. On a regional scale from a delta to karstic terrain and on a local scale from complex drainage systems to entire artificial recharge & recovery schemes.

triwaco offers an open structure for easy linkage between different models within or outside the triwaco working environment . For example the interaction between groundwater and individual watercourses (defined as line elements) can easily be linked to the branches of a hydraulic surface water model. This link is defined conceptually and will work for any numerical discretization.

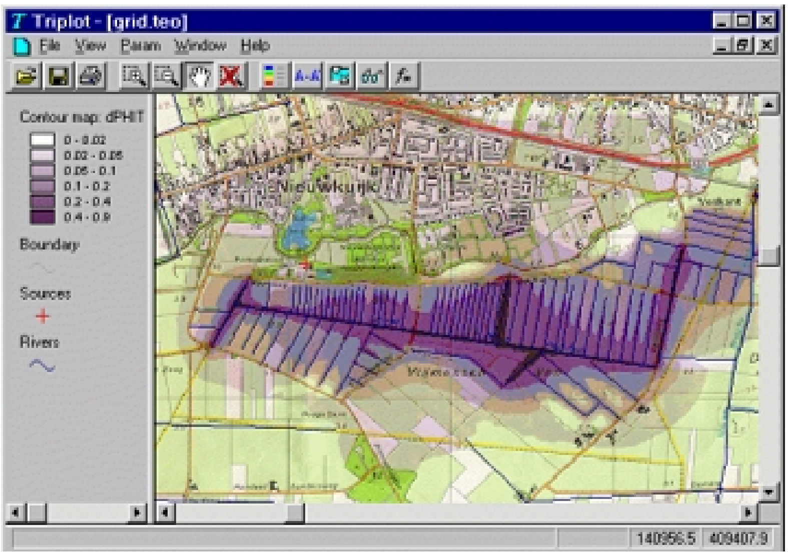

Fig. 1 Examples from Triwaco

4 AREA OF INTEREST

Solution of impact of floods to groundwater is assumed on hydrologically important areas – water sources for Bratislava – Sihot Island (Fig.2) or Sedláčkov Island (Fig.3). Both of them are one of most important water sources of capital town. Because of old and not function monitoring system on Sihot island is more likely to do this research on Sedlackov island where the monitoring system working correctly. But this island is small and here are just four wells and one of they are destroyed. It will be planned to build one more and at these dates are geological and hydrogeological research finished and data are available which is, one of advantage for this area is. Modeling will be provided just fore one of these areas, on which it will be chosen on basis of given background data [4].

Fig.2 Sihoť island

Fig. 3 Sedláčkov Island, scale 1: 65 000 [5] 13

4.1 Groundwater source Sihot Island

Sihot island is (FIG 2, 3) is Danube island which is located in Danube at river kilometer 1872-1877. Is oblong shape with longer dimension of 3,7 km and with maximum width about 1.0 km and with area of 222 ha. From center of Bratislava it is 5 km far upstream Danube River. In present is there 12 wide diameter digged wells (Fig.4) with diameter from 2830mm to 4000mm and one wide diameter well Raney system with diameter of 5000mm and 32 drilled wells with diameter 800 mm and 1000 mm. Collecting pumping station CS3 is another wide diameter well. Water from wels is pumped by pump or syphons. Groundwater source Sihot supply water to west part of Bratislava - Karlova Ves, Dúbravka, Lamač, Devínska Nová Ves, Záhorská Bystrica and part of water to old town. Altitude level of Sihot island is from 135.80 m a.s.l. 139.00 m a.s.l. Island is flooded from 570 cm on Danube at water level in Bratislava [4].

Fig. 4 Wide diameter digged well on Sihot Island

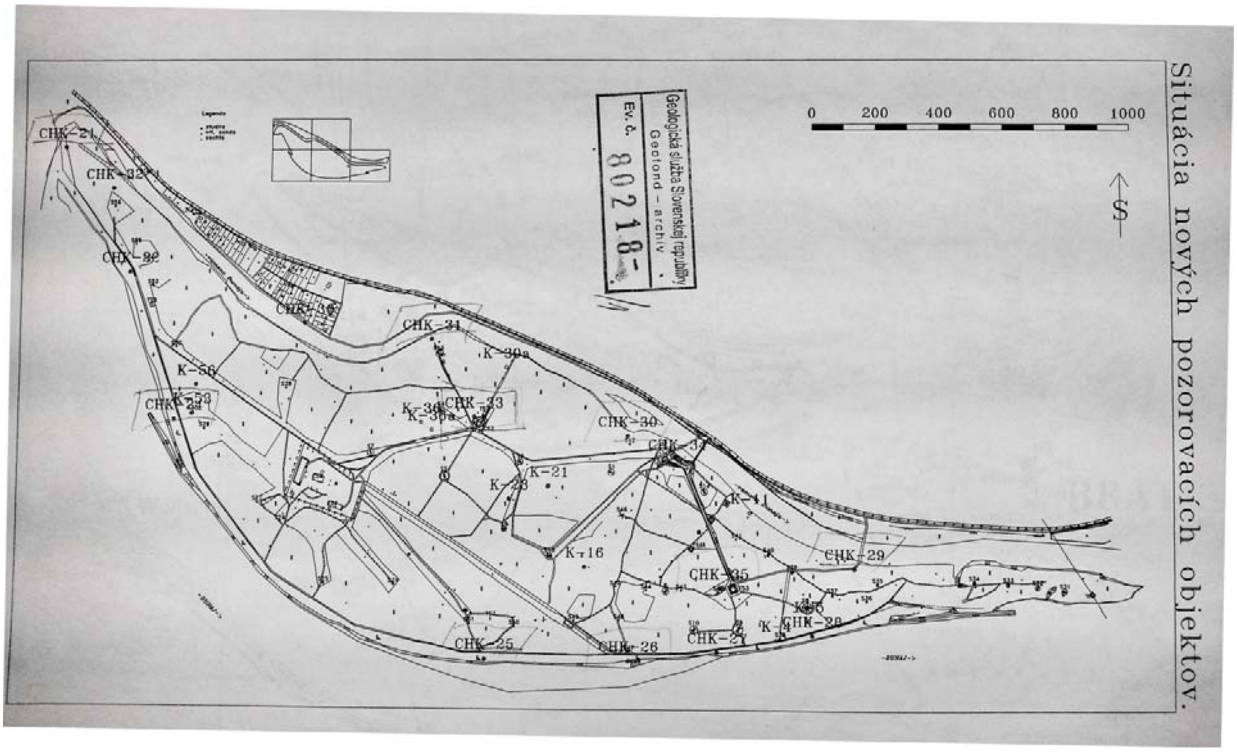

Fig. 5 Situation of monitoring wells on Sihot [5]

4.2 Groundwater source Devín - Sedláčkov Island

Groundwater source Sedláčkov Island (Fig.6) is located on left bank of Danube river under part of town called Devin. Sedlackov Island is oblong shape 800 m long and 350 m wide, with area about 15 ha. There is four wide diameters drilled wells with diameter 820 mm. and one more is designed jet (Fig.7). Depth of wells is 10 m from terrain. Water is pumped by submersible pumps. Sadláčkov island supply water to Devin and Devínska nova Ves [4].

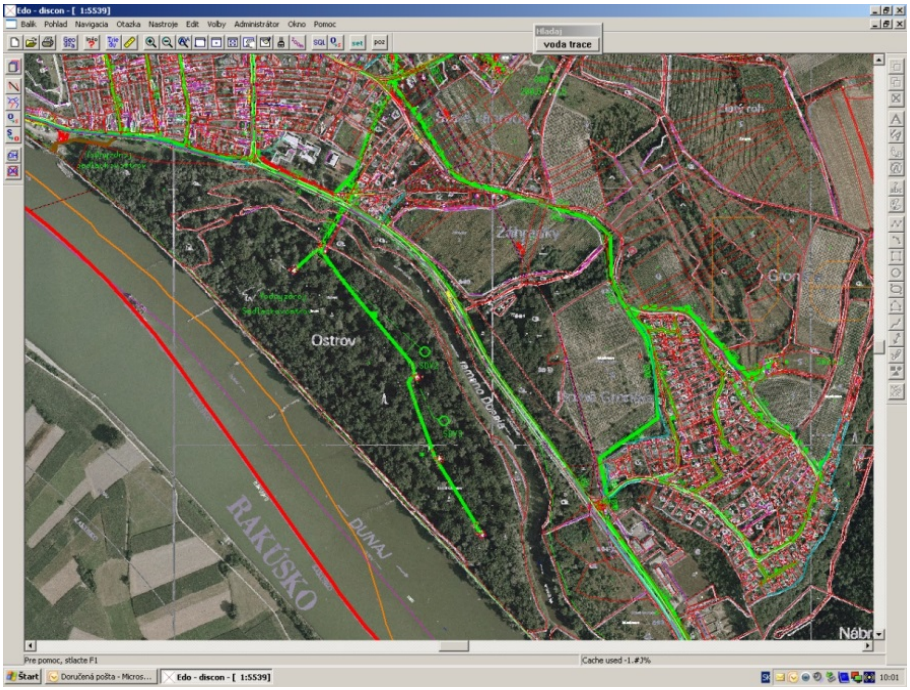

Fig.6 Groundwater source Sedláčkov Island

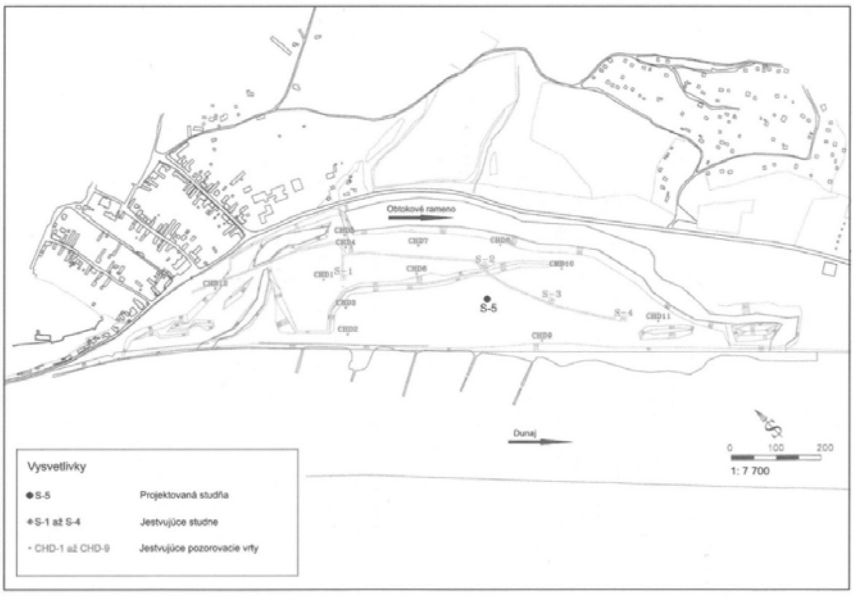

Fig. 7 Situation of designed well S5 in scale 1:7700 [5]

5 SOLUTIONS

Impact of floods to groundwater regime will be solved by TRIWACO modeling program. As was mentioned in part 3 TRIWACO is groundwater modelling program which is ideal for this problematic.

Solution will followed these steps:

1. Description of chosen area and processing of input data
2. Definition of boundaries of filtration and boundary conditions
3. Setup and calibrating model
4. Verify the model
5. Calculation
6. Evaluation of modeled results

Description of chosen area and processing of input data – choosing of area, description: geological, hydrogeological, hydrological, etc.

Definition of boundaries of filtration and boundary conditions – at first it is necessary from hydrological view correctly set up boundaries of area – which create a polygon for where will simulation run. Type of boundary condition will be chosen from hydrogeological data and measurements.

Setup and calibrating model - On given data (geographical, hydrogeological, hydrological, etc.) will be created model for fictious flow in Danube River and based on this calculations model will be calibrated for best image of reality.

Verification of model - Based on previous calculations (calibration) will be chosen specific time interval. On this specific time interval will run calculations again and it will be compared with measurements (verification)

Calculations - On verificated model we could run simulations for different water levels in Danube River

Evaluation of modeled results – Outputs from TRIWACO will be used for prognoses of groundwater flow in flood situations at Danube River. Outputs will be used for design of operation plans for mentioned water sources. One of the outcomes will be maps of threads in flood situations

6 CONCLUSION

On given geological, hydrogeological ant topographical data will be composed in TRIWACO modeling program chosen area. On this area will be model calibrated and verificated. Verification will be on dataset what will be obtained when the floods will come. Will be chosen specific time interval for verification. For this time interval will be obtained hydrological data from SHMU, hydrogeological data from BVS wells. At this time we don't know which wells are working correctly therefore we have to test BVS wells which are working and which not. After verification will be model ready for modeling in specific range. It will be assumed another verification on different dataset. After this model will be ready for research of groundwater flow in floods situations.

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PROPERTIES OF ONE DIMENSIONAL OPEN-CHANNEL STEADY FLOW EQUATIONS

Wojciech Artichowicz[1] and Romuald Szymkiewicz[2]

ABSTRACT

In this paper properties of discrete forms of one dimensional steady gradually varied flow equations are discussed. Such forms of flow equations are obtained as a result of approximation of their differential forms, which is required to solve them numerically. For such purpose explicit or implicit numerical approximation schemes for ordinary differential equations can be applied. It turns out that dependently on the chosen approximation scheme, discrete forms of steady gradually varied flow equations can have more than one root. This property can lead to major issues during process of numerical solution of steady flow equations, as in such situation the choice of proper root is crucial to the obtained result.

Standard steady gradually varied flow equation, energy equation and steady Saint-Venant equations were examined from the viewpoint of mentioned properties.

Keywords
Open channel flow, steady gradually varied flow, one dimensional, flow equations, energy equation

1. INTRODUCTION

Steady gradually varied flow (SGVF) is one of the classical problems considered in open channel hydraulics. Commonly two equations are used for the purpose of one dimensional open channel flow modelling: so-called standard SGVF equation used for prismatic channels and discrete form of energy equation, which in fact is Bernoulli's

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principle, used for computations in natural channels ([1], [2], [3], [4], [5]). Mentioned equations can be derived from general model describing one dimensional open channel flow known as de Saint-Venant system of equations:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = 0 , \tag{1}$$

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left(\frac{\beta \cdot Q^2}{A}\right) + g \cdot A \frac{\partial h}{\partial x} = -g \cdot A \cdot S , \tag{2}$$

where used symbols denote the following quantities:
A – wetted cross-sectional area,
Q – flow discharge,
h – water stage,
t – time variable,
x – spatial variable,
β – momentum correctional coefficient,
g – gravitational acceleration.

The energy slope S can be expressed with Manning formula as follows:

$$S = \frac{n^2 \cdot Q^2}{A^2 \cdot R^{4/3}} , \tag{3}$$

where n is Manning roughness coefficient and R is hydraulic radius.

If flow is steady, then time derivatives vanish, giving in result the following system of so-called steady state Saint-Venant equations describing SGVF flow in open channels:

$$\frac{dQ}{dx} = 0 , \tag{4}$$

$$\frac{d}{dx}\left(\frac{\beta \cdot Q^2}{A}\right) + g \cdot A \frac{dh}{dx} = -g \cdot A \cdot S . \tag{5}$$

Equation (4) simply states that flow discharge is constant and can be included in further derivations as $Q = const.$.

The system of equations (4) and (5) can be used directly for SGVF computations without any further transformations ([6]). However commonly it is transformed into differential form of energy equation ([5]):

$$\frac{d}{dx}\left(h + \frac{\alpha \cdot Q^2}{2g \cdot A^2}\right) = -S , \tag{6}$$

with α denoting Coriolis energy correctional coefficient.

If prismatic channels are taken into consideration due to simplifications, energy equation can be transformed into standard form of SGVF equation ([3], [5]):

$$\frac{dH}{dx} = \frac{s - S}{1 - \alpha \cdot Fr^2} , \tag{7}$$

where H is water depth and s denotes channel bed slope. The Froude's number Fr is expressed with following formula:

$$Fr = \frac{U}{\sqrt{g \cdot H}},$$

in which U denotes the average flow velocity.

If flow discharge is constant, equations (5) and (6) can be used for flow computations in any type of channel, regardless if it is natural or prismatic one. When applied to prismatic channel equations (5), (6) and (7) can be used interchangeably. In [5] Szymkiewicz pointed out that dependently on stepsize discrete form of energy equation can have multiple roots, which is a very important property from computational point of view, and may cause serious issues during numerical solution process. As presented, SGVF equations have the common origin, therefore a question arises if difference approximations of Eqs. (5), (6) and (7) have similar properties.

2. NUMERICAL SOLUTION OF INITIAL PROBLEM FOR SGVF EQUATIONS AND THEIR DISCRETE FORMS

If flow discharge is known, initial value problem for each of SGVF equations can be formulated. In such case function describing water level and fulfilling given initial condition is to be determined. Eq. (5) requires known water stage $h(x_0)=h_0$ to be imposed as an initial condition. Energy equation requires energy stage $E(x_0) = E_0$ to be given as an initial condition, whereas standard equation needs known water depth $H(x_0) = H_0$. Although Eqs. (5), (6) and (7) require different type of initial condition, the required information is in fact identical, that is water stage or water depth in the first or last channel cross-section and flow discharge.

To obtain solution of any of mentioned differential equations, numerical methods are necessary to be involved. Energy equation and standard equation can be solved using common methods like Euler's method, implicit trapezoidal or Runge-Kutta methods. Whereas steady de Saint-Venant equation can be solved using finite differences method. Usually to approximate standard or energy equation implicit trapezoidal rule is used. However to obtain more general approximation than one provided by implicit trapezoidal rule, general two level formula will be used. To introduce this formula consider general ordinary differential equation:

$$\frac{dy(x)}{dx} = y'(x, y), \tag{8}$$

where:
$y(x)$ – unknown function of independent variable,
x – independent variable,
$y'(x,y)$ – derivative of $y(x)$ function.

General two level formula approximating ordinary differential equation (8) has the following form ([7]):

$$y_{i+1} = y_i + \Delta x_i \cdot ((1-\theta) \cdot y'(x_i, y_i) + \theta \cdot y'(x_{i+1}, y_{i+1})), \tag{9}$$

where θ is weight parameter taking values from <0;1> interval, i is the node index and Δx_i denotes i^{th} integration step size. This formula allows to easily switch between approximation methods by changing weight parameter value. For example with θ=0 it becomes explicit Euler's formula, with θ=0.5 - implicit trapezoidal rule and with θ=1- implicit Euler's formula.

Applying general two level formula to energy equation in its differential form one obtains:

$$h_{i+1} + \frac{\alpha \cdot Q^2}{2g \cdot A_{i+1}^2} = h_i + \frac{\alpha \cdot Q^2}{2g \cdot A_i^2} + \Delta x_i \cdot \left(-(1-\theta)\cdot S_i - \theta \cdot S_{i+1}\right). \tag{10}$$

When implicit trapezoidal rule is used, that is weighting parameter takes value θ=0.5, resulting discrete form of energy equation coincides with commonly used application of Bernoulli principle to SGVF modelling which is known as standard step method ([1], [2]). Graphical interpretation of Eq. (10) is shown in Fig. 1.

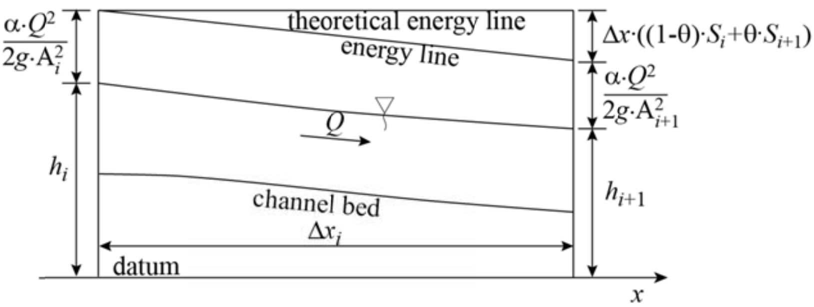

Fig. 1 Sketch of considered channel reach.

Approximating equation (7) with general two level formula yields:

$$H_{i+1} = H_i + \Delta x_i \cdot \left((1-\theta)\frac{s - S_i}{1 - \alpha \cdot Fr_i^2} + \theta \frac{s - S_{i+1}}{1 - \alpha \cdot Fr_{i+1}^2} \right). \tag{11}$$

To obtain discrete form of Eq. (5) finite differences method can be used. Let the point of approximation P be set as shown in the Fig. 2.

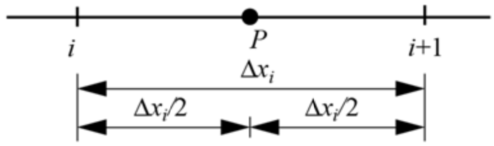

Fig. 2 Discretization scheme, P is the point of approximation.

Such an approximation can be regarded as a method equivalent to the implicit trapezoidal rule. After applying this scheme to Eq. (5) its discrete form is obtained:

$$\frac{1}{g \cdot A_P} \frac{\beta \cdot Q^2}{\Delta x_i} \left(\frac{1}{A_{i+1}} - \frac{1}{A_i} \right) + \frac{1}{\Delta x_i}\left(h_{i+1} - h_i\right) + S_P = 0 \tag{12}$$

with:

$$A_P = \frac{1}{2}(A_i + A_{i+1}) \tag{13}$$

$$S_P = \left(\frac{Q^2 \cdot n^2}{A^2 R^{4/3}}\right)_P = \frac{1}{2}\left[\left(\frac{Q^2 \cdot n^2}{A^2 R^{4/3}}\right)_i + \left(\frac{Q^2 \cdot n^2}{A^2 R^{4/3}}\right)_{i+1}\right] \tag{14}$$

To obtain solution of discussed ordinary differential equations, it is necessary to solve resulting non-linear algebraic Eqs. (10), (11) and (12) of unknown water stage or depth. Regarding direction of integration two cases should be considered. First case is typical for computations performed when supercritical flow is to be modelled. In such case integration direction is the same as the flow direction and unknown water stage or depth values are indexed as i+1. Such case occurs when water surface profile behind drainage is to be modelled. The second situation is typical for subcritical flow computations, which are performed in the direction opposite to flow. An example of such situation is met when water surface profile before a dam is being computed. In such case unknown values are indexed as i, and in turn weighting parameter values result in different approximation schemes: for θ=0 it becomes implicit Eulers formula, for θ=1 - it becomes explicit Eulers formula. For θ=0.5 resulting approximation scheme remains unchanged that is implicit trapezoidal rule.

1 3. ANALYSIS OF DISCRETE FORMS OF SGVF

Let us consider two neighbouring cross-sections of rectangular channel. For convenience of further analyses water stage will be expressed as a sum of water depth and bottom level, that is h=H+z and datum level will be assumed at the bottom level of cross-section indexed as 1 (Fig. 3). Additionally constant bed slope s is assumed within this channel reach, which allows to express channel bed stage in second cross-section as z_2=-$\Delta x \cdot s$.

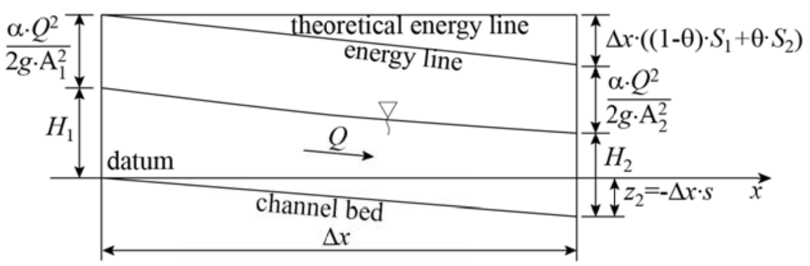

Fig. 3 Sketch of considered channel reach.

This allows to use in Eqs. (10) and (12) water depth as an unknown variable instead of water stage, which is more suitable for performing analyses.

Let us assume subcritical flow, then integration direction is opposite to flow, therefore discrete forms of flow equations can be expressed as functions of unknown water depth in cross-section denoted with index 1:

$$f_E(H_1) = H_2 - \Delta x \cdot s + \frac{\alpha \cdot Q^2}{2g \cdot A_2^2} - \left(H_1 + \frac{\alpha \cdot Q^2}{2g \cdot A_1^2}\right) - \Delta x \cdot \left(-(1-\theta) \cdot S_1 - \theta \cdot S_2\right), \tag{15}$$

$$f_S(H_1) = H_2 - H_1 - \Delta x \cdot \left((1-\theta)\frac{s - S_1}{1 - \alpha \cdot Fr_1^2} + \theta \frac{s - S_2}{1 - \alpha \cdot Fr_2^2}\right), \tag{16}$$

$$f_M(H_1) = \frac{1}{g \cdot A_P} \frac{\beta \cdot Q^2}{\Delta x} \left(\frac{1}{A_{i+1}} - \frac{1}{A_i} \right) + \frac{1}{\Delta x} \left((H_2 - \Delta x \cdot s) - \cdot H_1 \right) + S_P, \qquad (17)$$

where f_E denotes discrete form of energy equation, f_S denotes discrete standard equation and f_M means the discrete steady de Saint-Venant equation.

To analyse functions listed above let us define an example in which a rectangular channel of width B=4 m and roughness coefficient n=0.03 s/m$^{1/3}$ is considered. Known flow parameters are constant discharge Q=4 m^3/s and water depth in the last cross-section H_2=1 m, which is greater than critical depth H_c=0.467 m. Coriolis coefficient is also assumed to be constant and equal to α=1. Flow in the channel is subcritical. For functions (15), (16) weighting parameter value θ=0.5 was assumed. In Figs. 4, 5 and 6 plots of functions f_E, f_S and f_M respectively are displayed. It is remarkable that dependently on the size of integration step Δx, each function can have multiple zeros. It can be noticed that functions resulting from discretization of energy equation and momentum equation differ slightly, and their shape changes in the same way with the change of Δx value. As for discrete form of standard equation it can be noticed that it also has variable number of roots, but the shape of the function differs from the shapes of discrete energy and momentum equations. It has up to two roots in the range of supercritical depths (H_1<H_c). In the range of subcritical depths it has always one root.

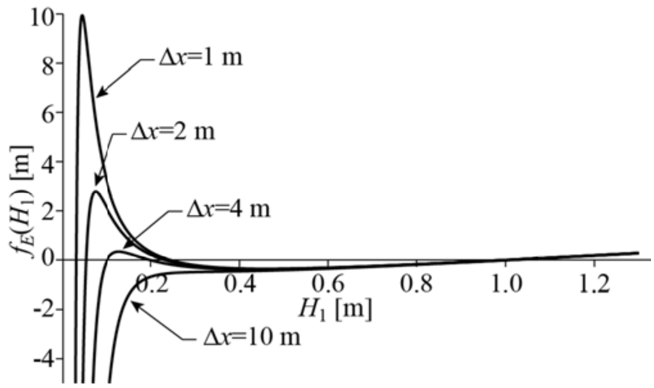

Fig. 4 Plot of $f_E(H_1)$ for different Δx values.

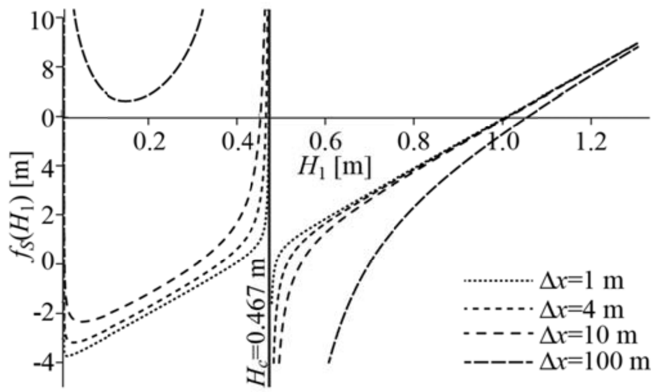

Fig. 5 Plot of $f_S(H_1)$ for different Δx values.

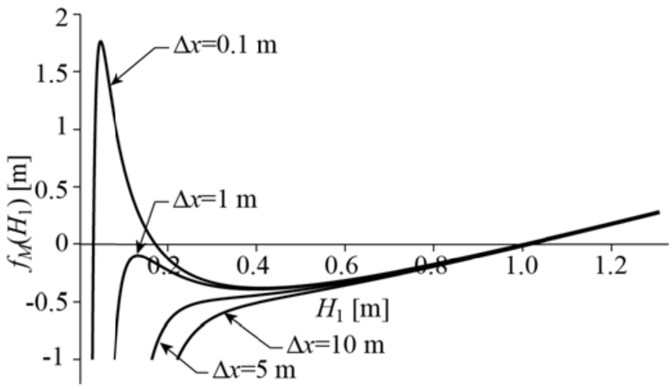

Fig. 6 Plot of $f_M(H_1)$ for different Δx values.

Consider identical example as previously, but this time constant integration step value Δx will be taken and the influence of weight parameter θ on the functions (15) and (16) will be examined. The results are displayed in Fig. 7, where it can be noticed that when using explicit Euler's formula (θ=1) function f_E has always two roots, regardless if stepsize value is small (Δx=1 m) or big (Δx=50 m). When implicit formulas are used number of roots of discrete form of energy equations varies with the stepsize value.

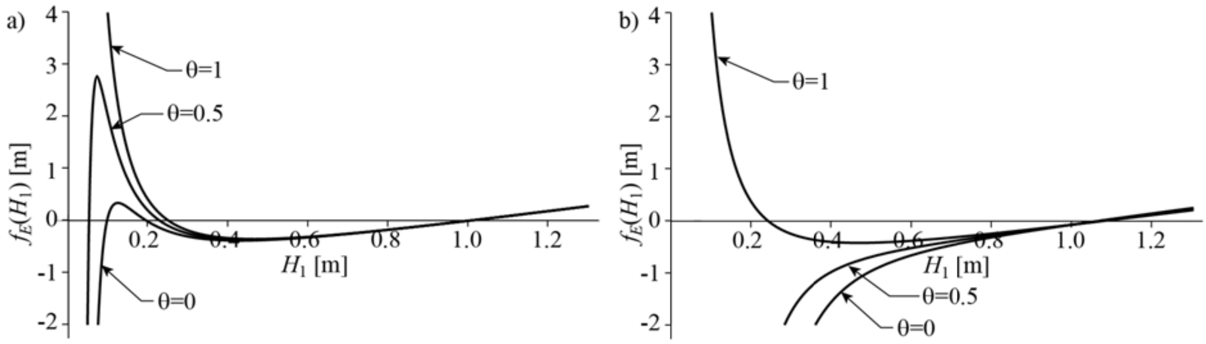

Fig. 7 Influence of θ value on function $f_E(H_1)$: a) Δx=2 m, b) Δx=50 m.

Similar behaviour can be seen in Fig. 8 displaying discrete form of standard equation. However when explicit approximation formula is used f_S becomes linear function of unknown variable H_1 regardless to integration step. When an implicit formula is used number of zeros of obtained function also varies with size of the Δx value.

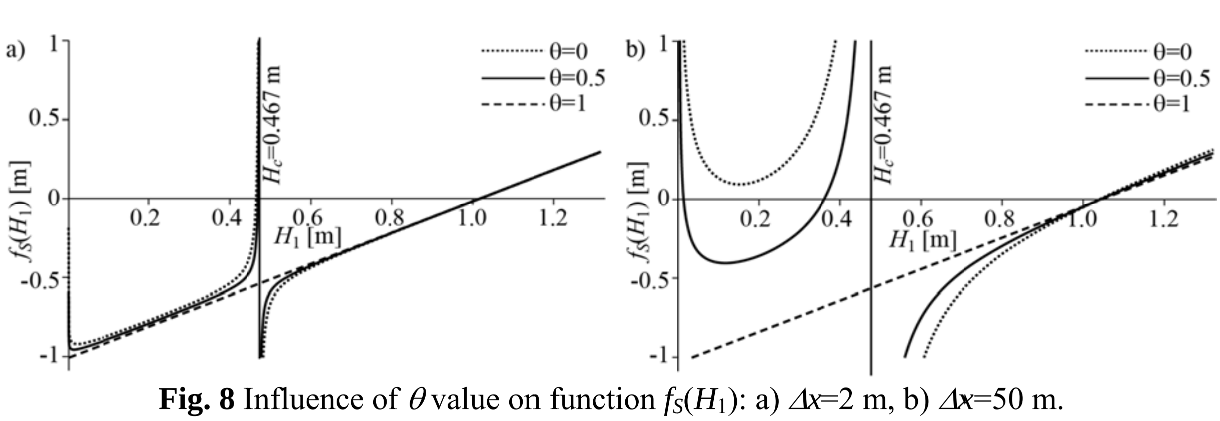

Fig. 8 Influence of θ value on function $f_S(H_1)$: a) Δx=2 m, b) Δx=50 m.

When supercritical flow is considered integration direction is the same as the flow direction. Therefore, although mentioned functions are the same, they become the functions of unknown water depth in the cross-section denoted as 2, that is H_2:

$$f_E(H_2) = H_2 - \Delta x \cdot s + \frac{\alpha \cdot Q^2}{2g \cdot A_2^2} - \left(H_1 + \frac{\alpha \cdot Q^2}{2g \cdot A_1^2} \right) - \Delta x \cdot \left(-(1-\theta) \cdot S_1 - \theta \cdot S_2 \right) \tag{18}$$

$$f_S(H_2) = H_2 - H_1 - \Delta x \cdot \left((1-\theta) \frac{s - S_1}{1 - \alpha \cdot Fr_1^2} + \theta \frac{s - S_2}{1 - \alpha \cdot Fr_2^2} \right) \tag{19}$$

$$f_M(H_2) = \frac{1}{g \cdot A_P} \frac{\beta \cdot Q^2}{\Delta x} \left(\frac{1}{A_{i+1}} - \frac{1}{A_i} \right) + \frac{1}{\Delta x} \left((H_2 - \Delta x \cdot s) - \cdot H_1 \right) + S_P \tag{20}$$

Let us perform similar analyses as before. To this order let the example remain unchanged. However direction of integration is different and therefore water depth at cross-section 1 has to be imposed, assumed value is H_1=0.2 m. Imposed depth is less than critical depth. Functions described by Eqs. (18), (19) and 20 are displayed in Figs. 9,10 and 11. Plots show their variability due to integration step changes. Each function has two zeros, regardless to Δx value. Discrete forms of energy equation and momentum equation again have shape similar to each other, and their behaviour to stepsize value change is similar as well. The implicit approximation of standard equation also has only two roots, however both of them are on the left hand side of the asymptote placed in the value of critical depth, which suggests that one of the roots may be unphysical.

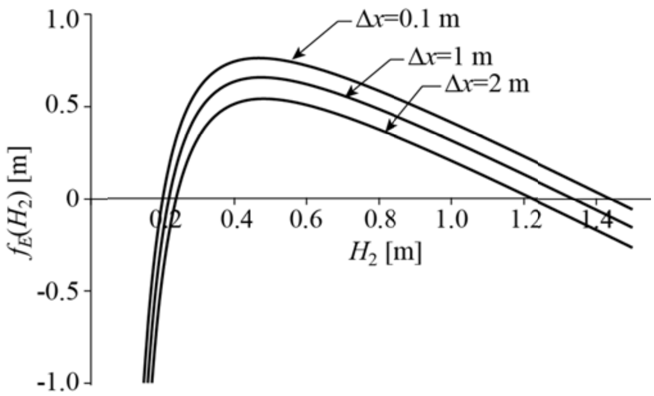

Fig. 9 Plot of $f_E(H_2)$ for different Δx values.

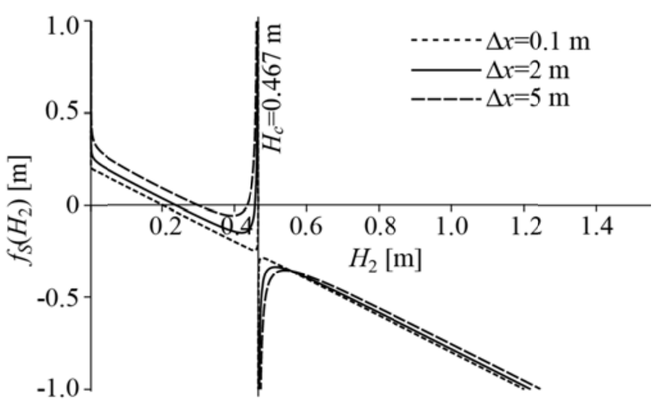

Fig. 10 Plot of $f_S(H_2)$ for different Δx values.

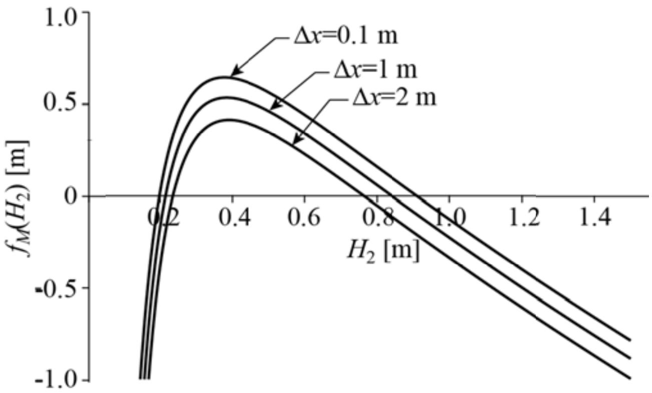

Fig. 11 Plot of $f_M(H_2)$ for different Δx values.

Again let us examine the influence of approximating scheme on the resulting difference form of energy or standard equation. Fig. 12 shows that the function does not change its character regardless of weight parameter value and has two zeros.

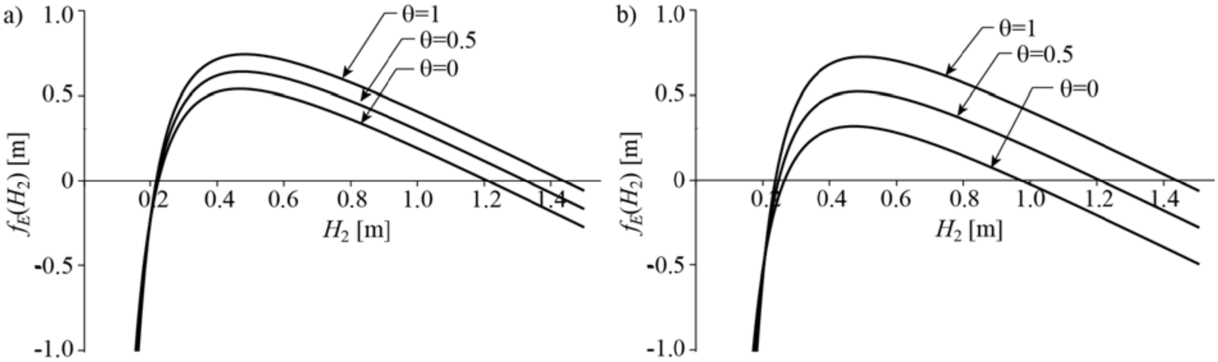

Fig. 12 Influence of θ value on function $f_E(H_1)$: a) Δx=1 m, b) Δx=2 m

Discrete form of standard SGVF equation however does change according to weight parameter value. When explicit Euler's formula is used it remains linear function of the unknown variable. If implicit formula is used function becomes non-linear and has to two roots when supercritical flow is considered (Fig. 13).

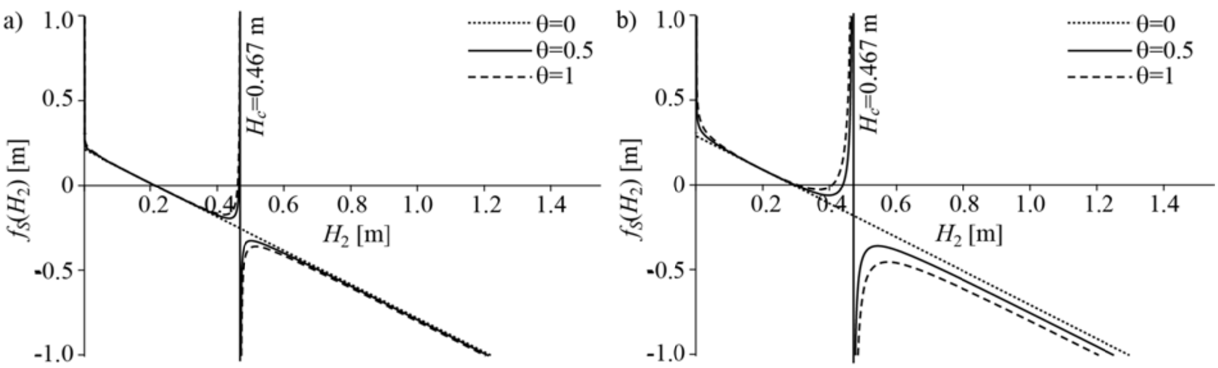

Fig. 13 Influence of θ value on function $f_S(H_1)$: a) Δx=1 m, b) Δx=5 m

2 AS IT WAS MENTIONED IN THE INTRODUCTION, ALL THREE SGVF DIFFERENTIAL EQUATIONS HAVE THE SAME ORIGIN. EVEN MORE, ENERGY

EQUATION IS DERIVED FROM MOMENTUM EQUATION WITHOUT ANY ADDITIONAL ASSUMPTIONS, AND ALSO STANDARD EQUATION IS DERIVED FROM ENERGY EQUATION WITHOUT ANY ADDITIONAL ASSUMPTIONS NOR SIMPLIFICATIONS. THEREFORE TO INVESTIGATE THE ROOT OF PROPERTIES OF SGVF EQUATIONS IT IS ENOUGH TO EXAMINE ONE OF THEM. CONSIDER EQ. (15) WRITTEN IN THE FOLLOWING MANNER:

$$\left(H_2 + \frac{\alpha \cdot Q^2}{2g \cdot A_2^2}\right) - \left(H_1 + \frac{\alpha \cdot Q^2}{2g \cdot A_1^2}\right) - \Delta x \cdot s - \Delta x\left(-(1-\theta)\cdot S_1 - \theta \cdot S_2\right). \tag{21}$$

Let us recall now an equation describing cross-sections specific energy (Chanson, 2004):

$$SE = H + \frac{\alpha \cdot Q^2}{2g \cdot A^2}. \tag{22}$$

On the plot of the specific energy of a cross section can be noticed, that there are two depths for which energy has the same stage – depth of rapid flow H_R and depth of tranquil flow H_T (Fig. 14).

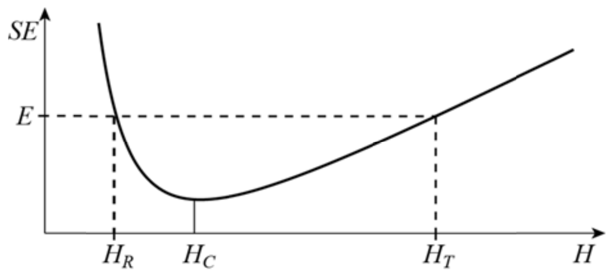

Fig. 14 General plot of specific energy of a simple cross-section

This property is also present in discrete energy equation as one of its terms is specific energy of the cross-section at which unknown water depth is to be estimated. That also leads to a conclusion that two roots are physical. When integration is done along with flow direction energy equation always has two roots, regardless to stepsize or approximation scheme. However when integration is done in the direction opposite to flow, the case when Eq. (15) has two roots occurs only when explicit approximation schemes are used. If implicit methods are used, dependently on the integration step, discrete energy equation has variable number of roots. To investigate the reason of such situation let us assume that unknown depth H_1 is being sought, and reorder Eq. (21):

$$f_E(H_1) = \underbrace{-SE_1 + \Delta x \cdot (1-\theta)\cdot S_1}_{\text{variable}} + \underbrace{SE_2 - \Delta x \cdot s + \Delta x \cdot \theta \cdot S_2}_{\text{constant}}. \tag{23}$$

It can be noticed that Eq. (23) consists of terms that are dependent on the unknown depth, and independent ones which are in such case constant. If weighting parameter θ is equal to 1, then the term $\Delta x \cdot (1-\theta)\cdot S_1$ vanishes and the function is just specific energy of the cross-section 1 translated by the constant. That explains the fact why it has always two roots – with θ=1 Eq. (23) straightforwardly can be interpreted as specific energy translated by a constant, and therefore corresponds to situation shown in the Fig. 17. If θ<1 then discussed term does not vanish and may change the shape of function (23). To simplify analysis of the influence of this term let us take θ=0, which will result in following function:

$$f_E(H_1) = \underbrace{-SE_1 + \Delta x \cdot S_1}_{\text{variable}} + \underbrace{SE_2 - \Delta x \cdot s}_{\text{constant}}. \tag{24}$$

The roots of equation $f_E(H_1)=0$ are sought, therefore one can reorder Eq. (24) to obtain:

$$SE_1 + \Delta x \cdot s - SE_2 = \Delta x \cdot S_1. \tag{25}$$

It can be noticed that Eq. (25) has solutions in points where the function consisting of first cross-sections specific energy translated by a constant value (left hand side of Eq. (25)) intersects the function describing energy slope which is friction loss multiplied by integration step value (right hand side of Eq. (25)). To investigate the reason of another root appearance let us plot the terms of Eq. (25) separately (Fig. 15). To this order previously defined example data will be used. For stepsize value Δx=2 m functions have three intersection points, for Δx=10 m one such point exists. As term S_1 is multiplied by integration step value, Δx can be treated as multiplier influencing the shape of this function, and thus influencing the number of intersection points.

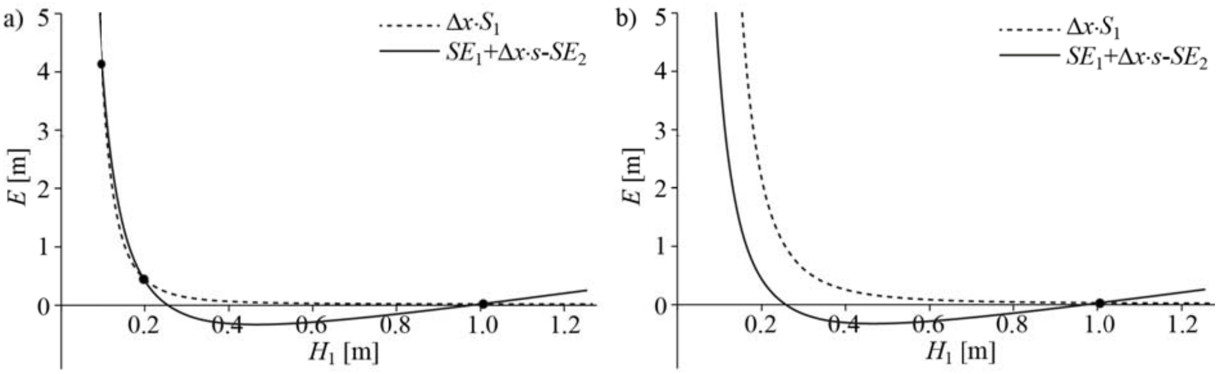

Fig. 15 Plots of the terms consisting to Eq. (24), black dot denotes intersection points: a) Δx=2 m, b) Δx=10 m

This additional root appears only when implicit approximation schemes are used, and has no theoretical background in hydraulics theory, therefore should be regarded as unphysical ([8]). Also vanishing of the root corresponding to rapid flow for big stepsize values can be treated as purely mathematical effect caused by strongly non-linear term representing friction loss.

3 4. THE CHOICE OF THE PROPER ROOT

If an equation describing physical process has more than one solution a question about the proper choice of one of them arises. In case of SGVF equations two physical roots exist which correspond to rapid and tranquil flow conditions. Therefore for proper choice of root an expected type of water surface profile or flow conditions on considered channel reach should be known.

Using implicit approximation schemes increases the accuracy of computations, but when integration is performed in a direction opposite to flow, discrete form of energy equation has variable number of roots dependently on integration step value. Such property is a major issue entangling process of numerical solution. However explicit approximations of energy equation always have two roots, therefore a predictor-corrector approach seems a valid way to solve this class of problems. In prediction step roots of explicit-scheme approximation should be found, and it should be decided if root corresponding to sub- or supercritical flow is sought. If channel cross-section geometry can be expressed with an equation, roots of explicit approximation of energy equation can be usually found analytically (for rectangular cross-section Cardano formula can be used). If a cross-section is of natural type, and is described with a set of coordinates, numerical methods of solving non-linear algebraic equations have to be used for this purpose anyway. But in such case the number of possible solutions is known and the fact that one root lies below critical depth and second is placed above it can be used to improve the process of prediction. The roots of discrete form of energy equation obtained by explicit scheme approximation should be used as initial values for numerical solution of implicit approximation of energy equation.

4 5. COMPUTATIONAL EXAMPLE

Consider following example in which SGVF flow behind a dam occurs and computations are performed using standard step method (θ=0.5). Rectangular channel has length of L=2000 m, bed slope s=0.001, and width B=4 m. Material of which the channel is constructed has Manning roughness coefficient equal to n=0.03 s/m$^{1/3}$, flow discharge in the channel is equal to 3 m^3/s. Coriolis coefficient is assumed to be constant and equal to α=1. For the purpose of computations stepsize Δx=2 m was taken. Obtained solution is shown in Fig. 16. Due to random choice of roots of algebraic non-linear equation at one of the cross-sections, the solution became unphysical.

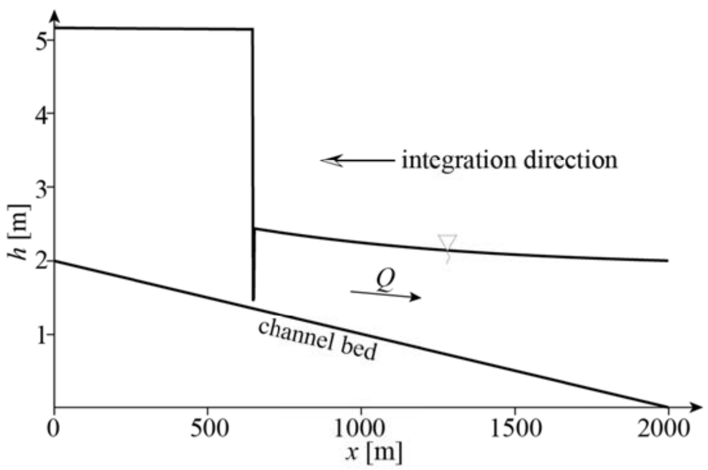

Fig. 16 Unphysical flow profile obtained by improper choice of root

In open channel hydraulics a case is commonly met, when SGVF regime changes from tranquil to rapid as a result of channel bed slope change. An experiment was performed in which such situation has been reproduced in laboratory flume. To this order a rectangular channel of width B = 0.40 m and of length equal to 10 m with adjustable longitudinal slope was used (Fig. 17).

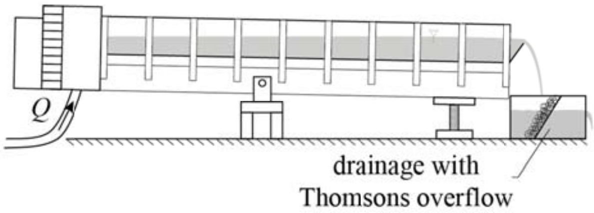

Fig. 17 Laboratory flume used to perform the physical experiment

The uniform longitudinal bed slope of the channel was locally modified to arrange the required conditions for passing from the subcritical to supercritical flow. To obtain such flow conditions, an element shown in Fig. 18 was installed inside the flume. This element narrowed the flume so that the width of channel reach at which it was installed was equal to B=0.38 m. The Manning roughness coefficient was evaluated as equal to n = 0.010 s/m$^{1/3}$.

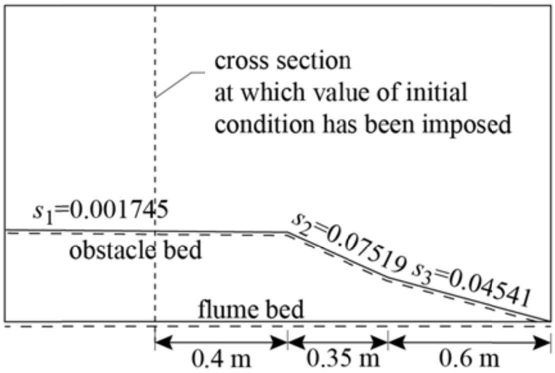

Fig. 18 Sketch of channel section with variable slope installed in the laboratory flume

In the experiment following flow parameters were measured: flow discharge Q = 16.903 dm^3/s, water stage at the upstream end h_0 = 16.4 cm, flow depth at the upstream end H_0 = 6.5 cm. Estimated critical depth was H_c = 6.0 cm.

The equation of mechanical energy was solved using the implicit trapezoidal rule with Δx=0,0125 m. The initial condition was imposed in the cross-section located at x=0.6 m and moreover it was assumed that over the first section of channel with bed slope equal to s_1 a subcritical flow takes place (Fig. 18). Consequently, during computations along this section the root corresponding to the subcritical depth was taken. For the next sections, having the bed slope s_2 and s_3 respectively, supercritical flow has been assumed and consequently the root lying below critical depth was chosen. In Fig. 19 the results of physical experiment are compared with ones provided by computations. As one can see, the agreement of both flow profiles is quite satisfying.

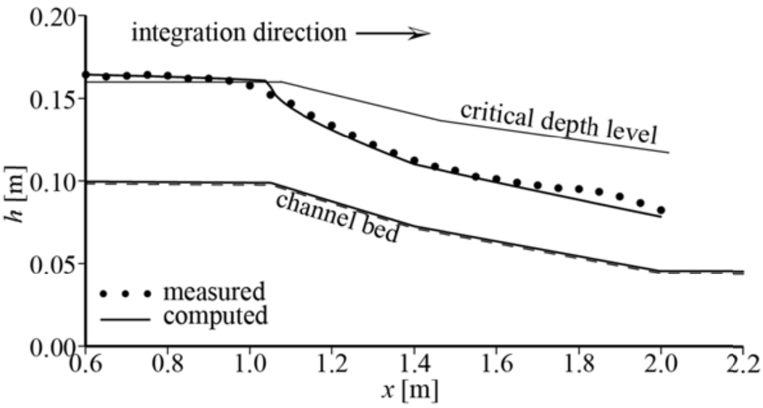

Fig. 19 Flow profiles observed and computed in the flume with variable bed slope

5 6. CONCLUSIONS

In this paper it was shown that discrete form of each of the SGVF equations has more than one root. Two roots are physical and correspond to depths at rapid and tranquil flow conditions, and one unphysical root may appear. In this sense all of those equations have identical properties. Although all three equations have the same origin and can be treated as identical in strictly mathematical sense, it was exposed that discrete form of standard flow equation used for prismatic channels differs significantly from discrete forms of momentum and energy. Therefore rather momentum or energy equation should be considered as basic model for one dimensional SGVF modelling, as standard equation cannot be used for natural channels or in cases where flow passes through critical one. On the other hand energy equation can be approximated with any formula for solving ordinary differential equations, while momentum equation can be discretised only with finite differences or similar type method. Thus energy equation seems the most useful model.

The number of possible solutions of energy equation varies due to changes of stepsize only when implicit approximation schemes are used and the integration direction is opposite to the direction of flow. If explicit scheme is applied to approximate differential energy equation, its discrete form always has two roots. Therefore a good choice for solving problems of SGVF modelling in open channels seem to be predictor-corrector methods.

The reason why discrete form of energy equation has variable number of roots when implicit approximation formula is used is the influence of strongly non-linear term describing friction loss and is purely a mathematical effect.

Cautious choice of the root of discrete form of energy equation allows to compute the flow profile with passing through critical depth without the necessity of imposing two initial conditions.

The final conclusion is that the choice of root of discrete forms of SGVF equations has to be done very thoroughly as the wrong choice may lead to unphysical solutions.

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IMPACT OF EXTREME HYDROLOGICAL CONDITIONS ON GROUNDWATER LEVEL REGIME IN URBAN AREAS – CASE STUDY IN SMALL CARPATHIANS

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Abstract

The development of hydrological conditions in Small Carpathian region in last years (2006, 2010, 2013) has caused an unfavourable increase of groundwater level regime mostly in spring time followed by flooding of cellars of houses in urban regions situated at the foot of a mountain. This problem raised a lot of emotion among people whose houses were built in the vicinity of small creeks thinking that these are the reason for their problem.

The contribution is dealing with an analysis of such a case study in Trnava town where houses in the vicinity of a small Trnavka flow were affected by a groundwater surplus. Unfortunately, in the same time the Water Board Authority has finished one kilometre long river training on this stream and inhabitants of this region have thought that this is the reason for their situation not taking any hydrological conditions development into account. The detailed hydrological analysis based on precipitation and groundwater level data from Slovak Hydro-meteorological Institute and on results of proposed monitoring observation borehole system has shown the real reason for the arisen problem.

Keywords

Extreme hydrological conditions, monitoring system, groundwater level regime, Water Board Authority, TRIWACO

1 INTRODUCTION

The reason for the research review was the evaluation of the impact of a part of the Trnávka River reconstruction on groundwater level regime increase in adjacent area – especially in so called Vajslova Valley and with that connected water surplus in cellars of houses built up in mentioned region. After analysing the background and input documents, especially the analysis of the colleagues from the Department of Geotechnical Engineering we came to decision that the problem has to be solved in broader consequences.

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Therefore we have collected all available meteorological and hydrological data in the solved region, for all precipitation data as well as groundwater level courses in the basic observation network of the Slovak Hydrometeorological Institute (SHMI) in this region. Illustration of the position of observation boreholes of SHMI is shown in Fig.1. It is apparent that there is no observation borehole in the vicinity of the mentioned stream and therefore we were concentrated on the borehole No.42 Hrnčiarovce which is close to a parallel Parná stream. Water levels in both streams (Trnávka, Parná) are controlled by water reservoirs which are located under the Small Carpathian mountains.

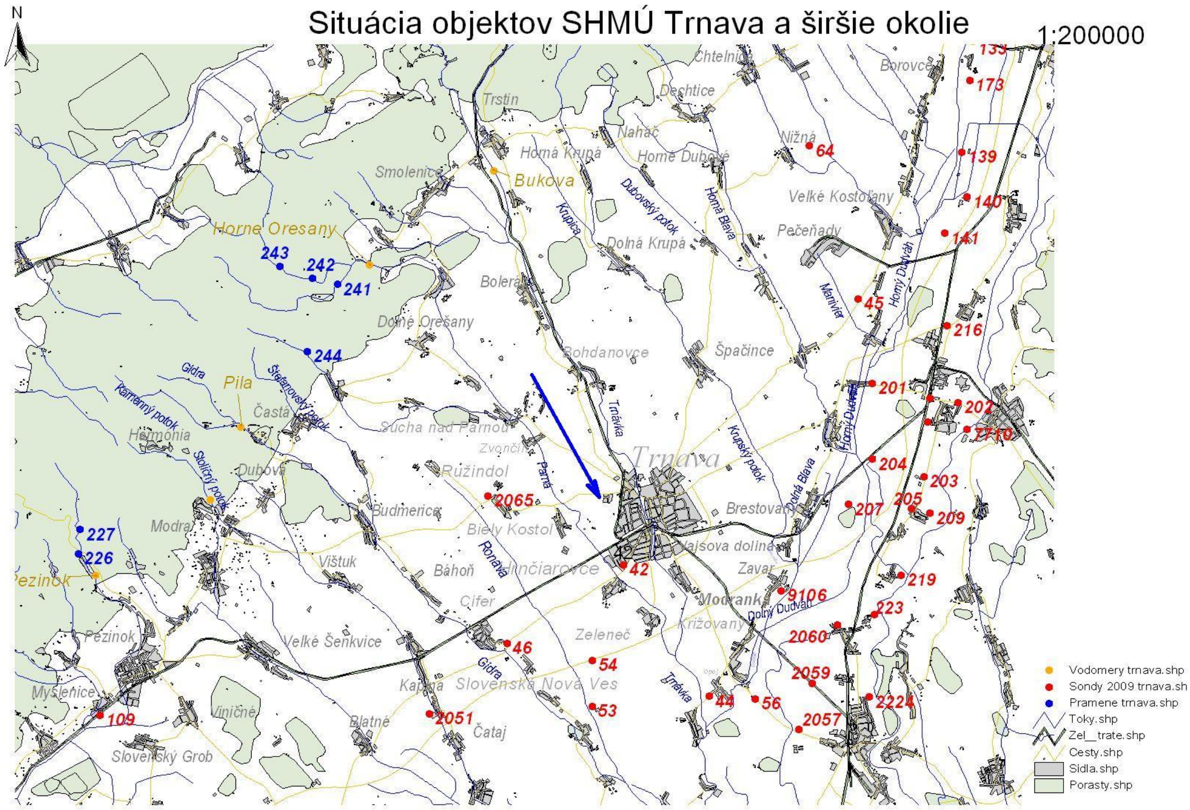

Fig. 1 Situation of SHMI groundwater level observation boreholes

Next input information very interesting for the solution was the development of meteorological situation in the solved area in the period from autumn 2009 until end of 2011. This development is apparent from the Tab.1 as well as in Fig. 2 where the monthly precipitation amount in % of the normal in May 2010 is illustrated. The "normal value" is the precipitation mean value in the period 1961-1990 (www.shmu.sk). There is a series of figures showing the great surplus of precipitation water in this period which culminated in the western part of Slovakia in May 2010 when three times more precipitation appeared. The consequence of the rich rainfall activity was an enormous increase of groundwater resources in the adjacent region (it was valid for the whole Slovakia) as well as groundwater level increase of more 1,5 m. This fact is evident even from the Fig.3 where the course of precipitation together with the groundwater level in two SHMI observation boreholes is illustrated.

Tab. 1. Overview of precipitation in Trnava meteorological station

Annual precipitation amount in mm	
Year	Total
2005	654,0
2006	552,6
2007	620,0
2008	559,0
2009	615,2
2010	905,0

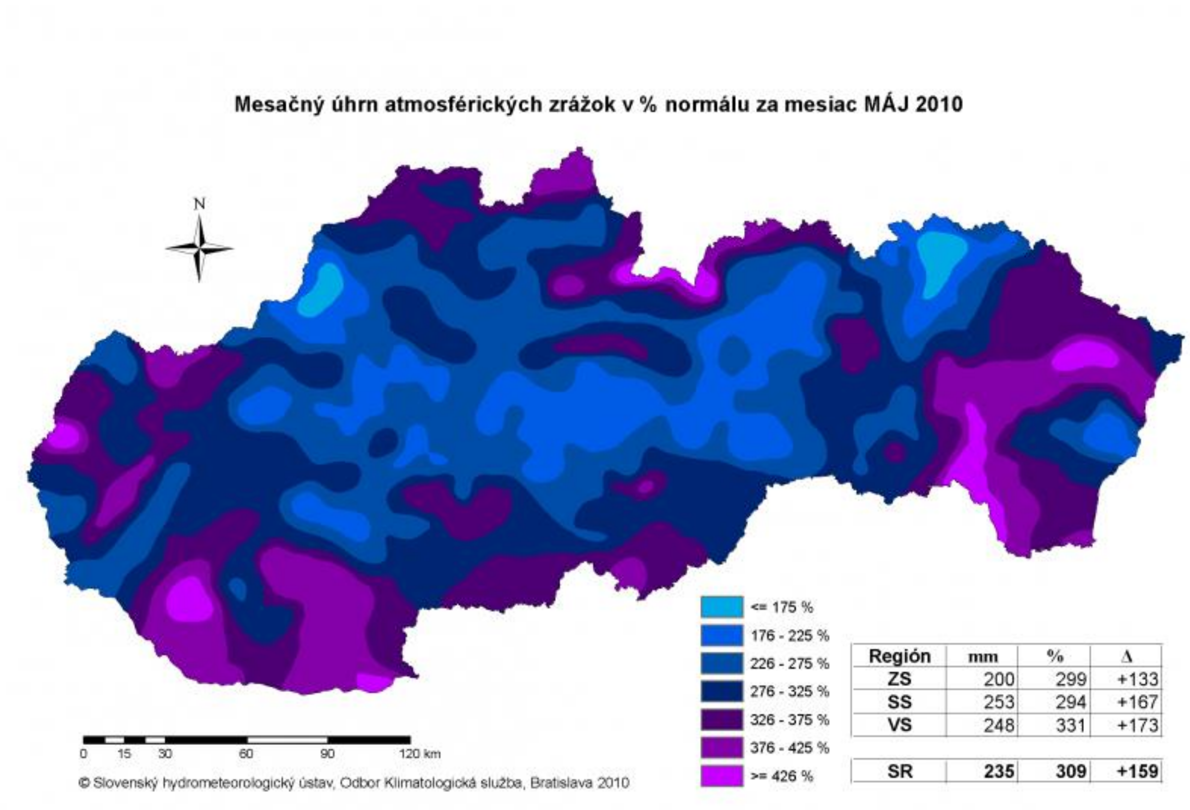

Fig. 2 Monthly precipitation amount in % normal in May 2010 in Slovakia (www.shmu.sk)

The development of the rainfall activity in the solved region is shown in Fig.4, where the course of groundwater level in given boreholes is shown together with the cumulative amount of atmospheric precipitation in 365 days. From the Fig. 4 is apparent that extreme precipitation values held on until the first half of the year 2011 and there was a great surplus of groundwater resources in that time.

The same situation happened in the spring time 2006 but the duration was shorter – the groundwater level decreased in two months already.

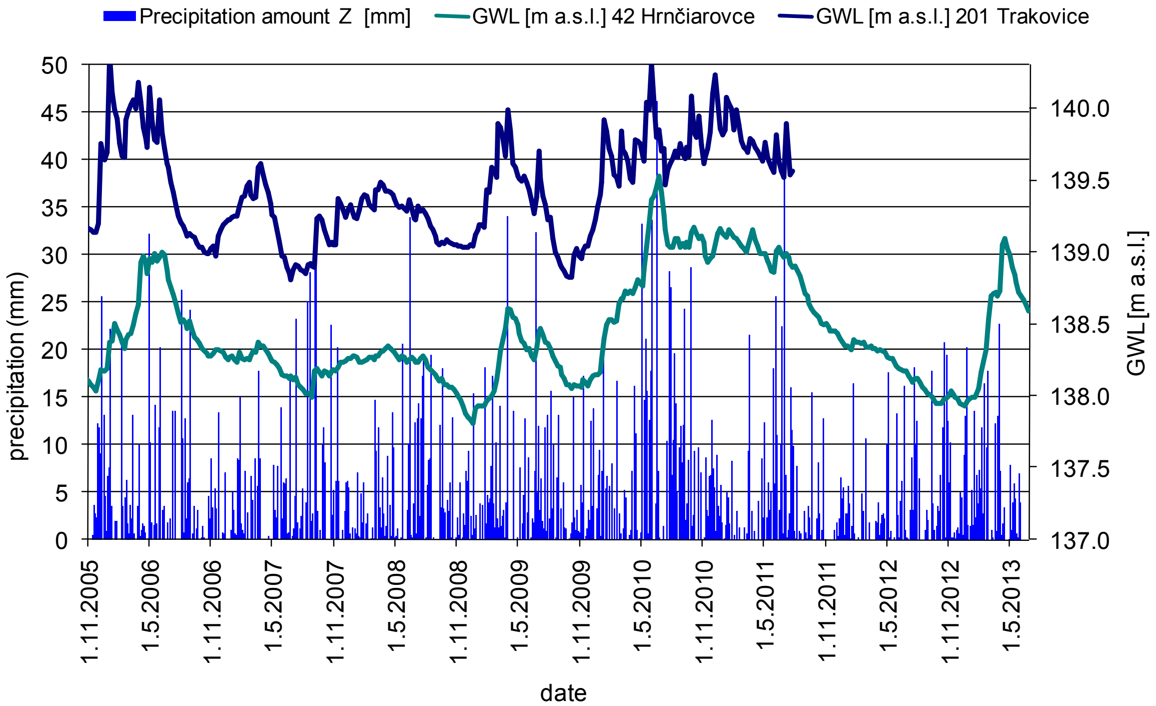

Fig. 3 The course of atmospheric precipitation and GWL in m a.s.l. in two SHMI observation boreholes in the period 2005-2013.

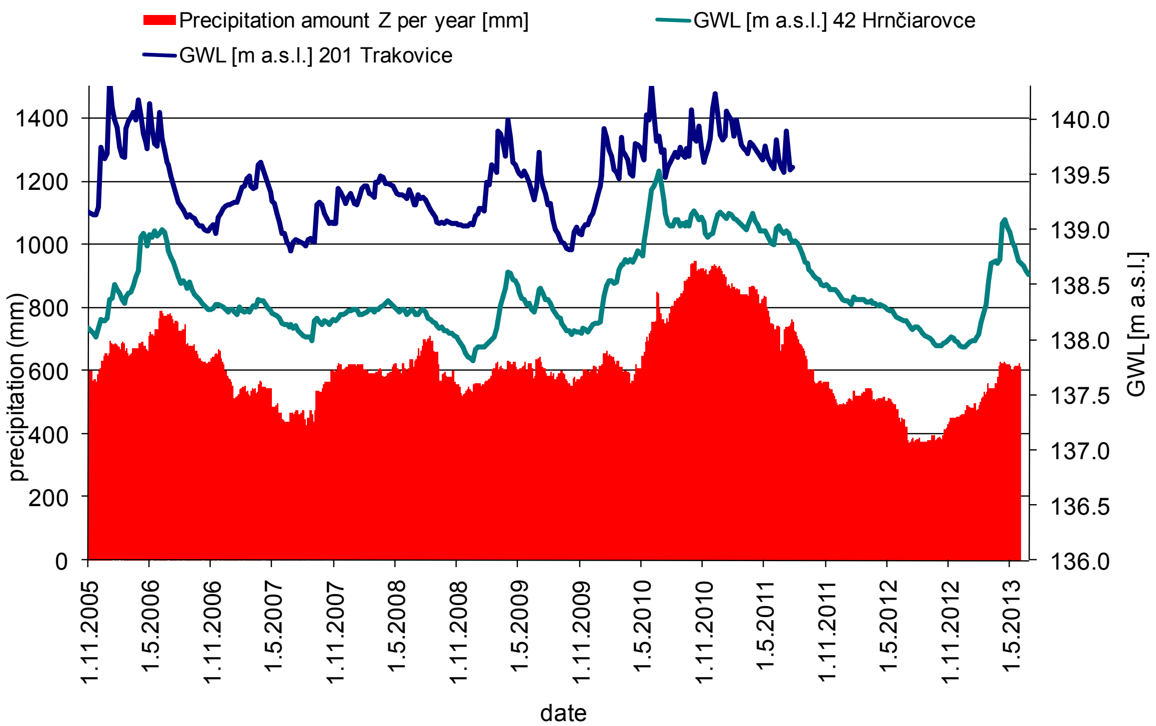

Fig. 4 Illustration of cumulative precipitation amount with the GWL development

2 MATERIAL AND METHODS

In May 2009 (May – November) performed the responsible Váh River Water Board the reconstruction of the mentioned Trnávka stream (approx. 1 km) in the urban area of Trnava town. The realised reconstruction was the continuation of smaller similar correction of the Trnávka stream in years 1968, 1971 and 1976. It came from the requirement for conservation of discharge capacity and for securing the run-off of hundred year water in the given reach of the stream. The project for the river training was elaborated by the Cabex, Ltd. projection company (Cabex, 2008).

The reconstruction itself was realised in geological structure consisting of neogenic and quaternary sediments. The technical solution of the stream reconstruction respected in maximum extent the stream tracking, stability is given by massive concrete blocks. The cross-section of the stream reconstruction is illustrated in Fig. 5.

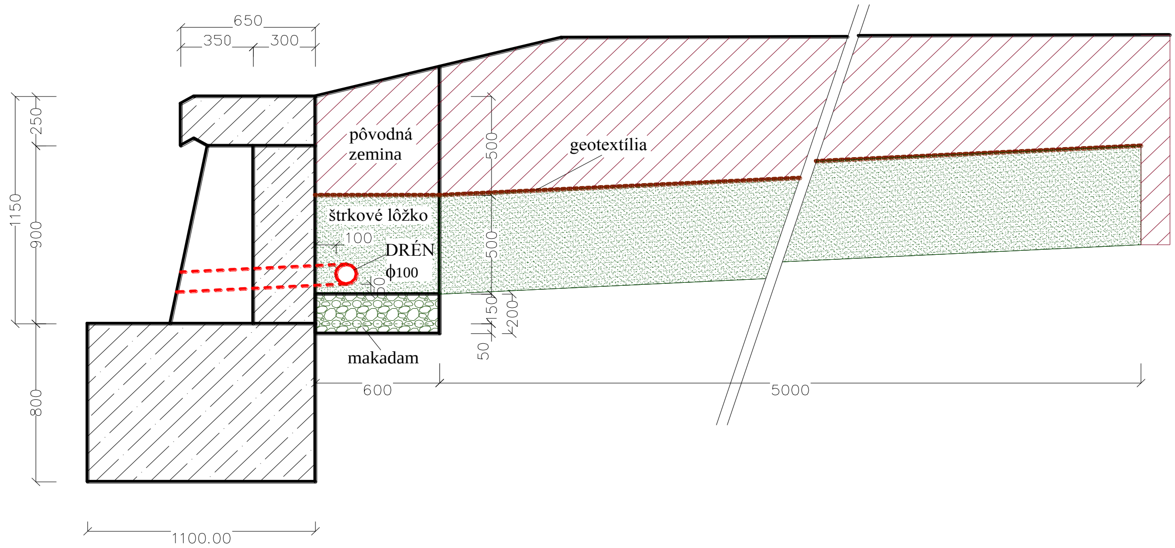

Fig. 5 Illustration of the proposed cross-section improvement of location of drainage tubes

The drainage tube (Fig. 5, red colour) was not technologically correct performed and it could not fulfil the drainage function in full extent. This insufficiency was eliminated and the Water Board had to prove the efficiency of this element. To realise the requirement of inhabitants of houses in adjacent area where the cellars were attacked by groundwater surplus a monitoring system along the reconstruction has been designed.

In March 2012 a monitoring system for GWL observation was built up for observing the impact of reconstruction of the Trnávka river bed of on GWL regime. The location of boreholes of the monitoring system is illustrated in Fig. 6. All of them were built up as twin boreholes on both sides of the river bed along the Trnávka River. To obtain a detailed analysis the last twin of boreholes (TV-4, TV 4´) was built up below the reconstruction of the river bed, i.e. in conditions of the natural streamflow. The monitoring period was set up to be at least one year coming from March 2012 until June 2013 to involve the whole one year period. This decision was correct because this period was distinguished by long lasting rainfalls, lot of snow in the spring time as well as a long lasting dry period.

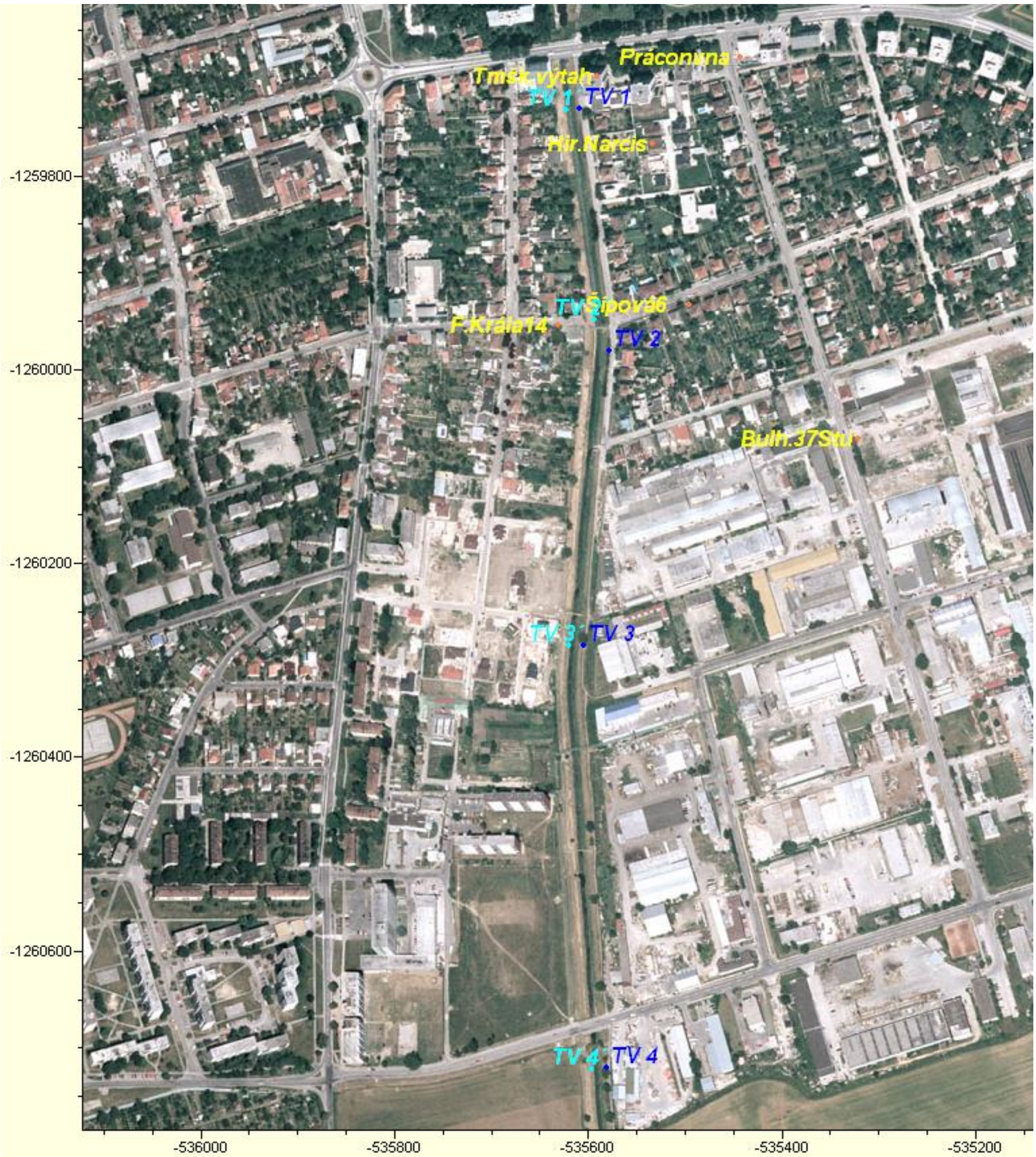

Fig. 6 Location of observing boreholes of the proposed monitoring system

3 RESULTS

Results of the GWL monitoring in the observation boreholes as well as in cellars of houses of attached inhabitants during the monitoring period are represented in the Fig.7. In the same figure there is the course of precipitation during this period as well as groundwater level regime in two chosen boreholes of the SHMI observation system.

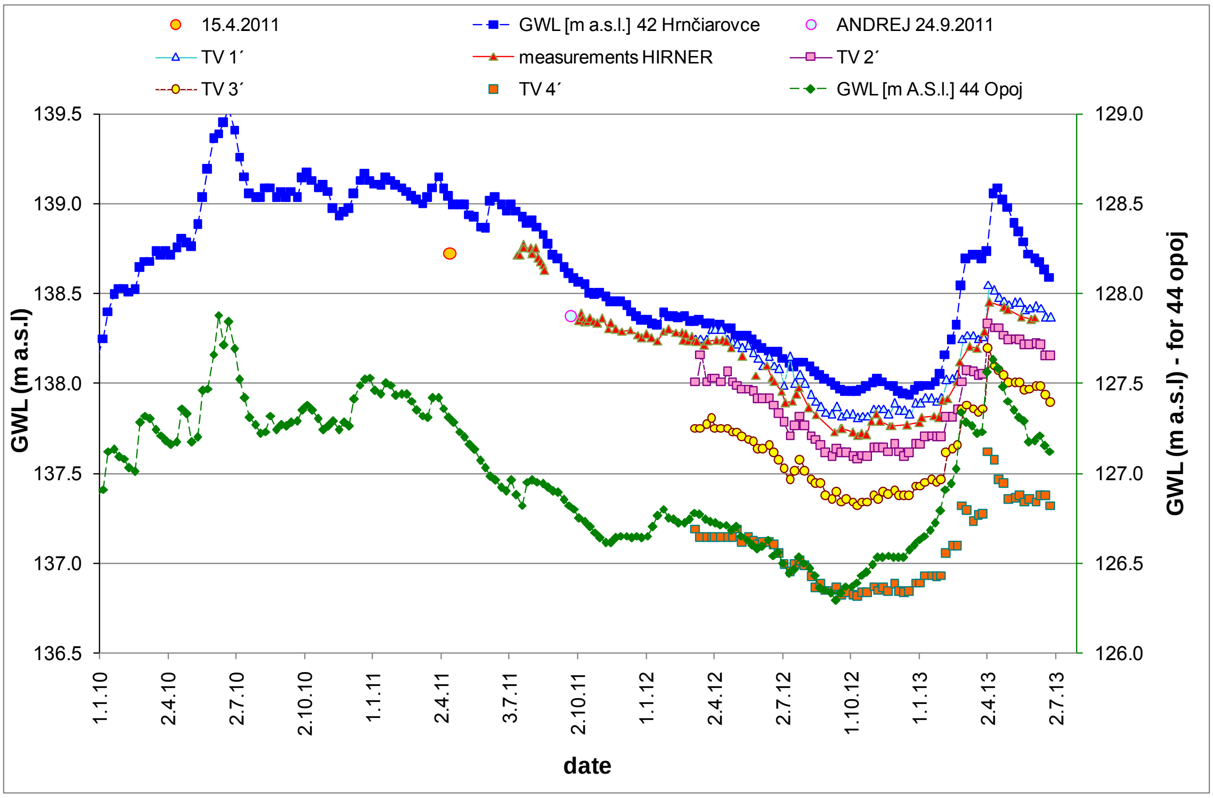

Fig. 7 Measured GWL course (m a.s.l.) during the monitoring period (October 2010 – June 2013)

From these measurement came out quite clearly that the groundwater flow is coming generally from the Small Carpathian region in north-east direction and the water level regime of a small Trnávka stream trained for many tens of years has no effect on the groundwater level regime in the adjacent region. It is apparent even from the next Fig.8 where the simulation model from measured data shows clearly the groundwater flow parallel to the stream of the Trnávka River.

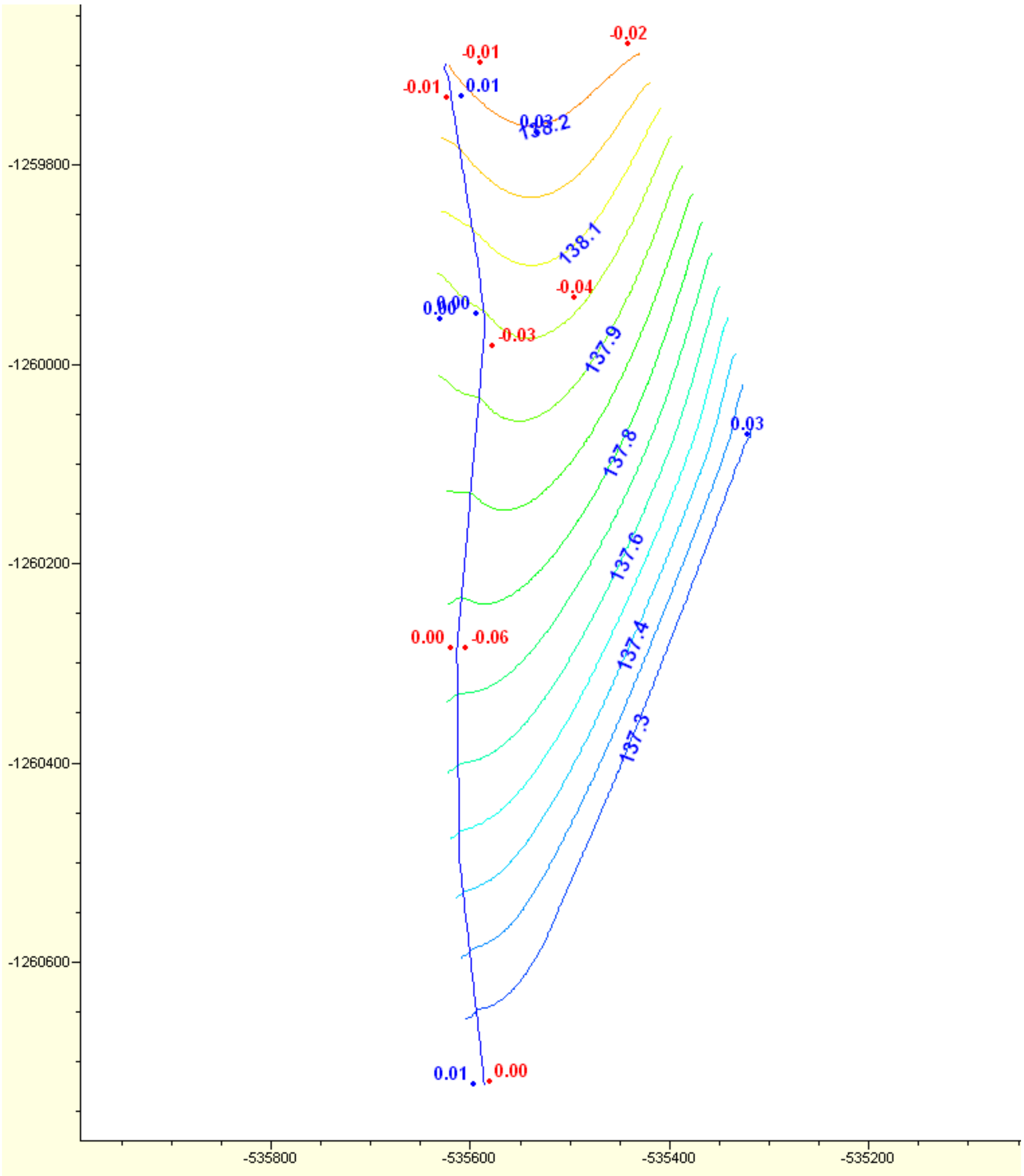

Fig. 8 Groundwater flow model in the solved region with differences of calculated and measured GWL value according to the monitoring measured results

4 CONCLUSION

The GWL measurements together with the analysis of precipitation load of the region below Small Carpathians have shown that the realized reconstruction (training) of the Trnávka River by the Váh River Board has no effect on GWL regime in the vicinity of the stream. Groundwater surplus was affected by enormous precipitation activity on the whole territory of Slovakia not just in the western part of the country. The reason for the moistened cellars are very deep cellars not respecting groundwater fluctuation in extreme hydrological situations.

Acknowledgements
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2D NUMERICAL SIMULATIONS OF 2012 FLOOD WAVE PASSAGE THROUGH HPP SYSTEM ON THE RIVER DRAVA

Damir Bekić[1], Ana Mioč[2], Igor Kerin[3]

Abstract

An extreme flood event on the River Drava in November 2012 was the highest in the last 60 years. It has caused over-spilling of levees and breaching on several levee sections, and consequently produced an extensive flooding of neighbouring settlements and farm fields and property damages in Slovenia and Croatia. This paper shows the results of the hydrological and hydraulic analysis of the flood wave propagation from Austria, through Slovenian hydro power plants to HPP system in Croatia. The hydrological analysis includes the flood flow transformation through the chain of HPPs. The detailed hydraulic analysis was performed on the HPP Varaždin for which a 2D hydrodynamic model has been developed. The model simulations showed the impact of the over-topping and breaching of levees on the extreme flows and flood wave propagation on the HPP Varaždin.

Keywords

flood wave, hydraulic analysis, hydrological analysis, 2d mathematical model, River Drava

1 INTRODUCTION

The flood wave on the River Drava from November 2012 compared to historical flood events was exceptional on several grounds. Considering the peak discharge of 3311 m^3/s and the flow rates on the HPP Varaždin this was the highest flood wave on the Croatian part of the River Drava in the last sixty years. The highest historical recorded flood wave was in 1966

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before the construction of HPPs in Croatia with the peak discharge of 2843 m^3/s on g.s. Varaždin. The second highest recorded flood event was in October 1998 after the construction of the HPP Varaždin with the peak discharge of 2221 m^3/s. The threefold flow increase on the HPP Varaždin within just 6 hours resulted in a very steep upward flood hydrograph. The November 2012 flood wave has caused over-spilling and breaching of levees on several locations, and consequently produced a great amount of damage on hydro-power and flood protection systems. In turn, the extensive flooding of the surrounding settlements occurred with damages to the local population, environment and industry. Furthermore, the flood wave entered the Croatian area during the night hours which was an aggravating circumstance for the monitoring of flood wave and hazards.

This paper shows the propagation of the 5/Nov/2012 flood wave through the Slovenian and Croatian hydro power systems and provides analysis of collateral events that caused its unexpected transformation in the area of the HPP Varaždin. The hydrological analysis is not given in any comprehensive way, as the emphasis is placed on the characteristic elements of wave transformation and on the distinctiveness and complexity of this exceptional flood event.

Fig. 1 Chain of HPPs on the River Drava in Austria, Slovenia and Croatia

The first part of the paper shows the wave propagation from the Austrian HPP Labot through the Slovenian hydro power systems and the Croatian HPPs (**Fig. 1**). The flood wave entered the Slovenia area (HPP Labot) on the 5^{th} Nov 2012 around 16:00 hrs and in the next 11 hours caused the flow increase by 2060 m^3/s, reaching the peak flow of 2592 m^3/s at 15:00 hrs. This sudden increase of discharge is one of the main reasons for flood damages in the downstream sections. The flood wave peak on the HPP Formin of 2840 m^3/s was recorded on the 5^{th} Nov at 23:00 hrs. The peak flow of 3311 m^3/s on the HPP Varaždin was 471 m^3/s higher than the peak on the HPP Formin, which is also one of the particularities of this flood event.

The second part of the paper gives an analysis of flood wave propagation through 3 Croatian hydro power plants, and focuses on the area of the HPP Varaždin. A detailed hydrodynamic model of the area between HPP Formin and HPP Varaždin was not available. For the hydraulic analysis of 5/11/2012 flood wave a 2d hydrodynamic model of this area was developed. The model comprises the River Drava old channel from HPP Formin dam till HPP Varaždin dam, HPP Formin outlet canal from HPP Formin powerhouse to the restitution into the River Drava’s old channel and two River Drava tributaries, Dravinja and Pesnica. The flood wave caused two levee breaches of the HPP Formin outlet canal so one of the goals of the hydraulic analysis was to determine the impact of the levee breaches on the flood wave transformation. On the hydrodynamic model several simulations of flood wave were made, which enabled new insights into the complexity of flood wave propagation on the area.

The input data for the analysis were used from several sources in Slovenia and Croatia [1], [2], [3]. The discharge data was used for HPP Labot, HPP Vuhred, HPP Ožbalt, HPP Mariborski otok and HPP Zlatoličje. The discharge and water levels data for 2010, 2011 and 2012 flood events were used for the analysis of flood wave transformation in the area between HPP Formin and HPP Varaždin, and comprised: dam and powerhouse of HPP Formin, dam and powerhouse of HPP Varaždin, gauging stations Borl and Ormož on the River Drava, g.s. Videm on the River Dravinja and g.s. Zamušani on the River Pesnica.

2 PROPAGATION OF 5/NOV/2012 FLOOD WAVE THROUGH SLOVENIA AND CROATIA

The River Drava rises in the South Tyrol in Italy, and flows eastwards through Austria, Slovenia, Hungary and Croatia, where it meets the Danube near Aljmaš. The River Drava is 719 km long and is also the fourth largest and the fourth longest Danube tributary. Along the upper reaches, in Austria, Slovenia and Croatia, 21 large hydro power plants were constructed. Ten of them are in Austria, eight in Slovenia and three in Croatia. The last Austrian hydropower plant on the River Drava is HPP Labot.

About 2 km downstream from HPP Labot, the River Drava forms the state border between Austria and Slovenia. The first in chain of Slovenian HPP system is HPP Dravograd followed by HPP Vuzelnica, HPP Vuhred, HPP Ožbalt, HPP Fala, HPP Mariborski otok, HPP Zlatoličje and HPP Formin. The chain of HPP system in Slovenia has small reservoir volumes up to HPP Formin (**Tab. 1**) so there is little or no storage capacity for flood flows in the upper Slovenian reservoirs.

The River Drava enters Croatia near Dubrava Križovljanska settlement. There are three hydro power plants in Croatia. The first in chain is HPP Varaždin followed by HPP Čakovec and HPP Dubrava. The reservoir of HPP Varaždin of 8.0 hm^3 is significantly smaller compared to the ones of the HPP Čakovec of 51.0 hm^3 and 93.5 hm^3 of the HPP Dubrava (**Tab. 1**).

Tab. 1 Characteristics of the hydro power plants in Slovenia and Croatia [**1**], [2]

HPP	Country	Type	Reservoir [hm^3]	Installed flow [m^3/s]	Total spilling capacity [m^3/s]
HPP Dravograd	Slovenia	impoundment	5.6	420	5400
HPP Vuzelnica	Slovenia	impoundment	7.1	550	5600
HPP Vuhred	Slovenia	impoundment	10.3	550	5800
HPP Ožbalt	Slovenia	impoundment	10.5	550	5800
HPP Fala	Slovenia	impoundment	4.2	550	4800
HPP Mariborski Otok	Slovenia	impoundment	13.1	550	5600
HPP Zlatoličje	Slovenia	diversion	4.5	530	4800
HPP Formin	Slovenia	diversion	17.1	500	4800
HPP Varaždin	Croatia	diversion	8.0	500	4800
HPP Čakovec	Croatia	diversion	51.0	500	4800
HPP Dubrava	Croatia	diversion	93.5	500	4800

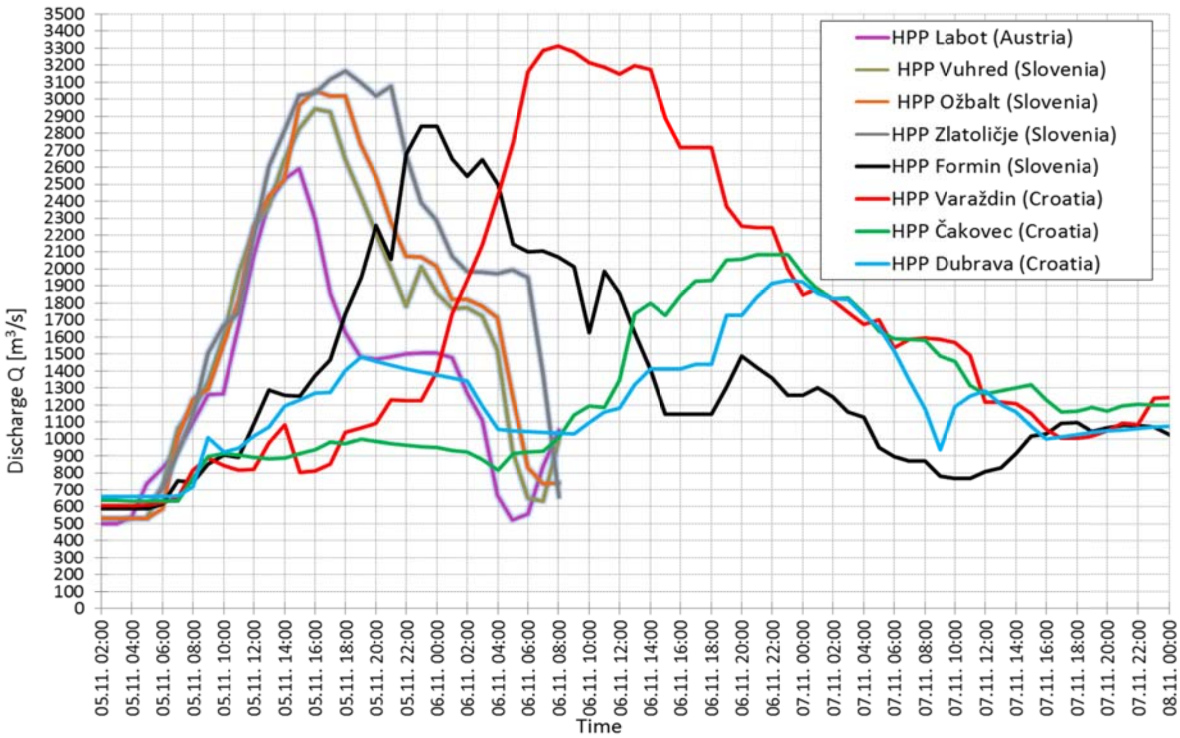


Fig. 2 Flood event 5/Nov/2012 - Flow hydrographs on HPPs in Austria, Slovenia and Croatia

Fig. 2 shows recorded hydrographs on HPP Labot in Austria, HPPs Vuhred, Ožbalt, Zaltoličje and Formin in Slovenia and also on HPPs Varaždin, Čakovec and Dubrava in Croatia during the 5/Nov/2012 flood wave (sources [1], [2], [3]). Considering the fact that the HPP system on Zlatoličje, Formin, Varaždin, Čakovec and Dubrava is a diversion type of the hydro power plant, the hydrograph of these systems represents a total discharge of a dam and a powerhouse.

Tab. 2 shows the timing and the peak flow on the chain of HPPs on the River Drava. The flood wave entered Slovenia on the 5th Nov at 16:00 hrs and in the next 11 hours caused the flow increase by 2060 m^3/s. The peak flow of 2592 m^3/s on the HPP Labot was reached on the 5th Nov at 15:00 hrs. On HPP Fomin the flood wave peak of 2840 m^3/s was reached on the 5th Nov at 23:00 hrs. The highest peak flow of 3311 m^3/s in Croatia was recorded on HPP Varaždin on the 5th Nov at 08:00 hrs.

Tab. 2 Discharges and timing of the peak flows [1], [2]

HPP	Country	Discharge at peak Q [m^3/s]	Time at peak	Peak discharge diff ΔQ [m^3/s]	Time between two peaks Δt [hrs]
HPP Labot	Austria	2592	5/11/2012 15:00		
HPP Vuhred	Slovenia	2945	5/11/2012 16:00	+353	1
HPP Ožbalt	Slovenia	3040	5/11/2012 17:00	+95	1
HPP Zlatoličje	Slovenia	3170	5/11/2012 18:00	+130	1
HPP Formin	Slovenia	2840	5/11/2012 23:00	-330	5
HPP Varaždin	Croatia	3311	6/11/2012 08:00	+471	9
HPP Čakovec	Croatia	2085	6/11/2012 21:00	-1226	13
HPP Dubrava	Croatia	1930	6/11/2012 23:00	-155	2

The flow hydrographs show sudden, almost simultaneous flow increase on HPPs Labot, Vuhred, Ožbalt and Zlatoličje. The peak flows on these HPPs on the 5th Nov were 2592, 2945, 3040 and 3170 m^3/s at 15:00, 16:00, 16:00 and 18:00 hrs. While passing through the reservoir of HPP Formin the first transformation of flood wave occurred. According to the hydrographs on **Fig. 2** it can be seen that:

- The peak flow of 2840 m^3/s on HPP Formin (black line) occurred on the 5th Nov at 23:00 hrs, and it was 330 m^3/s lower than the peak flow on HPP Zlatoličje.
- The peak flow of 3311 m^3/s on HPP Varaždin (red line) occurred 9 hours later, on the 6th Nov at 08:00 hrs. The peak flow on HPP Varaždin was 471 m^3/s higher than the peak flow on HPP Formin.
- The flow rates on HPP Čakovec were significantly reduced due to the combined water losses on levee over-spilling and breaching and on infiltration. The peak flow of 2085 m^3/s on HPP Čakovec (green line) occurred 13 hours later than on HPP Varaždin, on the 6th Nov at 21:00 hrs. It was 1226 m^3/s lower than the peak flow on HPP Varaždin.
- The peak flow of 1930 m^3/s on HPP Dubrava occurred 2 hours later, on the 6th Nov at 23:00 hrs, and it was 155 m^3/s lower than the peak flow on HPP Čakovec.

The hydrographs on the 5/Nov/2012 and the past hydrographs show that the flood wave transformation in the area between HPP Formin and HPP Varaždin was unexpected, as were the recorded flow rates on HPP Varaždin (red line **Fig. 2**). The sum of the peak discharges of the two tributaries (Dravinja and Pesnica) of 200 m^3/s, and the extensive levee over-spilling in the area, shows that that the peak flow on HPP Varaždin of 3311 m^3/s was unpredictable. The discharge was 471 m^3/s higher than the peak flow of 2840 m^3/s on HPP Formin. The unexpected increase of the peak flow on HPP Varaždin is associated with the levee breaches of HPP Formin outlet canal, and that was the main task of the hydrodynamic model simulations.

3 FLOOOD WAVE ON 5/NOV/2012 IN THE AREA BETWEEN HPP FORMIN AND HPP VARAŽDIN

3.1 Description of the area

The area between HPP Formin and HPP Varaždin has complex hydrographic characteristics (**Fig. 3**). The area is bordered with the HPP Formin canal in the north and with the high terrain in the south. The inlet canal connects the reservoir and the powerhouse, whereas the outlet canal connects the powerhouse with the River Drava's old channel. During the low and the medium inflows the majority of the flow is diverted through the powerhouse and only a minor part (biological minimum) is discharged through the River Drava's old channel. An opposite split occurs during high inflows when the majority of flows are discharged over the dam to the River Drava's old channel. The installed flow capacity of HPP Formin powerhouse is 500 m^3/s and the spilling capacity of HPP Formin dam is 4300 m^3/s. Two main tributaries of the River Drava are the Dravinja stream (left tributary) and the Pesnica stream (right tributary). In the area there are four gauging stations: g.s. Borl and g.s. Ormož on the River Drava, g.s. Videm on the Dravinja stream and g.s. Zamušani on the Pesnica stream. An average flow of the River Drava near Varaždin is 330 m^3/s, and the average maximum annual flow is 1286 m^3/s [4].

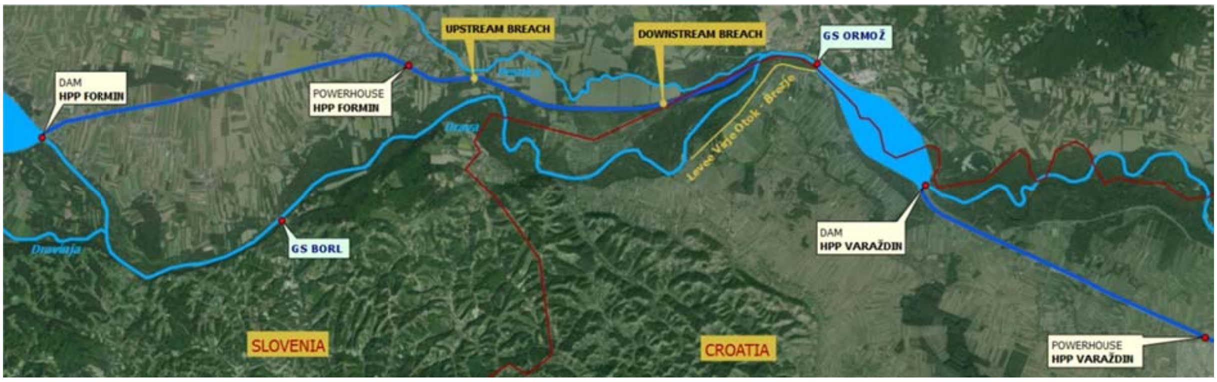


Fig. 3 The River Drava section between HPP Formin and HPP Varaždin

The Drava River valley between HPP Formin and HPP Varaždin has a complex morphology. From the mouth of the Dravinja stream, the river valley width is reduced from 2300 m to 800 m near Zavrč settlement in the downstream direction. The valley then widens to 1700 m near Veliki Lovrečan settlement, and then again gently reduces towards the beginning of the Virje Otok-Brezje levee to 1100 m. From the beginning of the Virje Otok-Brezje levee the valley width is reduced first to 580 m near the mouth of HPP Formin outlet canal, than to 290 m near the Virje Otok settlement and finally to 210 m at the road bridge in Ormož.

3.2 Hydrographs of flood waves

For a general overview of the unusual transformation of the 5/Nov/2012 flood wave, a comparison (**Fig. 4**) of input (solid line) and output (dashed line) hydrographs in the area between HPP Formin and HPP Varaždin is given for the flood waves in 2010, 2011 and 2012 (according to [1], [2]).

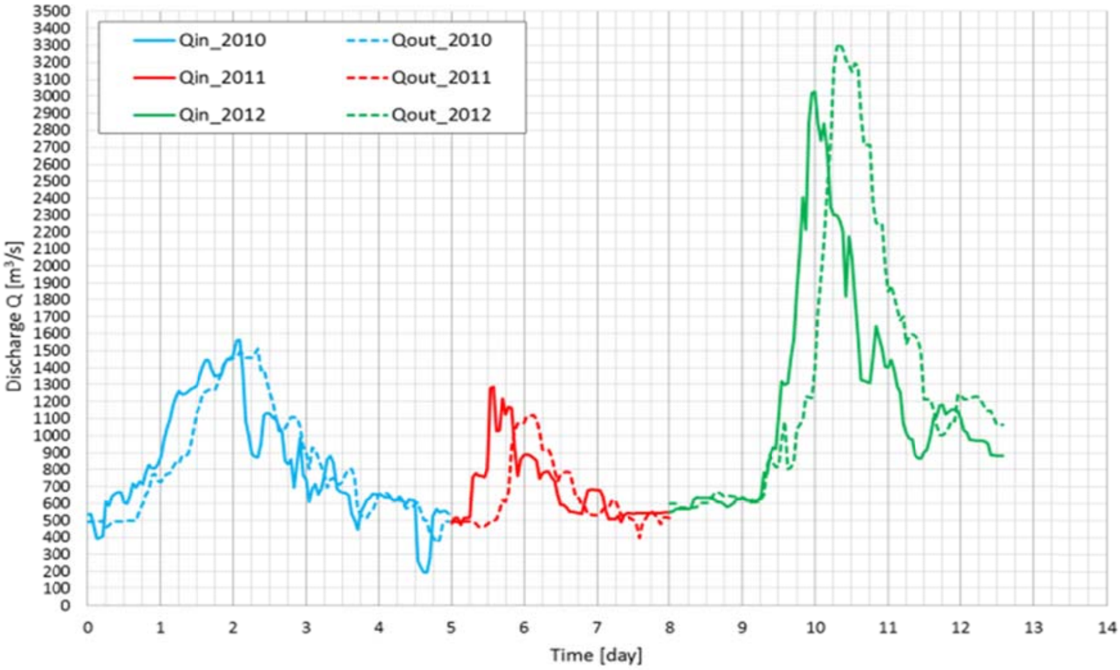


Fig. 4 Comparison of flow hydrographs for flood events in 2010, 2011 and 2012

The discharge Q_{in} represents the sum of flows on the dam and the powerhouse of HPP Formin together with the Dravinja and Pesnica streams. The discharge Q_{out} represents the sum of flows on the dam and the powerhouse of HPP Varaždin. The flood wave hydrographs in 2010 and 2011 are actually typical in this area and show that the highest output flows from the area (Q_{out}) are generally lower than the highest input flows (Q_{in}). However, the highest output flow in November 2012 was 300 m^3/s (10%) higher than the highest input flow.

The rate of the 5/Nov/2012 flood wave onset can be seen when compared to the theoretical flood hydrographs (**Fig. 5**). Based on the statistical analysis of flood waves in 2005, 2008, 2009, 2010 and 2011, the theoretical flood hydrographs for g.s. Borl were produced [5]. It can be seen that the peak discharge in 2012 has between 100 and 1000 years return period. The upward branch of the 2012 flood hydrograph corresponds to the upward branch of the 1000 years theoretical flood wave (**Fig. 5**).

The flood inflows of the River Drava and the Dravinja and Pesnica streams were estimated (**Tab. 3** and **Fig. 6**) in several previous and one recent study ([5], [6], [7]). The estimated discharges from 1974 and 2013 show relatively small differences of the total input flows, while the flows from VGI 1997 study are slightly higher. **Tab. 3** shows that the peak input flow of the 5/Nov/2012 flood wave (Q_{in}=3100 m^3/s) has 200 years return period.

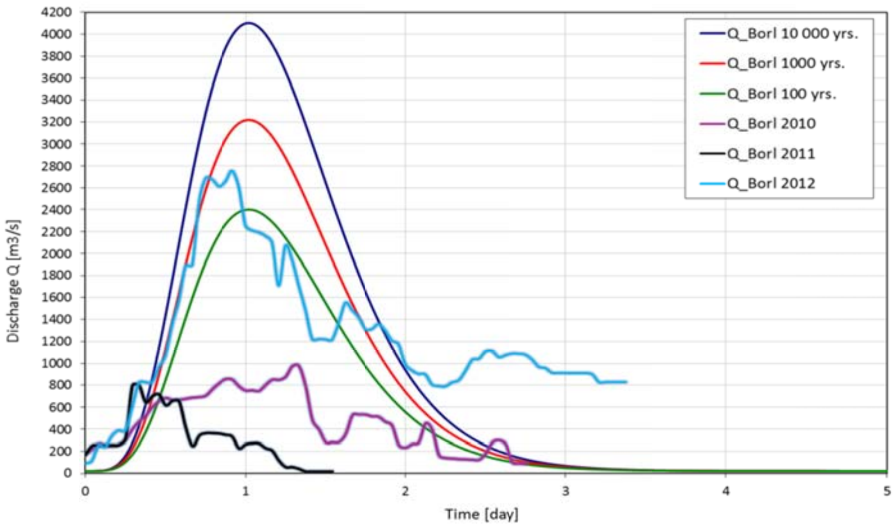


Fig. 5 Comparison of theoretical and measured hydrographs of the River Drava on g.s. Borl

Tab. 3 Comparison of estimated total flood inflows

Discharge Q [m^3/s]			**Return period [years]**		
No.	**Study**	**Including**	**100**	**1000**	**10.000**
1	Franković 1974	before construction of HEF and HEV	2800	3700	4600
2	VGI Ljubljana 1997	$Q_{damHEF} + Q_{phHEF} + Q_{Dravinja}$	2897	4208	-
3	IEE 2002	$Q_{damHEF} + Q_{phHEF} + Q_{Dravinja} + Q_{Pesnica}$	2600	-	-
4	IEE 2013(*)	$Q_{damHEF} + Q_{phHEF}^{(*)} + Q_{Dravinja}$	2849	3671	4553
5	IEE 2013	$Q_{Pesnica}$	166	223	279
6	IEE 2013(*)	$Q_{damHEF} + Q_{phHEF}^{(*)} + Q_{Dravinja} + Q_{Pesnica}$	3015	3894	4832

(*) discharge through HPP Formin powerhouse was assumed at 450 m^3/s

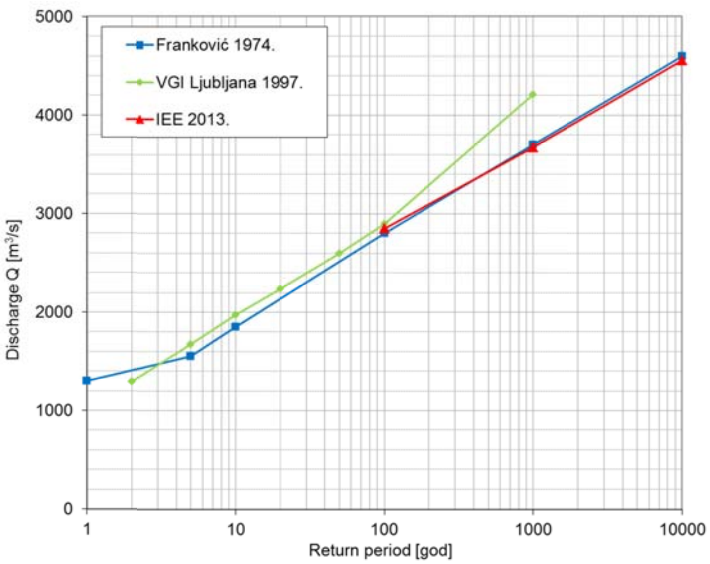


Fig. 6 Comparison of estimated total flood inflows

3.3 Levee breached on the HPP Formin outlet canal

During the 5/Nov/2012 flood wave the two levee breaches of the HPP Formin outlet canal occurred. The upstream breach was 150 m wide and is located 1.3 km downstream from the HPP Formin powerhouse. **Fig. 7** gives a comparison of the outlet canal geometry before (blue line) and after the breach (red line), and shows the extreme power of flow at the breach. The sudden flow of water washed away the right levee of the outlet canal and has left up to 12 m of sand and silt deposits in the outlet canal. The sudden flow of water also demolished the left bank of the canal, and eroded up to 50 m wide and 8 m tall part of the adjacent land.

(a) Aerial photograph of the levee breach [1] (b) Canal before and after the breach [1]

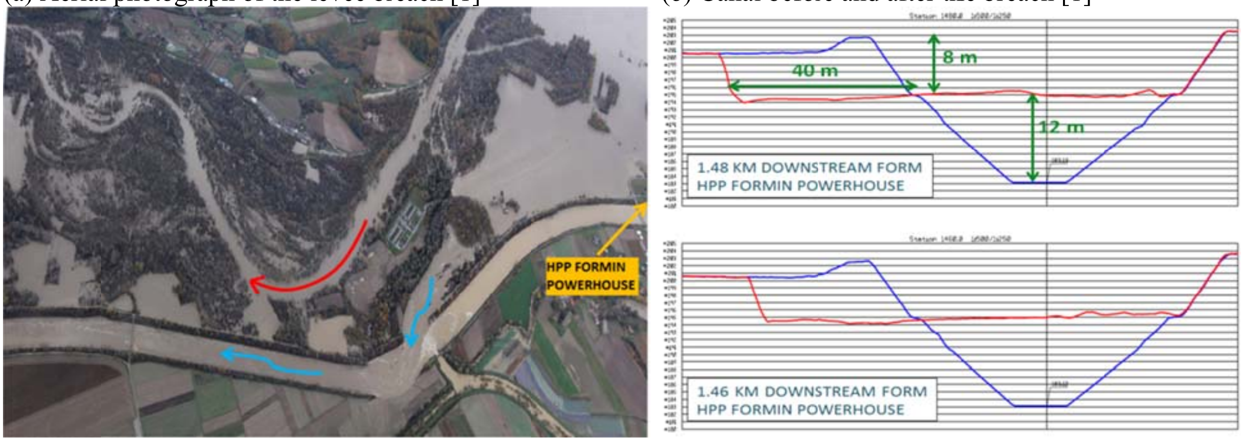


Fig. 7 Upstream levee breach of HPP Formin outlet canal

The downstream breach was 200 m wide and is located 6.3 km downstream from the HPP Formin powerhouse (**Fig. 8**). According to the DEM report [1], the breach caused a 6 m tall levee damage leaving 300,000 m^3 of soil deposits in the outlet canal. The River Drava actually created a new riverbed through which diversion of flows to the outlet canal was evident even after the flood wave passage.

Fig. 8 Downstream levee breach of HPP Formin outlet canal (photo taken 9/Nov/2012) [1]

3.4 Transformation of 5/Nov/2012 flood wave between HPP Formin and HPP Varaždin

Hourly discharge recordings are shown for the dam and powerhouse of the HPP Formin and HPP Varaždin (**Fig. 9**) and for the Dravinja and Pesnica streams (**Fig. 10**), the data were provided from [1], [2]. The discharge Q_{tot} represents the total discharge for dam and powerhouse $Q_{tot}=Q_{dam}+Q_{ph}$. During the flood wave passage the two levee breaches of the HPP Formin outlet canal occurred as well as over-spilling and breaching of the Virje Otok-Brezje levee. The peak flow on the HPP Varaždin $Q_{tot.max}$=3311 m^3/s (6th Nov at 08:00 hrs) was significantly higher than the peak flow on the HPP Formin $Q_{tot.max}$=2840 m^3/s (5th Nov at 23:00 hrs). The HPP Varaždin peak flow was 471 m^3/s higher than the peak on the HPP Formin. The peak flows of the Dravinja 109 m^3/s and the Pesnica of 85 m^3/s [3] were insufficient to provide the recorded flow increase of 471 m^3/s on the HPP Varaždin (**Fig. 10**).

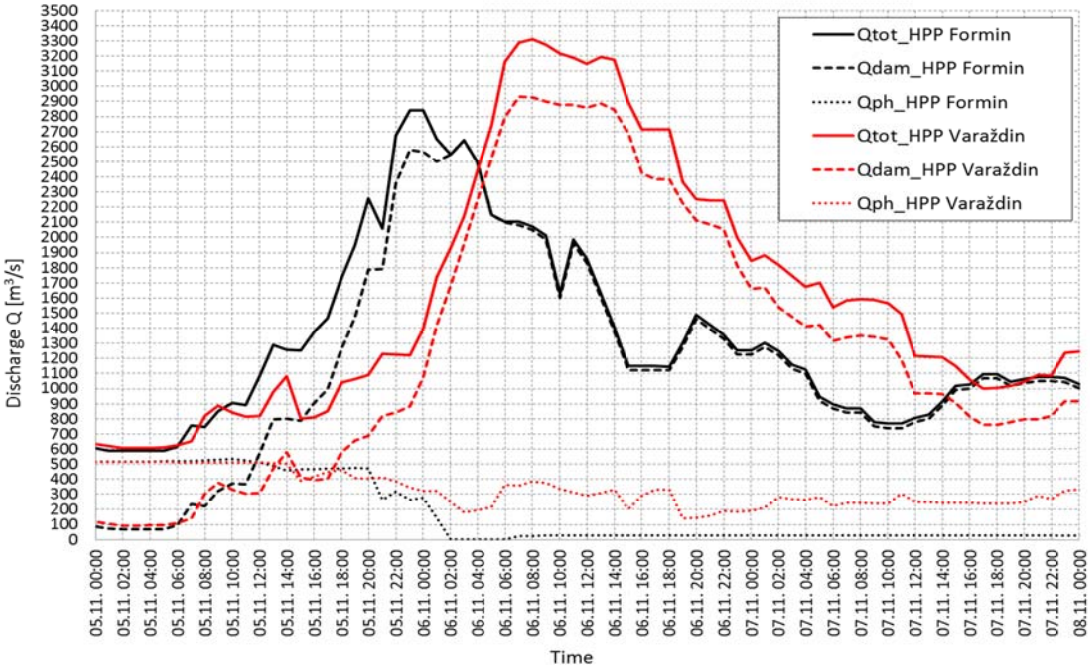


Fig. 9 Flow hydrographs on HPP Formin and HPP Varaždin on 5th November 2012

The discharge through the HPP Formin powerhouse (Q_{ph}) shows that on the 6th Nov between 00:00 and 02:00 hrs the powerhouse failure occurred. The failure is considered to be connected to the upstream levee breach of the HPP Formin outlet canal. The breach was located 1.3 km downstream from the powerhouse and caused backwater rising in the outlet canal and eventually the flooding of the powerhouse and the switchyard (**Fig. 7**).

On the 6th Nov, in the period from 02:00 to 06:00 hrs, an unexpected and an unnatural water level rising was recorded [2] on g.s. Ormož (**Fig. 11**). A 1.4 m water level on g.s. Ormož occurred at the time of the powerhouse failure, so the upstream levee breach of the outlet canal is most likely the main cause of the sudden water level rising.

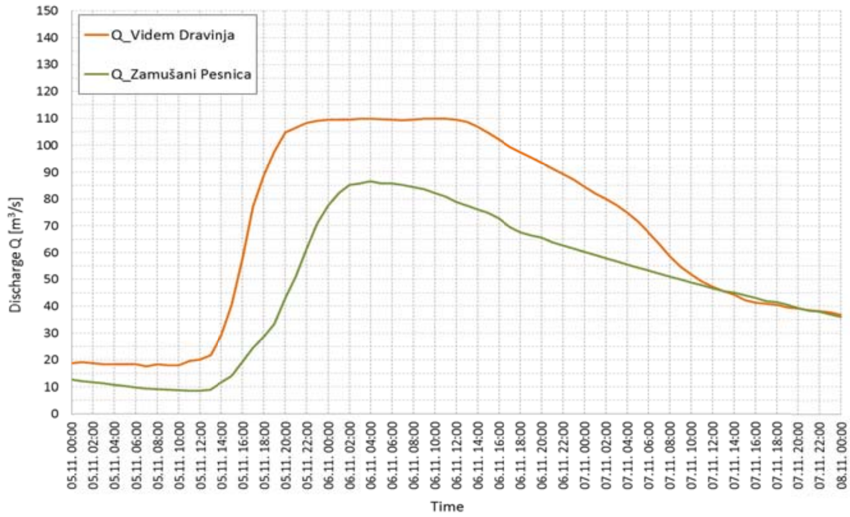


Fig. 10 Flow hydrographs of the Dravinja and the Pesnica stream during 5/Nov/2012

Fig. 11 Water levels on HPP Varaždin on 5th November 2012

4 HYDRAULIC ANALYSIS OF 5/NOV/2012 FLOOD WAVE

During the 5/Nov/2012 flood wave the levee of HPP Formin outlet canal was breached on two locations. The recorded hydrographs show that the two breaches of outlet canal caused the over-spilling of great amount of water from the River Drava's old channel to the outlet canal, which resulted in flow acceleration, shorter flood wave duration together with sudden increase of water levels and discharges on the HPP Varaždin.

During the passage of the flood wave an over-spilling and breaching of the Virje Otok-Brezje levee also occurred. The exact time of the outlet canal breaches is not known. But it is known that the over-spilling of Virje Otok-Brezje levee began on the 6th Nov at 04:30 hrs and that the breach happened on the 6th Nov after 11:00 hrs.

The analysis of the 5/Nov/2012 hydrographs on the HPP Formin, HPP Varaždin and HPP Čakovec showed that the flood wave on HPP Varaždin was unnatural. The flood wave transformation between HPP Formin and HPP Varaždin is probably the consequence of the levee breaching of the outlet canal. As there was no data on the exact timing of the outlet canal levee breaching it was necessary to: (a) consider all possible causes of flow increase on HPP Varaždin, (b) verify credibility of discharge recordings on HPP Varaždin and (c) estimate over-spilling on the levee Virje Otok-Brezje. Due to the complexity of the river system and due to high flood damages, a detailed hydraulic analysis was performed by using a 2d hydrodynamic numerical model.

4.1 Development of the hydrodynamic model

For the purpose of the analysis of the 5/Nov/2012 flood wave transformation between HPP Formin and HPP Varaždin a 2d hydrodynamic model was developed by using the Delft3D software package (Deltares Systems). The hydrodynamic model was developed on the basis of the recent terrain and bathymetry surveys. The model comprised the River Drava's old channel between the HPP Formin dam and the HPP Varaždin reservoir, and also the HPP Formin outlet canal from the powerhouse to the confluence with the River Drava's old channel (**Fig. 12**). The model also included the Dravinja and Pesnica streams, as well as all line objects in the modelled area, such as roads and levees. The total length of the modelled area was 27+675 m.

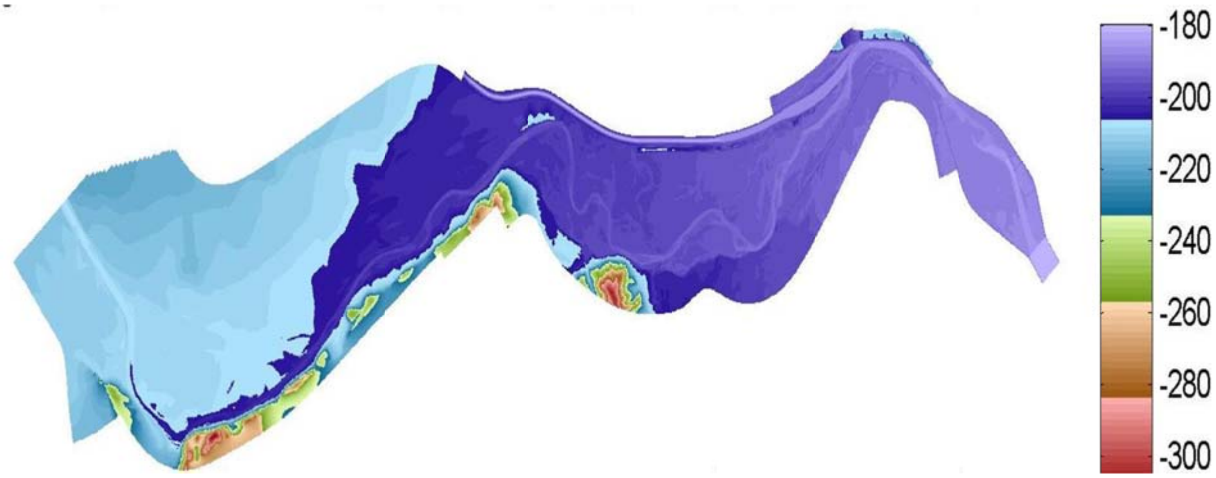


Fig. 12 Bathymetry of modelled area

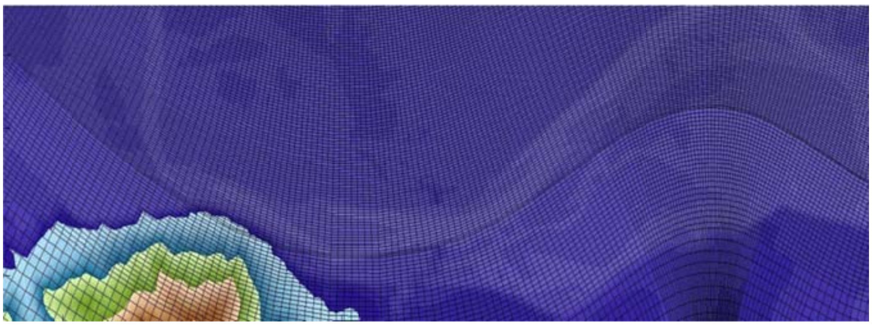


Fig. 13 Othogonal curvilinear grid of the model with bathymetry

The upstream open model boundaries are the dam and the powerhouse of the HPP Formin, which are both defined as a time-varying discharge. The downstream open model boundary is the upper water level at the HPP Varaždin dam, which was defined as a time-varying water level. The two tributaries, the Dravinja and Pesnica streams, are modelled as sources.

On the River Drava's old channel, from the HPP Varaždin dam to the HPP Formin dam, 41 no. control cross sections (P0-01 to P0-41) were set, together with 17 no. cross sections along the HPP Formin outlet canal from the restitution at the River Drava's old channel till the HPP Formin powerhouse (P1A-17 to P1A-34**)**. The model control point was g.s. Ormož for which water level measurements were available.

The initial analysis for the discharges above 100 m^3/s showed that the measured discharges on g.s. Borl on the River Drava are higher than the combined discharges of the HPP Formin dam and the Dravinja stream. It was concluded that the stage-discharge curve for g.s. Borl should be improved especially for higher flow rates. As the stage-discharge curve needs more adjustments, the discharges at g.s. Borl were excluded as a model control point.

The model calibration was made for g.s. Ormož by comparing the recorded and the computed water levels for the flood event from the 18th Jun 2011 at 01:00 hrs until the 22nd Jun 2011 at 00:00 hrs. The differences between measurements and calculations for g.s. Ormož were between ±5 cm with the largest deviation of 9 cm (**Fig. 14a**).

The model parameters were verified for the flood event from the 17th Sept 2010 at 01:00 hrs untill the 21st Sep 2010 at 00:00 hrs. The differences between measured and calculated water levels on g.s. Ormož were between ±5 cm with the largest deviation of 7 cm (**Fig. 14b**).

(a) Model calibration results for 6/2011 (b) Model verification results for 9/2010

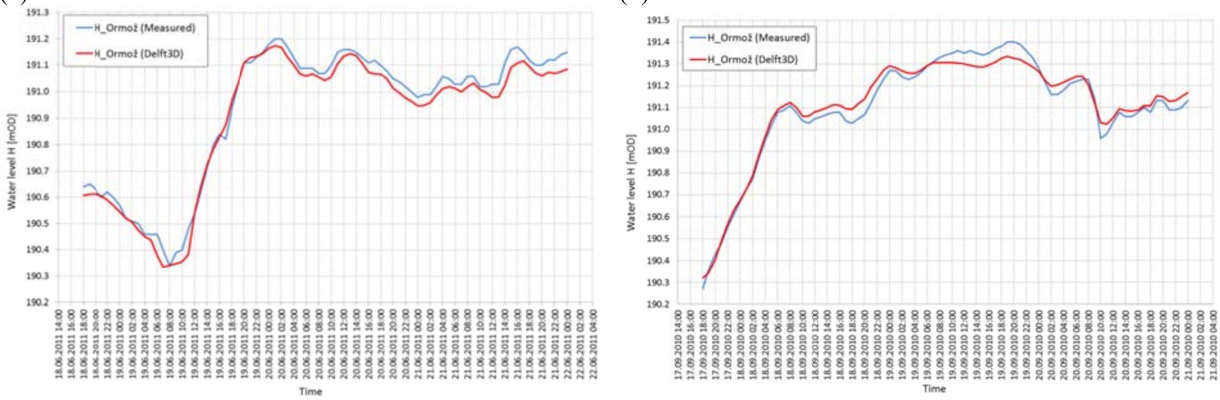


Fig. 14 Model calibration and verification results of water level on g.s. Ormož

The adopted Mannings roughness coefficients are as following: 0.027 $m^{-1/3}s$ for the River Drava's old channel from the HPP Formin dam to g.s. Borl, 0.026 $m^{-1/3}s$ for the River Drava's old channel from g.s. Borl to the HPP Varaždin dam, 0.180 $m^{-1/3}s$ for the woodland areas and 0.200 $m^{-1/3}s$ for the pasture areas.

4.2 Results of the model simulations

After the development and the calibration of the model several flood wave simulations were performed. The upstream open model boundaries were the recorded discharges of the HPP Formin dam and the powerhouse (**Fig. 9**), and the downstream boundary (Hrwl) were the recorded upper water levels of the HPP Varaždin dam (**Fig. 11**). The discharges of the Dravinja and Pesnica streams were also available (**Fig. 10**).

The specific objective of the hydraulic analysis was to determine the discharge through the HPP outlet canal as a result of embankment breaches and its impact on the measured discharges and water levels on g.s. Ormož. In order to obtain the timing and the peaking of the recorded water levels on g.s. Ormož the breaches of the HPP Formin outlet canal had to be included in the model. The breaches were modelled by the diverting of a provisional discharge from the overbank area of the River Drava (near the breach) into the outlet canal. In the simulations the following parameters were varied: the amount of flow diverted into the outlet canal, and the starting time of diversion (levee breaching) which was different for the upstream and the downstream breach. The model also enabled the free over-spilling of the Virje Otok-Brezje levee.

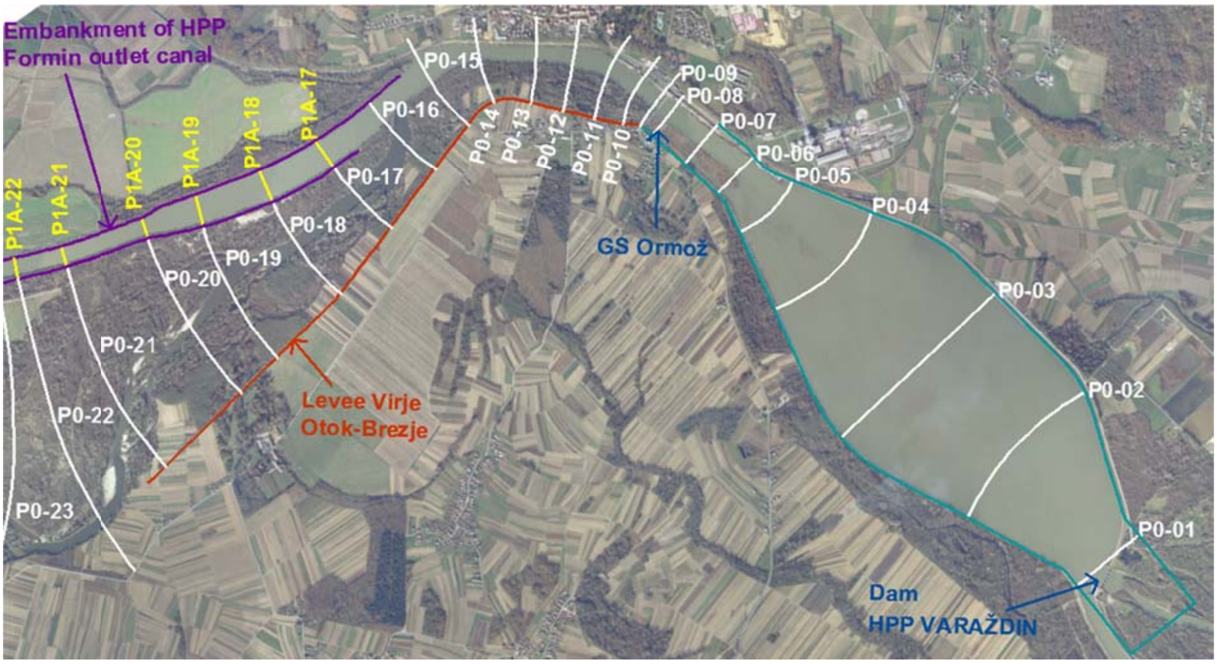


Fig. 15 Locations of control cross sections

Fig. 15 shows the locations of model control cross sections along the River Drava's old channel (P0-01 to P0-23) and along the HPP Formin outlet canal (P1A-17 to P1A-34). The most interesting cross sections on the River Drava's old channel are: cross section P0-17 located before HPP Formin outlet canal restitution into the River Drava's old channel, cross section P0-16 located after the restitution and cross section P0-08 at g.s. Ormož. The interesting cross section on the HPP Formin outlet canal is P1A-17 located before the canal restitution into the River Drava's old channel.

Fig. 16 shows measured and calculated water levels on g.s. Ormož for simulations A and B. The simulation A includes the levee breaches of the outlet canal and the simulation B is without the breaches.

The model results show that the deviations of the measured and calculated water levels for the simulation A (red solid line) are within 10 cm. The results of the simulation A are given until the 6th Nov at 11:00 hrs, after which the Virje Otok-Breze levee breach occurred. In this way another unknown element was avoided but the quality of the hydraulic analysis was retained because the peak flow on the HPP Varaždin occurred before 11:00 hrs.

The results for the simulation B show that for the case without levee breaches, the water level rising on the g.s. Ormož (red dashed line) is significantly slower and the peak computed water level is -60 cm lower than the highest recorded water level.

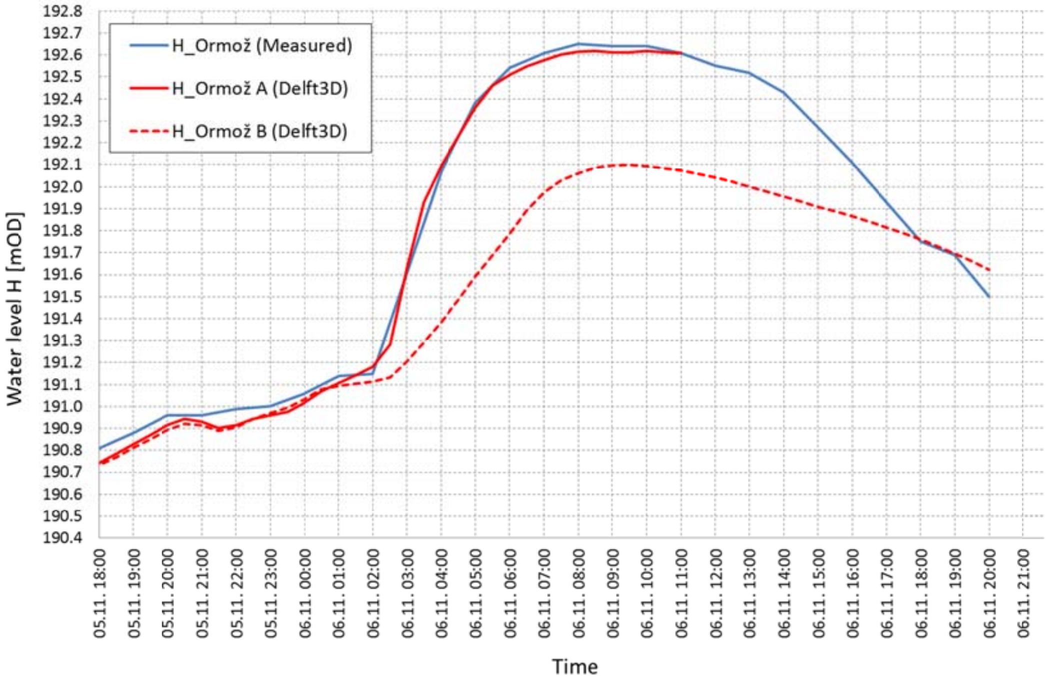
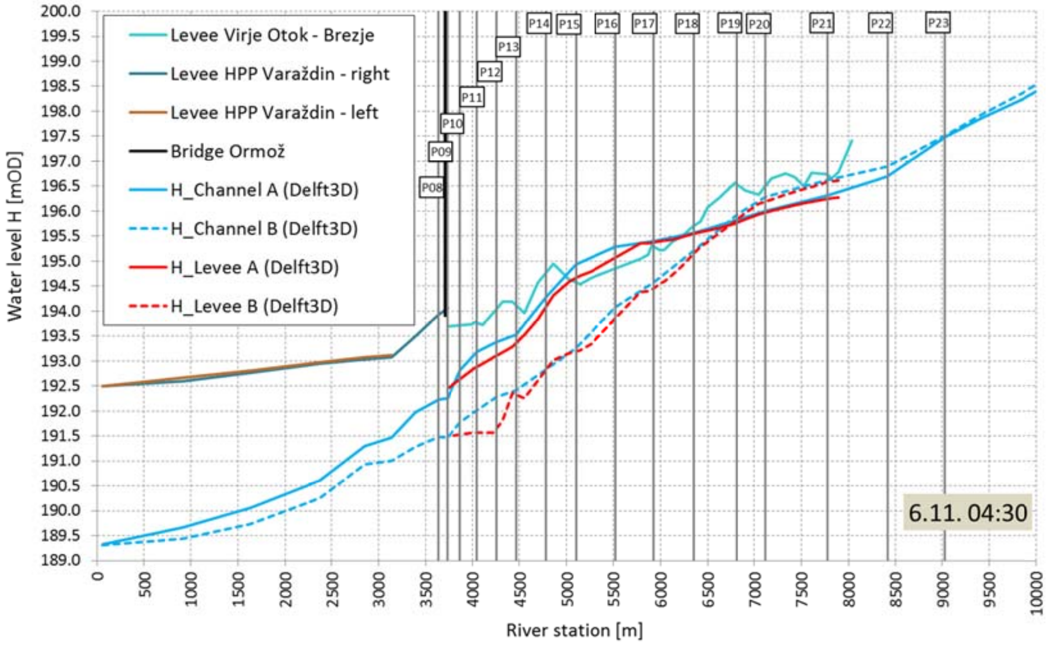


Fig. 16 Comparison of measured and computed water levels values on g.s. Ormož

The longitudinal water level profiles from simulations A and B are presented for two characteristic timings: for the beginning of over-spilling of the Virje Otok-Brezje levee on 6/Nov/2012 at 04:30 hrs (**Fig. 17a**) and for the peak flow on the HPP Varaždin on 6/Nov/2012 at 08:00 hrs (**Fig. 17b**). The computed water levels for simulation A show that over-spilling of the Virje Otok-Brezje levee begins at 04:30 hrs which fully corresponds to the actual event. At the time of the flow peaking on the HPP Varaždin, the over-spilling of the Virje Otok-Brezje levee occurs on a 1800 m long section between cross sections P0-14 and P0-19. This section has cca 50 cm higher flood levels than the levee crest levels (**Fig. 17b** and **Fig. 18**). The simulation B results show that the overtopping of the Virje Otok-Brezje levee is significantly lower in comparison to the simulation A.

The spatial views of flood extents are given for simulations A for the beginning of the over-spilling of the Virje Otok-Brezje levee (**Fig. 19a**) and for the peak flow on the HPP Varaždin (**Fig. 19b)**. On the longitudinal profiles the over-spilling of the Virje Otok-Brezje levee is visible at 04:30 hrs but on the flood extent it is visible at 05:00 hrs.

(a) 6/Nov, time 04:30 hrs, start of over-spilling of the Virje Otok-Brezje levee



(b) 6/Nov, time 08:00 hrs, time of peaking on HPP Varaždin

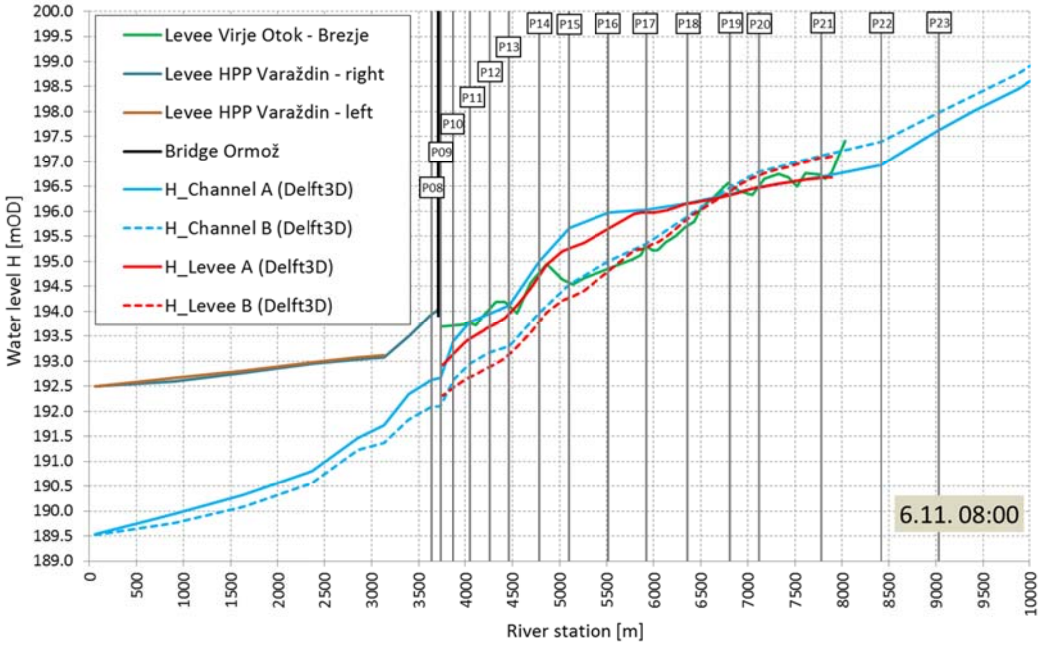
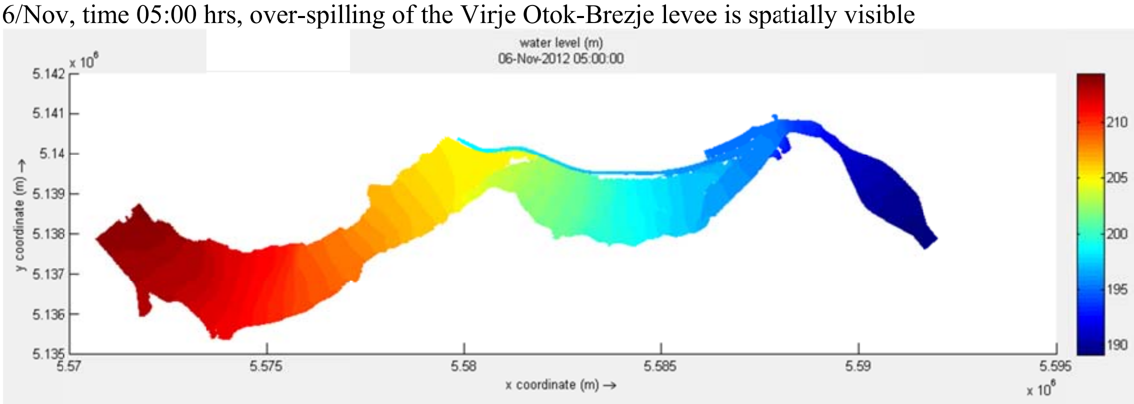


Fig. 17 Longitudinal profile of computed water levels from simulations A and B with levee crest level and locations of control cross sections

Fig. 18 Over-spilling of the Virje Otok-Brezje levee (time unknown, source Internet)

(a) 6/Nov, time 05:00 hrs, over-spilling of the Virje Otok-Brezje levee is spatially visible



(b) 6/Nov, time 08:00 hrs, time of flood wave peaking on the HPP Varaždin

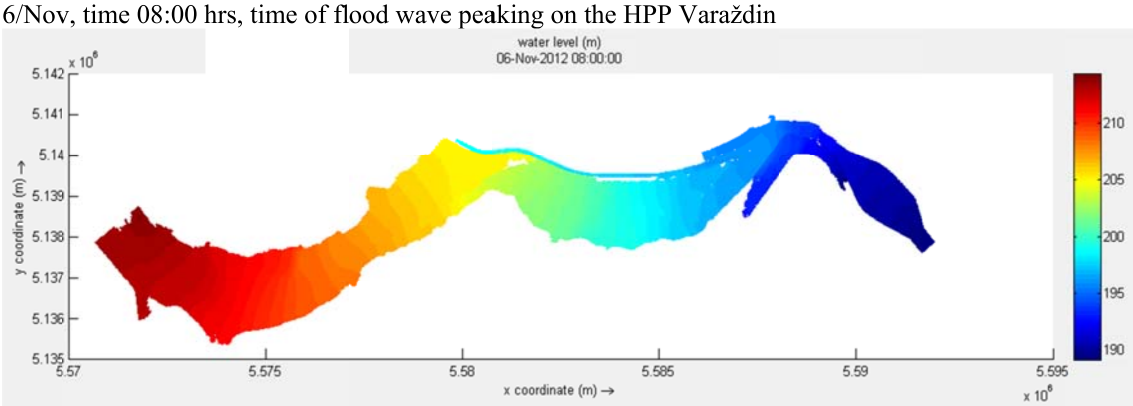


Fig. 19 Spatial view of computed water levels from simulation A

Fig. 20 shows the discharges on cross sections P0-17, P1A-17, P0-16 and P0-08 from simulation A (solid line) and from simulation B (dashed line). The measured peak flow on HPP Varaždin was 3311 m^3/s on the 6^{th} Nov at 08:00 hrs. The peak flow on g.s. Ormož (P0-08) is 3335 m^3/s on the simulation A (6^{th} Nov at 08:00 hrs), and is 2600 m^3/s on simulation B (6^{th} Nov at 09:00 hrs). In order to achieve the measured water level on g.s. Ormož, the discharge separation at the peak is 2400 m^3/s through the HPP Formin outlet canal (P1A-17) and 1270 m^3/s through the River Drava's old channel (P0-17). At the peak flow the over-spilling of the Virje Otok-Brezje levee reaches a value of 520 m^3/s (**Fig. 21**).

The results for simulation B show that the over-topping of the outlet canal occurs in the downstream part even without levee breaching. The discharge separation at the peak is 400 m^3/s in outlet canal (P1A-17) and 2100 m^3/s through the River Drava's old channel (P0-17).

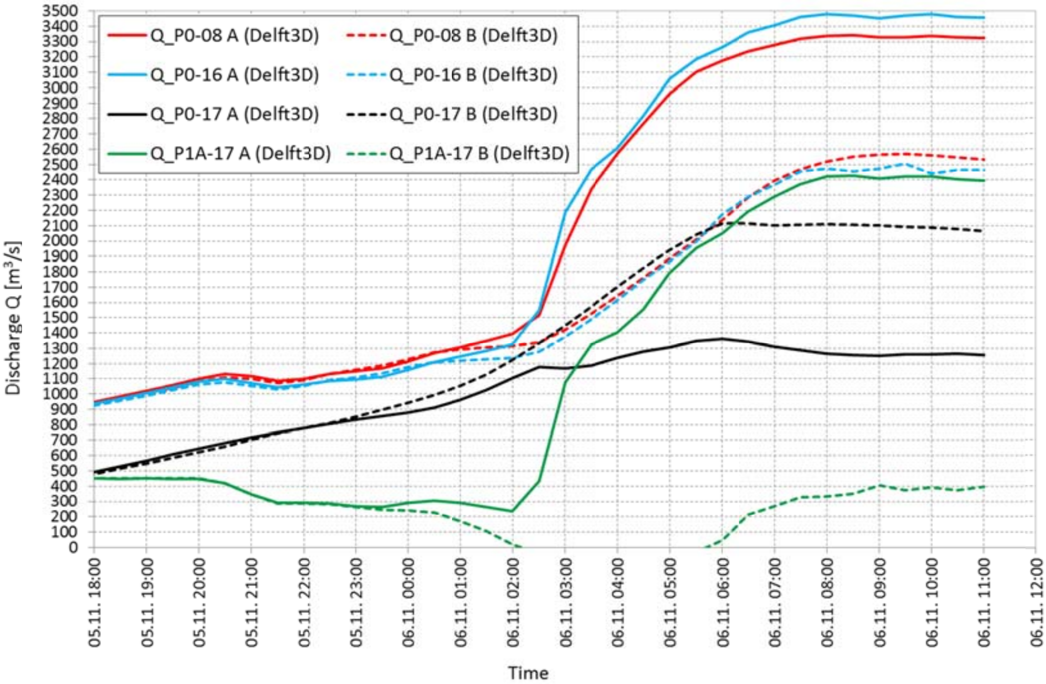


Fig. 20 Computed discharges at control cross sections

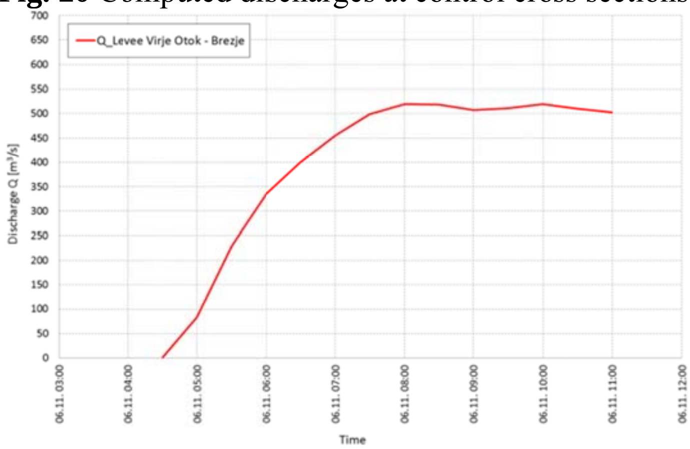


Fig. 21 Computed over-spilling over the Virje Otok-Brezje levee

5 CONCLUSIONS

The 5/Nov/2012 flood wave on the River Drava was the highest flood event in the last 60 years in the Croatian part of the river. The flood wave on g.s. Borl was between a 100 and a 1000 years return period when compared to the theoretical flood waves, but when compared to the estimates of total input flow rates this was a 200 years return period flood. The flow hydrographs on HPPs in Slovenia and Croatia showed an unexpected flood wave transformation in the area between the HPP Formin and the HPP Varaždin.

The levee breaches on the HPP Formin outlet canal caused the flow diversion from the River Drava's old channel to the outlet canal. The mathematical simulations showed that that sudden increase of recorded water levels and discharges on the HPP Varaždin can only be a consequence of this levee breaching of the HPP Formin outlet canal. The sudden flow increase on HPP Varaždin was a result of the flow acceleration in the outlet canal and of the shorter flood wave duration. The flood wave propagation through the outlet canal was also the most likely cause of over-topping of the Virje Otok-Brezje levee, as the simulation B without breaches showed negligible over-spilling of the levee.

The case without breaches (simulation B) showed that over-spilling to the outlet canal occurs even without the levee breaches. Considering the damages to the flood protection systems and the surrounding areas, further considerations to the design of the HPP Formin outlet canal and the Virje Otok-Brezje levee are required.

On the 28 km long section of the River Drava's old channel between HPP Formin and HPP Varaždin there are two critical subsections for flood wave propagation. The first subsection is near settlement Završ where the valley width is narrowed from 2300 m to 800 m. This is also the section of the upper breach of the outlet canal levee. The second critical subsection is just upstream of the Ormož bridge. The conveyance on this section is significantly reduced as the confluence of the HPP Formin outlet canal and the old river channel is located on the narrowest river channel width. The over-topping of the Virje Otok-Brezje levee occurred on this second critical section. In order to increase the level of flood protection of the area, additional measures for the enhancement of flood wave conveyance should be considered.

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HYDRAULIC INTERACTION OF THE PERFORATED SEAWALL AND SMOOTH SUBMERGED BREAKWATER

D. Carevic[1], G. Loncar[2], M. Paladin[3]

Abstract

Coastal structures are used for transition of people and goods between sea and land and for protection of coast and internal waters. With an aim to provide safe transition of people and goods, calm sea should be ensured in front of coastline in order to achieve vessels without large movements and no-overtopping coastline.

This work deals with special type of coastal structure which consists of two common types of structures: perforated seawall and submerged breakwater positioned in front of it. A new mathematical model was developed based on the experimental measurements of wave parameters between such tandem. Experimental investigations were conducted in wave channel for monochromatic and spectral waves varying length between constructions (1,2m, 2,4m i 6,2m) and submergence of breakwater (0,06m and 0,1m). The total of 54 hydraulic tests were achieved varying wave and geometrical parameters.

Using a new developed mathematical models for monochromatic and spectral waves the analyse of hydraulic behaviour of such construction is presented and comparison with some other coastal constructions (only solid seawall, solid seawall in tandem with submerged smooth breakwater).

Keywords

hydraulic interaction, submerged breakwater, perforated seawall.

1 INTRODUCTION

Perforated seawall is sea defence construction which attenuates reflected waves and reduces amount of overtopping. Submerged breakwater causes wave breaking in front of the sea line and reduces wave energy which approach to seawall (in this work perforated seawall). Combination of this two structures provide attenuation of wave heights between them and

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consequently reduction of the seawall toe erosion, calm sea for berthed vessels and lower seawall crown.
The original descriptions of the perforated seawall hydraulic behaviour, based on heuristic approach, have been published in work [1]. A theoretical model, based on long wave theory, was developed in paper [2], for transmission and reflection coefficient calculation. The model assumed two parallel perforated walls without back wall, and superposition of linear incident and reflected waves. In work [3], a simple analytical model for regular waves was developed predicted for the calculation of the perforated wall reflection coefficients consisted of one perforated and one solid wall. Deriving a several mathematical models in [4], [5] and [6], for reflection characteristics estimation, authors have included complex caisson geometry, influence of foundation embankment and irregular waves. Some other works which deal with perforated structures are [7] and [8].

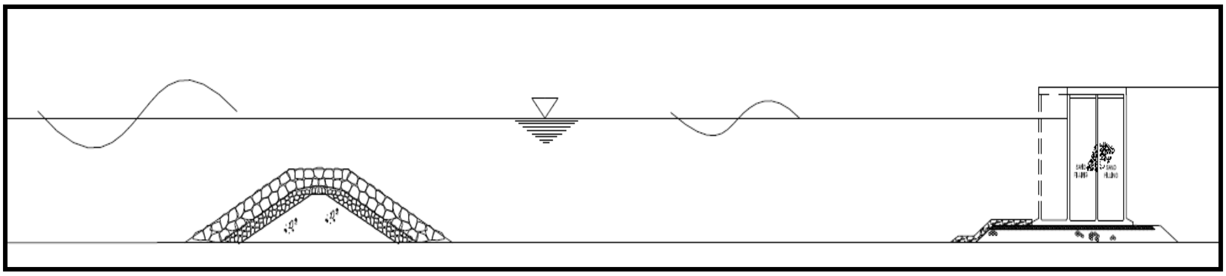

Fig. 1 Perforated seawall and submerged breakwater

LCS (low crested structure) is a type of rubble mound structure with emerged, submerged or zero freeboard causing the wave breaking and the dissipation of wave energy. LCS-s with rubble mound armour are usually used and their functional characteristics (transmission and reflection) are described in the works [9], [10], [11], [12], [13] and [14]. This paper deals with LCS with smooth armour, the type of structure rarely used. The possibility of generalizing the results of this work to be applied for rubble mound structures is obviously limited, so results of this work are mainly intended as basic research of such tandem hydraulic performance.
The defence of rubble mound breakwater with LCS positioned in front of it was investigated in paper [16], where the authors concluded that the run up and run down for the breakwater defenced by submerged LCS are reduced up to 30 and 60%. The damage of the optimally defenced breakwater is reduced by 40–100% compared to a non-defenced (single) breakwater.

The combination of smooth low crested structure and perforated seawall is the main objective of this paper, respectively, the wave attenuation caused by submerged breakwater and perforated seawall.

2 THEORETICAL MODEL

2.1 Theoretical Model for the Calculation of Wave Heights between Perforated Seawall and Smooth Submerged Low Crested Structure

The hydraulic interaction of the submerged LCS and the perforated seawall implies the following: 1. the influence of LCS on the wave heights, and 2. the influence of the perforated seawall on the wave heights.
Part of the wave energy is transmitted over the submerged breakwater in the form of the transmitted wave height H_t, (Fig. 2). Those waves travel toward perforated breakwater and reflect as reflected wave heights H_{tr}. Maximum wave heights which occur between LSC and perforated seawall are equal to summation of transmitted and reflected wave heights for regular waves (as it is indicated on Fig. 2).

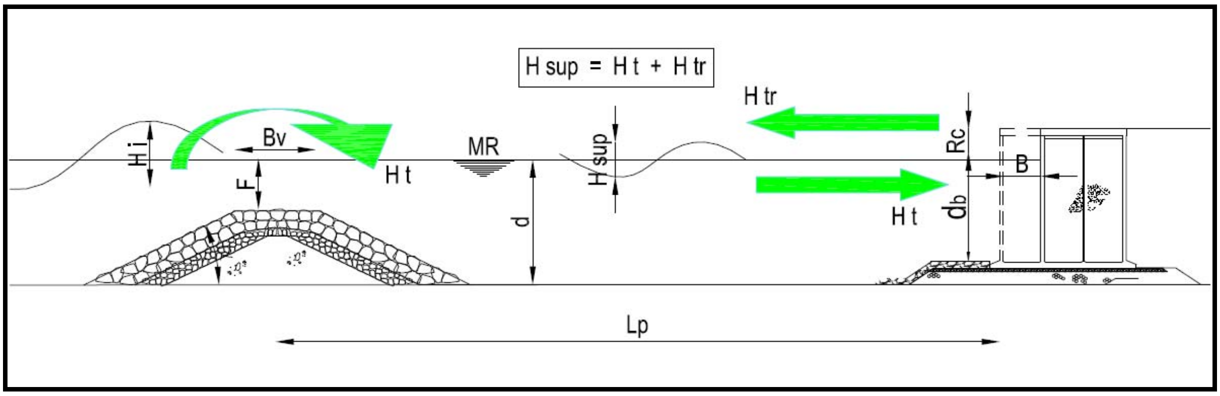

Fig. 2. Definition sketch of smooth submerged LCS and perforated seawall interaction for regular waves

Irregular waves are usually described by spectral and statistical methods. Short-term wave situation is defined by wave energy spectral density function $S_{\eta}(f)$. In the case of opposite direction traveling waves, the superposed significant wave height ($H_{s\text{-}sup}$) could be calculated according to [18] as it is indicated on Fig. 3.

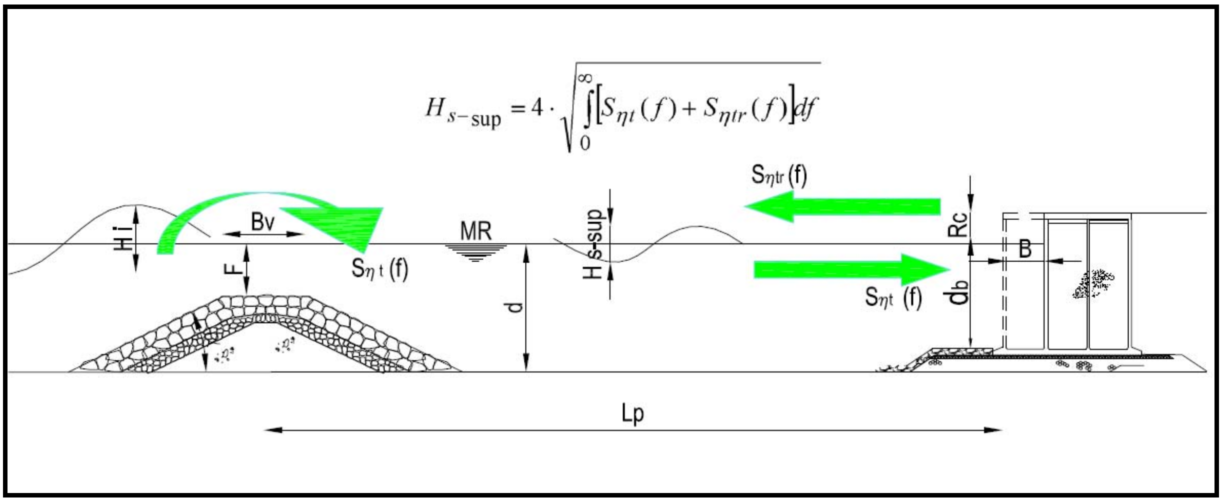

Fig. 3 Definition sketch of smooth submerged LCS and perforated seawall interaction for irregular waves

Transmission over the breakwater and reflection from seawall depend on geometrical characteristics and incident waves parameters. Both phenomena are well described by existing mathematical models. In this work, existing mathematical models for submerged breakwater (chapter 2.3) and perforated seawall (chapter 2.2) will be used for a new mathematical model development, with the aim of the description of the hydraulic performance of such tandem. A new formed mathematical model will allow calculation of wave heights between submerged breakwater and perforated seawall for different incident wave parameters and geometries of structures.

2.2 Theoretical Model for Perforated Seawall Reflection Coefficient Calculation

Regular waves

In the paper [3] authors have derived analytical model for reflection coefficient calculation based on assumption of regular long-crested waves and constant depth of the water in dissipation chamber and in front of the perforated wall.

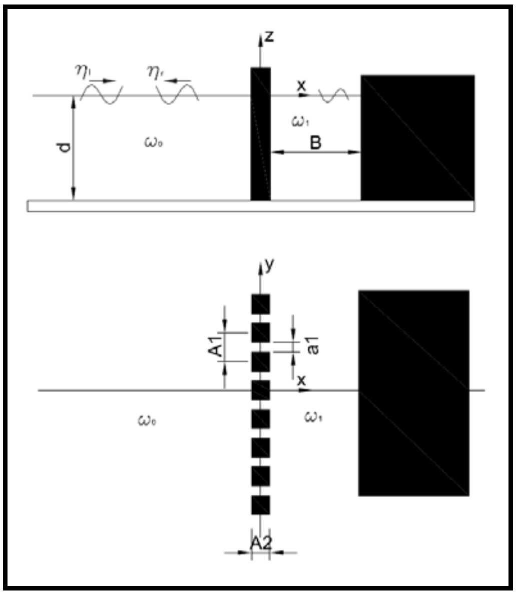

Fig. 4 Sketch of the perforated seawall

They separated domain in two regions, one in dissipation chamber and second one at the outer side. In each region they assumed incident and reflected wave velocity potential. Total potential was assumed as sum of potentials for incident and reflected waves. They solved system of differential equations with assumption that loss of potential energy on the perforated wall is according to [2], [19]:

$$\alpha = \left(\frac{1}{pC_C} - 1 \right)^2 \tag{1}$$

where are:

p porosity of perforated wall,

C_C contraction coefficient of the jet from the perforation hole, C_C=0,4-0,8 according to [19] and equation $C_C = 0,6 + 0,4p^2$ according to [20].

Solving differential equations they have got system of independent linear equations which give:

$$K_R = \frac{\left[\left(G^2 + W^2\right)^2 + W^2 R^2 \left(W^2 R^2 + 2G^2 - 2W^2\right)\right]^{0,5}}{G^2 + W^2 (1+R)^2} \tag{2}$$

where are:

$$P = lk \tag{3}$$

$$R = \beta\left(\frac{k}{\omega}\right) \tag{4}$$

$$W = \tan(kB) \tag{5}$$

$$G = 1 - PW \tag{6}$$

$$l = 2C; \quad C = \frac{A2}{2}\left(\frac{A1}{a1} - 1\right) + \frac{2A1}{\pi}\left[1 - \log\left(\frac{4a1}{A1}\right) + \frac{1}{3}\frac{a1}{A1} + \frac{281}{180}\left(\frac{a1}{A1}\right)^4\right] \tag{7}$$

$$\beta = \frac{8\alpha}{9\pi} H\omega \frac{W}{\sqrt{W^2 (R+1)^2 + G^2}} \frac{5 + \cosh(2kd)}{2kd + \sinh(2kd)} \tag{8}$$

Reflected wave height can then be defined as:

$$H_r = K_R \, H_i \tag{9}$$

Irregular waves

Mathematical model for reflection coefficient calculation of irregular waves was developed in paper [21]. Model is based on above described model for regular waves. Methodology is based on application of regular wave’s model on each spectral component independently. Dissipation coefficient β is calculated for each component with a same wave height H_{rms}-root mean square wave height. Each component of wave spectrum is denoted with subscript „n“:

$$K_{R_n} = \frac{\left[\left(G_n{}^2 + W_n{}^2\right)^2 + W_n{}^2 R_n{}^2 \left(W_n{}^2 R_n{}^2 + 2G_n{}^2 - 2W_n{}^2\right)\right]^{0,5}}{G_n{}^2 + W_n{}^2 (1+R_n)^2} \tag{10}$$

$$P_n = lk_n \tag{11}$$

$$R_n = \beta_n\left(\frac{k_n}{\omega_n}\right) \tag{12}$$

$$W_n = \tan(k_n B) \tag{13}$$

$$G_n = 1 - P_n W_n \tag{14}$$

$$\beta = \frac{8\alpha}{9\pi} H_{rms}\omega_n \frac{W_n}{\sqrt{W_n^{\,2}(R_n+1)^2 + G_n^{\,2}}} \frac{5+\cosh(2k_n d)}{2k_n d + \sinh(2k_n d)} \tag{15}$$

$$H_{rms} = \frac{4{,}004\sqrt{m_0}}{1{,}416} \tag{16}$$

where is:

m_0 zero momenth of incident wave spectrum, [m^2]

If discrete distribution K_{Rn} is transformed to continuous curve $K_R(\omega)$, the reflected wave spectrum can be solved as:

$$S_{\eta r}(\omega) = K_R^{\,2}(\omega)\, S_{\eta i}(\omega) \tag{17}$$

2.3 Theoretical Model for Calculation of Transmission Coefficients over Smooth LCSs

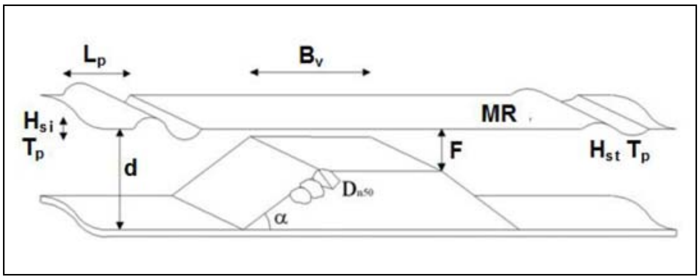

Fig. 5 Definition of symbols for smooth LCS theoretical model

LCS need not be necessarily covered with rock fill. Sometimes smooth and impermeable LCS can be covered with asphalt or concrete armour. The slopes of these LCSs are sometimes more gentle (1:3 or 1:4) than it is the case with the LCS with the stone armour, mostly due to construction reasons.

The asphalt and concrete LCSs are mostly built in dry conditions, and not under the water. The presence of tides enables the building of such structure in dry conditions.

Since the smooth LCSs are different in the process of hydraulic functioning than the breakwaters covered with rock, there are different formulas for the transmission coefficient. The wave transmission can be calculated according to the paper [15]:

$$K_T = [-0.3F/H_{si} + 0.75[1-\exp(-0.5\xi)]]\cos^{2/3}\beta \tag{18}$$

with the minimum K_T=0.075 and the maximum K_T=0.8, and with the following limitations: 1 < ξ < 3, 0° ≤ β ≤ 70°, 1 < B_v/H_{si} < 4. The symbols are :

F	-water depth at the crown, [m],
H_{si}	-significant wave height in front of LCS ($H_{si} = 4\sqrt{m_0}$), [m],
ξ	-Irribaren number, $\xi = tg\alpha/(s_{op})^{0.5}$, $s_{op} = 2\pi H_{si}/(gT_p^2)$,
B_v	-crown width, [m]

Eq.(18) takes the angle wave transmission into consideration by means of the expression $\cos^{2/3}\beta$.

3 PHYSICAL MODEL

3.1 Wave Channel and Measurement Equipment

The experimental research was made in the Laboratory of the Faculty of Civil Engineering in Zagreb. The wave channel width was 1m, the height was 1.1 m, and the depth of water in the channel was d=0.5 m.
The measuring equipment includes the piston wave generator with the installed AWACS system, and the data collection system (sampling frequency 40Hz) produced by DHI (Horsholm, Denmark). Capacitive gauges (DHI) G1-G8 (Fig. 6) were used for measuring the surface elevation. The analysis of the collected data was made by means of the system DHI Wave Synthesizer. The incident wave parameters in front of LCS were determined in the spectral domain by means of the WS Wave Reflection Analysis. The spectral analyses were performed with the following parameters: size of FFT block: 512, overlap: 0.667, Number of subseries: 68, lower cut-off frequency: 0.0 Hz, higher cut-off frequency: 20.0 Hz, Data window: Hanning method, frequency step: 0.078 Hz.

3.1.1 Physical Model of the Perforated Seawall (inter 0)

A wooden model of the perforated seawall was placed into the wave channel at the distance of 15.7m from the wave generator. The model of the perforated seawall was made of wood with the porosity of p=30% (p-ratio of the opening and the total surface of the wall) and with vertical longitudinal openings, and with the width of dissipation chamber of B=0.18m.

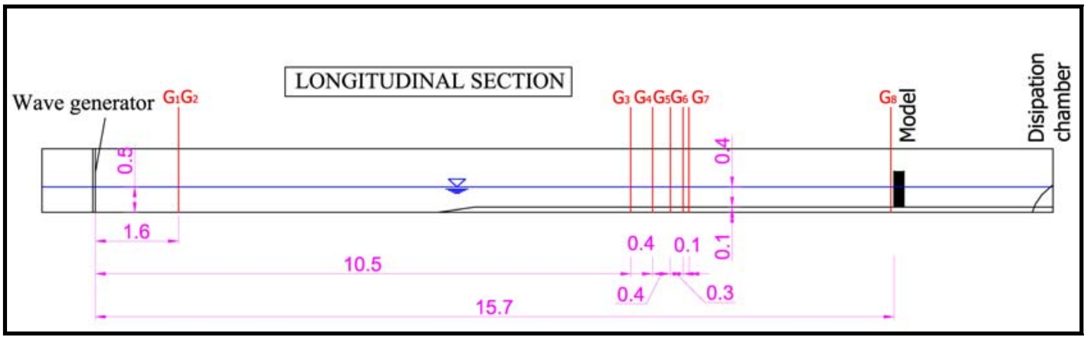

Fig. 6. Longitudinal section of wave channel with capacitive gauges (G1-G8) and perforated seawall (Model) positions (Inter 0)

Tab. 1 Wave parameters used in experiments without LCS (inter 0) and with LCS (inter 1-6)

JONSWAP; γ=3.3, σ_1=0.07; σ_2=0.09;				
Test	T_p [s]	H_s [m]	L_p [m]	L_p/H_s
1	0.68	0.06	0.72	12
2	0.81	0.06	1.02	17
3	1.01	0.06	1.50	25
4	0.89	0.10	1.20	12
5	1.10	0.10	1.70	17
6	1.45	0.10	2.50	25
7	0.99	0.12	1.44	12
8	1.24	0.12	2.04	17
9	1.68	0.12	3.00	25

3.1.2 Physical Model of the Perforated Seawall and Submerged Smooth LCS Interaction (inter 1-6)

The model of the smooth submerged breakwater was made of wood with the crown width of B_v=0,16m, the slopes of 1:2 and the possibility to change the submergence depth so that two depths 0.055m and 0.101m can be achieved (Fig. 7).

Fig. 7. Photographs of submerged low crested structure (LCS) positioned in channel for crown submergence F=0.055m

During the testing the distance of the LCS from the perforated seawall L_{pl} and seawall submergence F varied. There were three distances of the breakwater from the seawall L_{pl}=1.2m, 2.4m and 6.2m used, as well as two submergences F=0.055m and F=0.101m. In this way the total of 6 combinations was obtained. The combinations are called inter1, inter2, inter3, inter4, inter5 and inter6 (Fig. 8).

There were altogether 9 testing procedures for irregular waves conducted according to the Tab. 1. For regular wave the same values as in Tab. 1 have been used: $T=T_p$, $H=H_s$ and $L=L_p$. The wave parameters from Tab. 1 were used to perform experimental testing for each single interaction, (inter1÷6). Thus, altoghether 54 testing procedures were conducted.

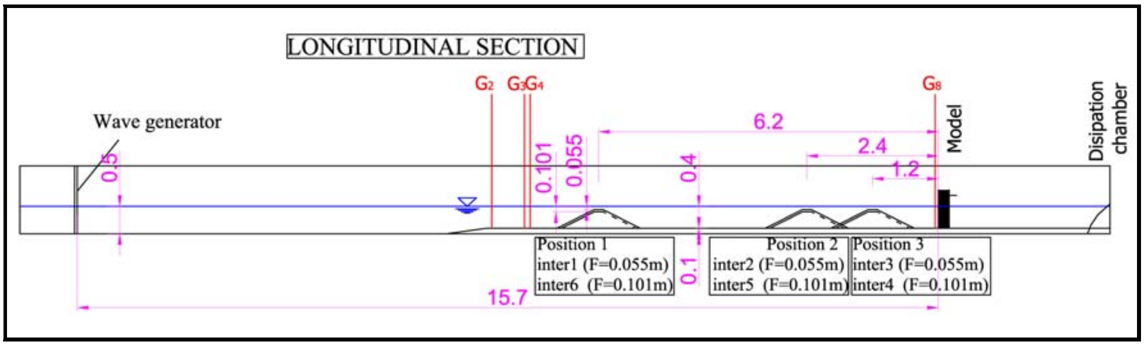

Fig. 8. Longitudinal section of wave channel with the positions of submerged LCS and perforated seawall for three different pool distances, L_{pl}=**1.2m, 2.4m and 6.2m** and two LCS submergence F=**0.055m and F=0.101m**, (inter1÷6)

The gauges G2-G4 were used to measure the oscillations of the water surface in front of the submerged breakwater and gauges G5-G7 between breakwater and seawall. The separation of incident and reflected spectra from the records on the gauges G2-G4 was undertaken by means of the method defined in the [22]. The incident H_{si}, reflected H_{sr} and superposed $H_{s\text{-}sup}$ significant wave heights can be obtained from the incident and reflected spectra.
The time duration for an experiment amounts to ~5 min., which is approx. three hundred waves per an experiment, pursuant to the recommendations from the paper [23]
The calibration of the mathematical model presented in chapter 2.2 was conducted using calibration coefficient Cc and initial measurements on model with only perforated seawall (Fig. 6). The calibration of the mathematical model presented in chapter 2.3 was conducted only for regular waves with aim to get satisfactory agreement of the Eq. 18 with measurements conducted in wave channel only with submerged breakwater. The agreement of the measurements and Eq. 18 for irregular waves was satisfactory.

4 RESULTS

Verification of a new formed mathematical models were conducted by comparison of H_{sup} for regular waves and $H_{s\text{-}sup}$ for irregular waves (Fig. 2 and Fig. 3.) obtained experimentally and theoretically (chapter **Chyba! Nenašiel sa žiaden zdroj odkazov.**). Verification is presented on the Fig. 9 and Fig. 10.

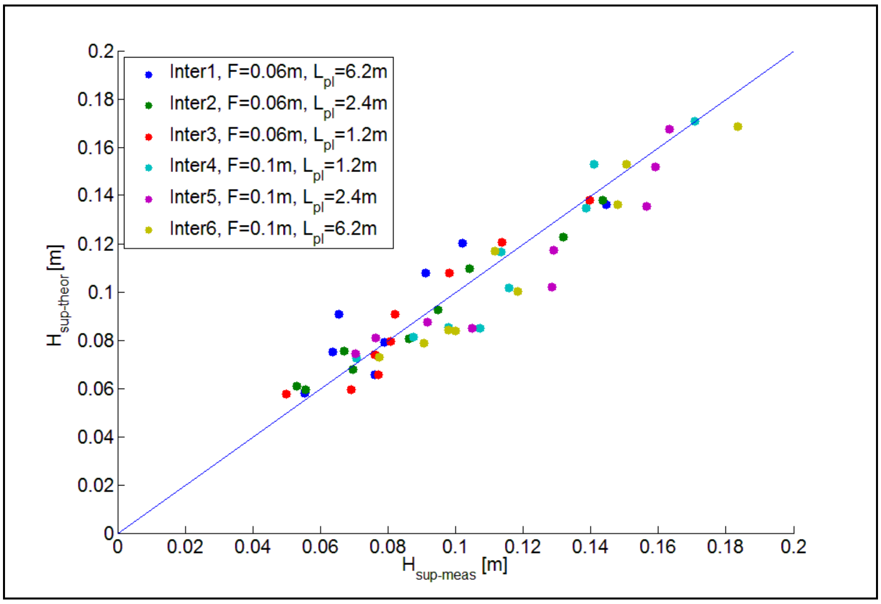

Fig. 9 Verification of the newly formed mathematical model for regular waves by results from wave channel measurements

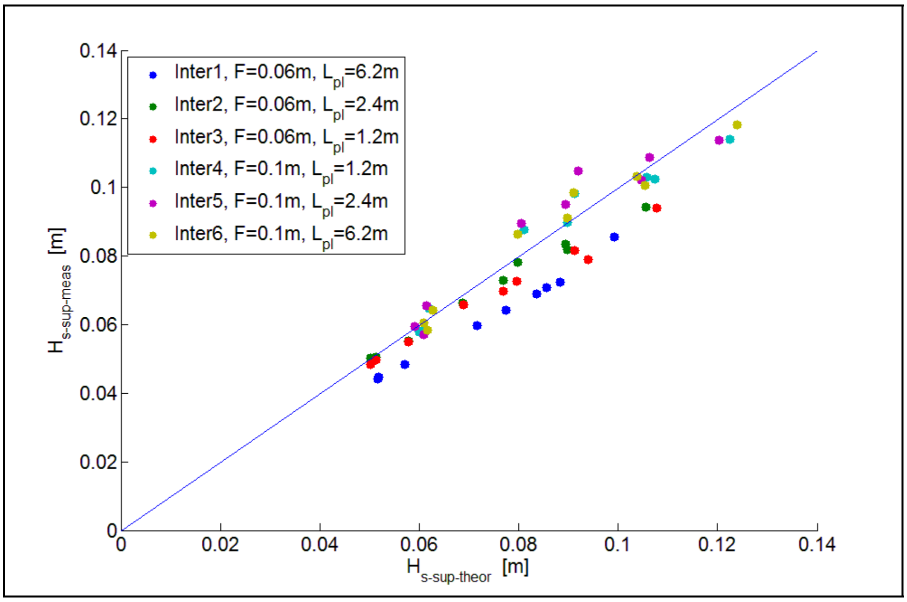

Fig. 10 Verification of the newly formed mathematical model for irregular waves by results from wave channel measurements

It could be concluded that a newly formed mathematical models represent well experimental results and they can be used in geometrical and wave parameters limits of the verification process.
Fig. 11 shows examples of the theoretical and experimental wave spectra reflected from perforated seawall. The agreement is satisfactory because mathematical model represents well energy around peak period as well as occurrence of the energy on higher harmonics.

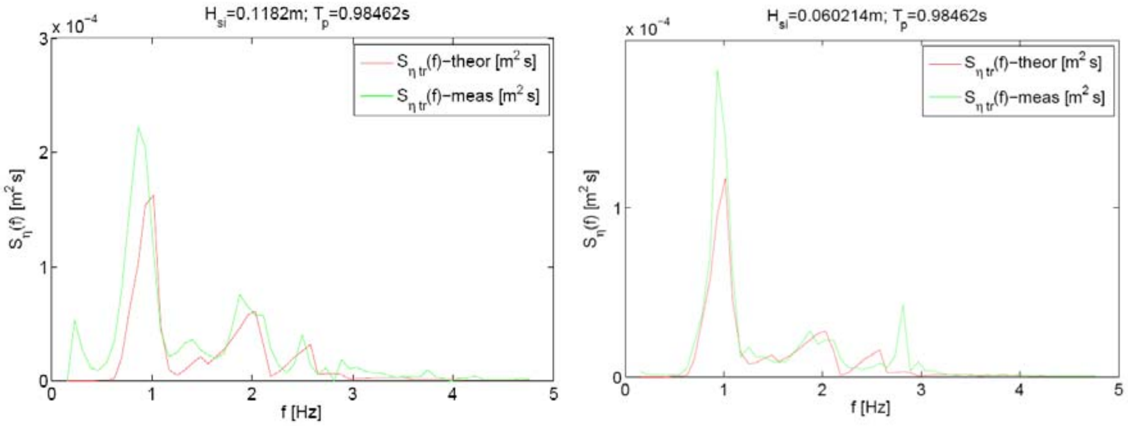

Fig. 11 Examples of theoretical and measured wave spectra reflected from perforated seawall

The variation of the pool length L_{pl} is involved in experimental research with aim to investigate the influence of L_{pl} on the hydraulic performance of such tandem, especially on superposed significant wave height H_{sup} and $H_{s\text{-}sup}$. General conclusion is that there are no obvious confirmations of pool length influence on H_{sup} and $H_{s\text{-}sup.}$

5 DISCUSSION

Further will be presented the comparison of the hydraulic behaviour of:

1. tandem submerged breakwater+perforated seawall (breakw+perf),
2. tandem submerged breakwater+solid seawall (breakw+solid) and
3. only solid seawall.

Superposed wave heights H_{sup} for only solid wall were calculated as superposition of incident and reflected wave heights with reflection coefficient K_R=1. H_{sup} for submerged breakwater and solid seawall were calculated as superposition of wave heights transmitted across submerged breakwater and waves reflected from solid seawall with reflection coefficient K_R=1. H_{sup} for submerged breakwater and perforated seawall were calculated according to new mathematical model presented in chapter **Chyba! Nenašiel sa žiaden zdroj odkazov.**.
Fig. 12 shows relationship of parameter H_{sup}/H_i and relative submergence F/H_i. This way is possible to analyse the influence of different types of coastal defence constructions on wave height H_{sup}.
Black dotted line presents theoretical H_{sup} in front of the only solid wall. In that case all wave energy is reflected what gives H_{sup}/H_i=2 (H_{sup}=2 H_i) and it is independent of the parameter $F/H_{i.}$
The other lines are separated according to the wave steepness (H/L=1/12, 1/17 and 1/25).
Lines “breakw+solid” presents the amount of energy dissipated by submerged breakwater in comparison to the situation without breakwater (“solid wall”). Curves “breakw+perf” are positioned lower than “breakw+solid” which represents additional energy dissipated by perforated wall.
Lines “breakw+solid” have breakpoints when they reach value H_{sup}/H_i =2. In those points breakwater transmits all incident energy and for greater values of relative submergences F/H_i

the tandem "breakw+solid" behaves like only solid wall. The same breakpoints are visible on curves "breakw+perf". Curvature of the curves "breakw+perf" is resulted because the reflection coefficients of the perforated seawall depend on the incoming wave lengths. The function of the reflection coefficient in relationship to incoming wave height (and length) has parabolic shape.
Colored dotted curves represent limits of the parameter H_{sup}/H_i for the variation of the input wave heights within $H_{i1.1} = 1.1 \cdot H_i$ and $H_{i0.9} = 0.9 \cdot H_i$.
The agreement between measurements and curves "breakw+perf" are satisfactory and it can be concluded that newly formed mathematical model describe well hydraulic behaviour of the submerged smooth breakwater and perforated seawall.
The same presentation as those on Fig. 12 could be produced but for submergence F=0.1m. Because of similarity with Fig. 12 this presentation is omitted.

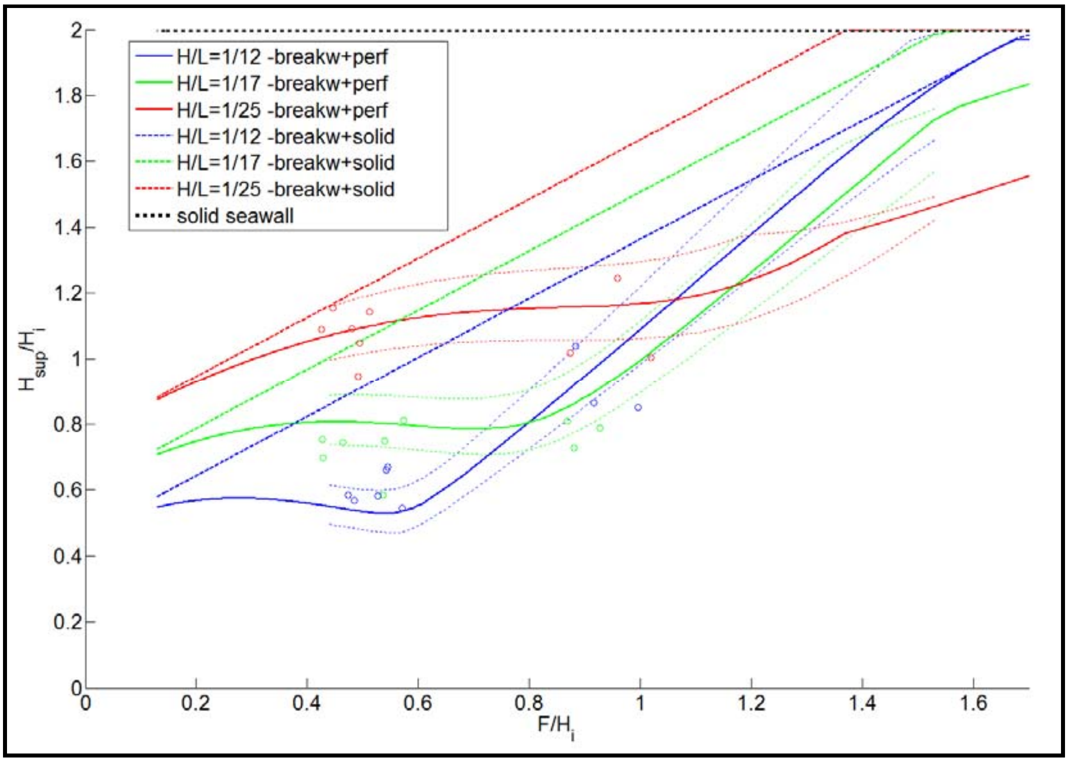

Fig. 12 Comparison of the parameter H_{sup}/H_i for only solid seawall, tandem submerged breakwater and solid seawall (breakw+solid), tandem submerged breakwater and perforated seawall (breakw+perf), F=0.06m, regular waves, "o"-measurements, "····"- upper limit for $1.1 \cdot H_i$ and lower limit for $0.9 \cdot H_i$

Fig. 13 shows the same as previous figure but for irregular waves. It is visible that in case of irregular waves parameter $H_{s\text{-}sup}/H_{si}$ can be maximum 1.41 what is in accordance to measurements presented in [18]. In the case of the irregular waves the assumption of the superposition of wave energies is valid (Fig. 3) unlike in case of the regular waves where the assumption of the summation of the wave heights is valid (Fig. 2.)
In case of the irregular waves the influence of the perforated seawall on the energy dissipation is less than in the case of the regular waves. That is visible from fact that curves for "breakw+perf" are at lower position from "breakw+solid" in case of the regular waves. The

main reason is in fact that presentation with spectral parameters as significant wave heights, includes all components from wave spectra which reflects with different reflection coefficient, which finally gives smaller influence of the perforated seawall.

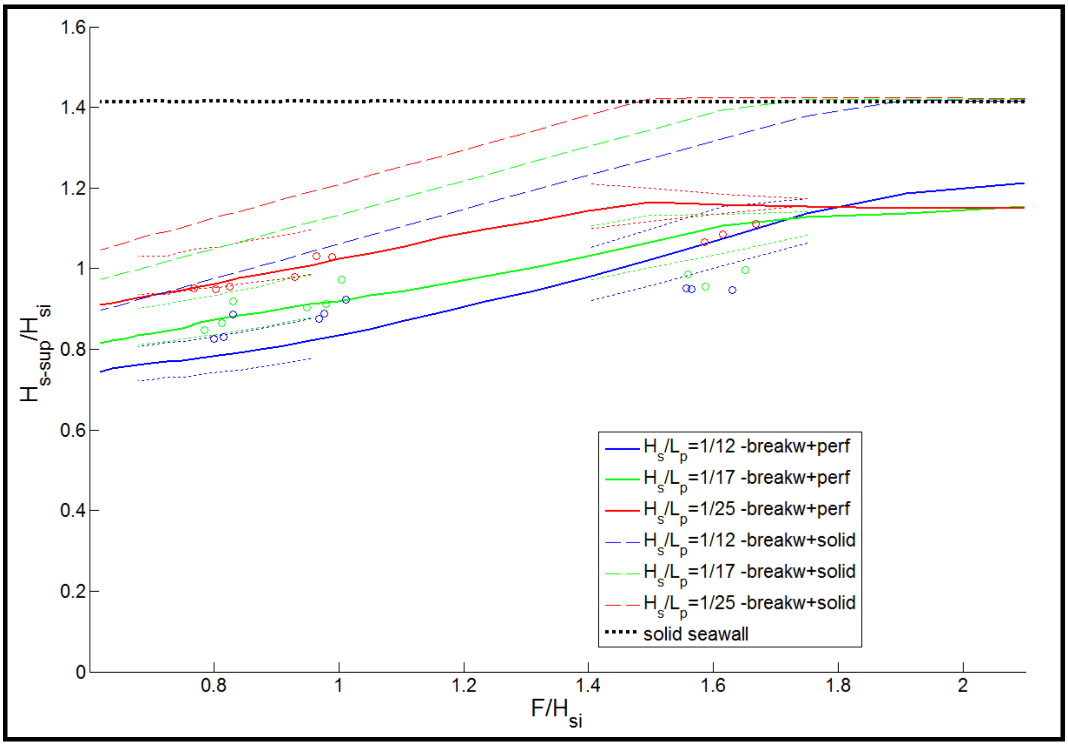

Fig. 13 Comparison of the parameter $H_{s\text{-}sup}/H_{si}$ for only solid seawall, tandem submerged breakwater and solid seawall (breakw+solid), tandem submerged breakwater and perforated seawall (breakw+perf), F=0.06m, irregular waves, "o"-measurements, "····"- upper limit for $1.1 \cdot H_{si}$ and lower limit for $0.9 \cdot H_{si}$

6 CONCLUSION

The experimental investigation, in wave channel, of the smooth submerged breakwater and perforated seawall is conducted with aim to form the mathematical model for calculation of the superposed wave heights between them. The mathematical model is formed from existing models for each type of construction. The verification of a newly formed mathematical model is conducted using results from measurements in wave channel.
Generally, the influence of the submerged breakwater on the energy dissipation is greater than the influence of the perforated seawall, if submergence F is small enough. The submerged breakwaters are cheap rubble mound constructions unlike reinforced concrete perforated seawalls. This leads to conclusion that the application of the submerged breakwater with the crown close to the zero water level in combination with solid seawall is better solution. Only in special cases with great tide oscillations and specific requests on submerged crown, the application of submerged breakwater and perforated seawall is acceptable.

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MULTIPURPOSE OBJECT ON THE SMALL HYDROPOWER PLANT

Lea Čubanová[1]

Abstract

The paper is dealing with the fish pass design at the small hydropower plant in the locality Podtureň at the Váh River. This object is planned as a multipurpose structure, which will be used for the fish migration and also for the recreational boat transport. These different requirements led to the creation of the mathematical model. The preliminary design was solved via one dimensional software HEC-RAS. By the geometry changing was achieved final design with suitable hydraulic parameters, such as velocities and depths for the specific fish region. For the better and more detailed result description was created also two dimensional model in the River2D software, where are clearly seen velocity fields as well as in the area of the slot. These hydraulic parameters in the slot are the most significant for the design consideration.

Keywords

Depth, Fish Pass, Mathematical Modelling, Slide, Velocity

1 INTRODUCTION

The water structure with small hydropower plant (SHPP) is planed about 1,3 km above the village Liptovský Ján and 600 m above Podtureň village, in the upper part of the Váh river on the right bank, in the chainage 357,00 km (it is cadastre of the villages Podtureň and Liptovský Ján). This structure is planned because of the hydropower utilization of the Váh river, for the electricity production. Energetic potential will created by the backwater of the water level upstream the weir and by the deepening of the river bed downstream the weir.

The location of the SHPP is under the junction of the Váh river and the Belá torrent. The Váh river in this section belongs to the middle-sized streams according to the discharge (long-time annual discharge $Q_a = 15{,}51\ m^3.s^{-1}$). The Belá river contributes to the discharges about 40 %.

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Favourable geological structure of the territory, high forested areas in the basin (60 %) create positive conditions for the discharge stabilisation. Longitudinal slope in the river bed of the affected area is 5,43 ‰. These factors (discharge conditions and big river bed slope) are giving very suitable conditions for the hydropower potential utilization. Discharge is not possible to accumulate either regulate, it must be utilised natural unaffected. Backwater is proposed at the level of the existed banks – elevation 622,00 m n. m. Utilised head is from 5,66 m to 3,50 m, depending on the discharge value (big head for small discharges and small for big ones). The hydropower plant is designed for the operation in the discharges range from 3,0 $m^3.s^{-1}$ to 23,50 $m^3.s^{-1}$ and will reach average production about 4300 MWh/year.

Main objects of the structure are:

- weir,
- hydropower plant with power outlet,
- biocorridor,
- river bed reconstruction (modifications).

In the chainage 357,00 km will be built weir, on its right side will be power plant and on the left side biocorridor [1].

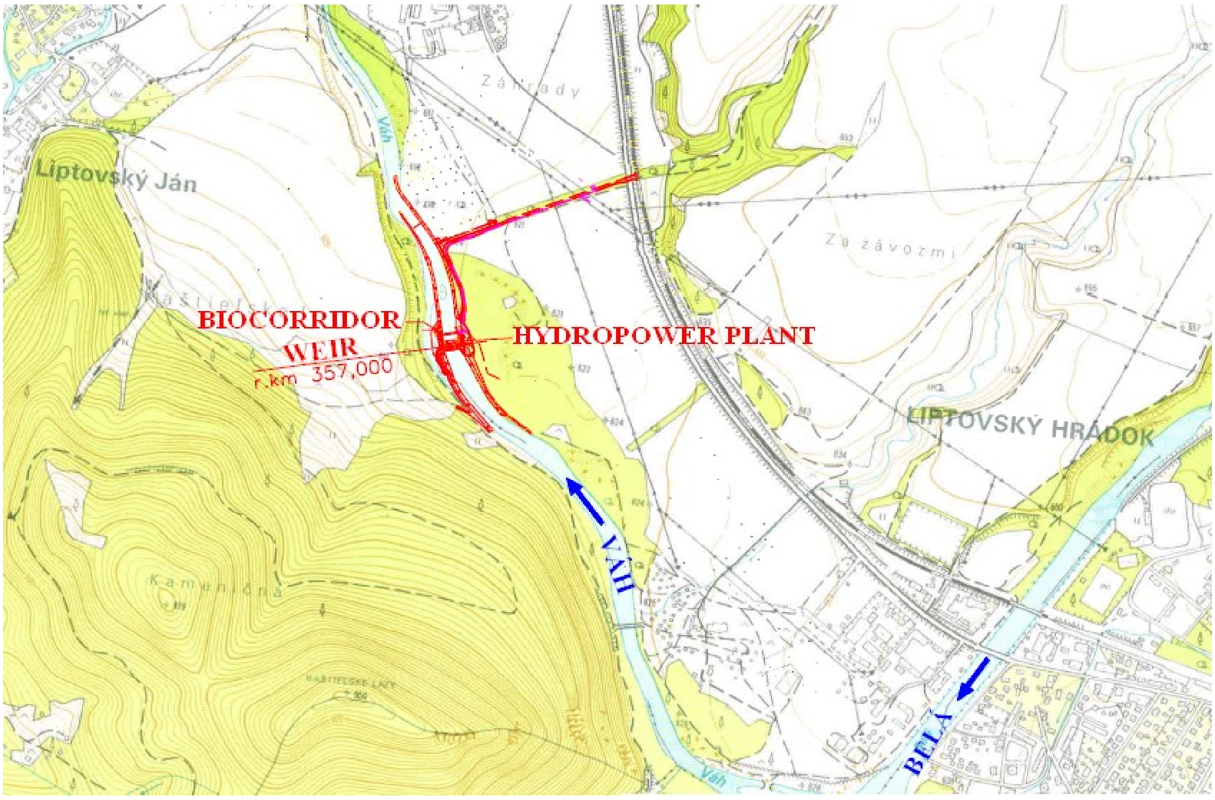

Fig. 1 Location of the SHPP on the Váh River

2 MATERIAL AND METHODS

Biocorridor serves for the passing of the obstruction created by the weir on the river for the ichthyofauna in both directions. It is located on the left side of the weir. It is not possible to design biocorridor by the power plant because of the planned shifting of tracks and bridge above the Váh river (protective zone of the railways). On the upstream side biocorridor will join the reservoir above the weir, on the downstream side biocorridor will join water level under the weir wing in the outflow area. According to the done ichthyological research (Příhoda: Ichthyological research and expert's statement of the SHPP Liptovský Ján influence on the ichthyofauna in the solved area), was found out very low restocking, especially low number of typical species for this river section: trout, grayling. The fish pass is designed to the parameters of the grayling zone. Recommended values are as follows [1]:

- max. velocity 2,0 m.s^{-1},
- min. depth under and above obstruction (barrier) 0,40/0,50 m,
- min. width in the water level 2 m,
- pool length 2,0 - 4,0 m,
- water level distance between pools 0,20 – 0,30 m,
- min. slot width in the barrier 0,30 m,
- min. discharge (approximate value) 0,40 m^3.s^{-1}.

Biocorridor proposed in the project had following parameters: max. water head 616,30 – 622,00 = 5,70 m, fish pass length cca 115 m, pool length is varying (min. 4,0 m), rest pools have length 8 – 10 m, river bed bottom is covered by the gravel and into the river bed are embedded boulders and stones. At the inflow is the inflow object, which is equipped by the floodgate for discharge regulating in the range 300 – 600 l.s^{-1}. Barriers are designed from wood, for the fish migration there are slots in barriers up to the river bed bottom – narrow trapezoid, bottom width 0,40 m, bank slope 4:1 (Cipoletti weir), in such slot suitable velocities can occur (1,5 – 1,8 m.s^{-1}). In front of the inflow floating barrage (beam) is placed, which prevents debris entering. Biocorridor cross section is trapezoid with width 2,0 – 2,5 m in the bottom, width in the water level more than 3,0 m. Banks above the water level will be plant by bushes, bottom and banks under the water level will be protect with the riprap (quarry fortification). In the operational manual of the SHPP will be determined, that during the migration period will be modified the inflow into the fish pass according to the fish species (trout - September/October, grayling – March/April) [1].

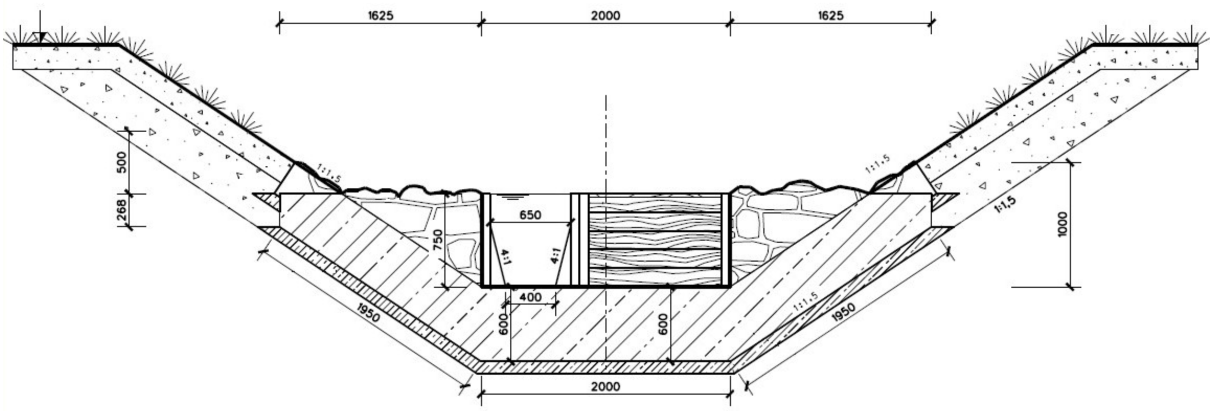

Fig. 2 The first design of the fish pass cross section according to the project [1]

Consequently the original design was modified, because the investor must solve a problem to make passable designed water structure for recreational and sport boats. He decided to build only one object (instead of fish pass and slide) with multipurpose functions – for fish migration and also for the boat transport. Mainly will be this object supported by discharge Q = 500 l.s^{-1}, during fish migration Q = 800 l.s^{-1} and by the boats passing (summer period) Q = 1500 l.s^{-1}.

Canoe slide enables weir overcoming in short time without boating stop. Favourable position of the slide is by the banks, in the case of the water structure with power plant the slide is better to situate in the opposite bank as the hydropower plant is placed [2], [3].

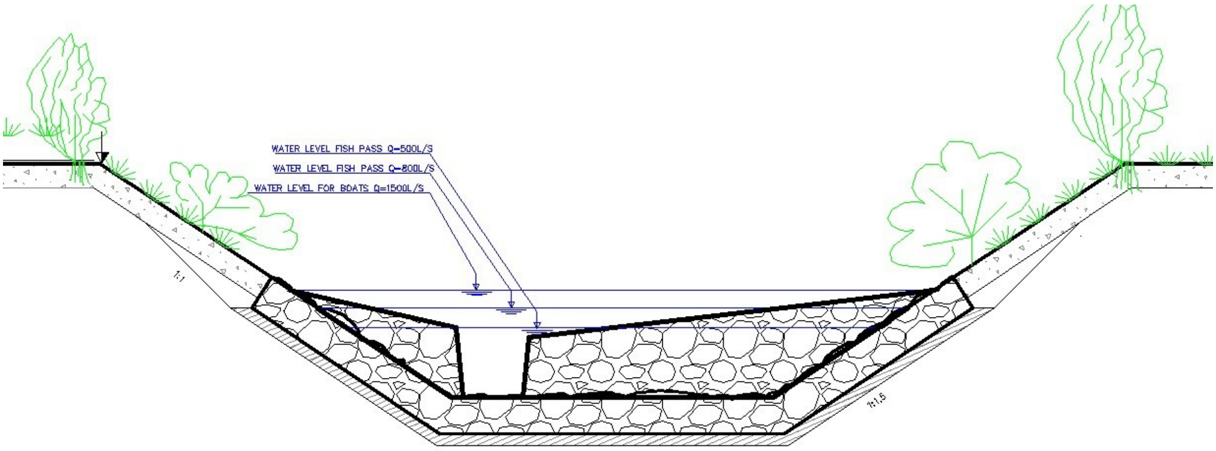

Fig. 3 Modified cross section of the fish pass according to the investor

Afterwards there were done modifications in 1D mathematical model up to the resultant variant, whereas some geometrical parameters of the fish pass river bed were changed (size and arrangement of the barriers (obstacles), dimensions of the pools and slots in barriers, the fish pass length, longitudinal river bed slope...).

The resultant variant has prismatic trapezoidal cross section, with width in the bottom b = 2,7 m, bank slope 1:1,5. Into the cross sections every 3 m were embedded barriers with slot up to the river bed bottom, slot width is 0,5 m and height 0,5 m. Chessboard arranged slots should create streamline meandering (their position was changed left and right). The height of the barrier near the bank is 0,8 m and fluently falls to 0,5 m near the slot. Into the fish pass river bed also 2 rest pools were included (approximately in 1/3 and 2/3 of the fish pass length), with parameters length 6 m and width 9 m.

Resultant mathematical model describes mentioned fish pass (multifunctional object) by 131 cross sections and 63 barriers (obstructions), average longitudinal river bed slope is i_o = 2,88 % (≈ 1:35). Difference in elevation of the river bed at the beginning and at the end of the simulated section is 5,70 m. Total length of the mathematical model is 198 m.

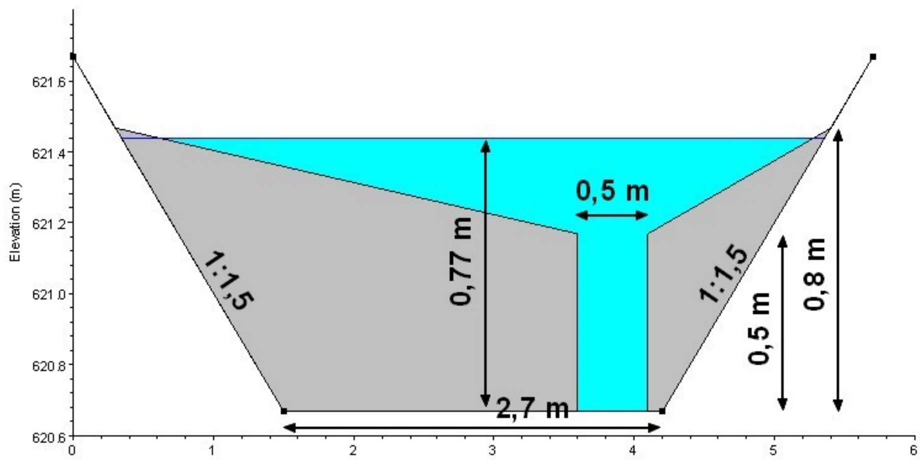

Fig. 4 Cross section of the resultant design of the multipurpose object

Except the simulations via 1D model HEC-RAS, resultant variant (just segment) was modelled also in 2D model (River2D), where is possible better illustrate streamline as well as velocity field, what is necessary not only for fish migration but also for boats passing.
The U. S. Army Corps of Engineers´ River Analysis System (HEC-RAS) is software that allows you to perform one-dimensional steady and unsteady flow river hydraulics calculations, sediment transport-mobile bed modelling and water temperature analysis. Steady Flow Water Surface component of the modelling system is intended for calculating water surface profiles for steady gradually varied flow. The steady flow component is capable of modelling subcritical, supercritical and mixed flow regime water surface profiles. The basic computational procedure is based on the solution of the one-dimensional energy equation. Energy losses are evaluated by friction (Manning´s equation) and contraction/expansion (coefficient multiplied by the change in velocity head). The momentum equation is utilized in situations where the water surface profile is rapidly varied. These situations include mixed flow regime calculations (i. e., hydraulic jump), hydraulics of bridges and evaluating profiles at river confluences (stream junctions). The effects of various obstructions (bridges, culverts, dams, weirs, etc.) may be considered in the computations [4].
River2D (Depth Averaged Model) solves the basic mass conservation equation and two (horizontal) components of momentum conservation. Outputs from the model are two (horizontal) velocity components and a depth at each point or node. Velocity distributions in the vertical are assumed to be uniform and pressure distributions are assumed to be hydrostatic. Important three-dimensional effects, such as secondary flows in curved channels, are not included. The River2D model is a two-dimensional, depth averaged hydrodynamic and fish habitat model developed specifically for use in natural streams and rivers. It is a Finite Element model, based on a conservative Petrov-Galerkin upwinding formulation. It features subcritical-supercritical and wet-dry area solution capabilities. Ice covers with variable thickness and discontinuous ice covers can be modelled. It is intended for use on natural streams and rivers and has special features for accommodating supercritical /subcritical flow transitions, ice covers, and variable wetted area. It is basically a transient model but provides for an accelerated convergence to steady-state conditions. The fish habitat module is based on the PHABSIM weighted usable area approach, adapted for a triangular irregular network geometrical description. The hydrodynamic component of the River2D model is based on the two-dimensional, depth averaged St. Venant Equations expressed in conservative form. These three equations represent the conservation of water mass and of the two components of the momentum vector. The dependent variables actually solved for are the depth and discharge intensities in the two respective coordinate directions [5].

3 RESULTS

From the simulations in 1D model HEC-RAS it was possible to analyze depths and velocities in the proposed multifunctional object on the Podtureň SHPP.

Tab. 1 Results of the resultant variant of the multipurpose object from HEC-RAS

discharge	depths	water level difference above and under barrier (obstruction)	$v_{dominant}$	v_{min}
($m^3.s^{-1}$)	(m)	(cm)	($m.s^{-1}$)	($m.s^{-1}$)
1,5	0,72 – 0,77	9	0,62 – 0,67	0,21

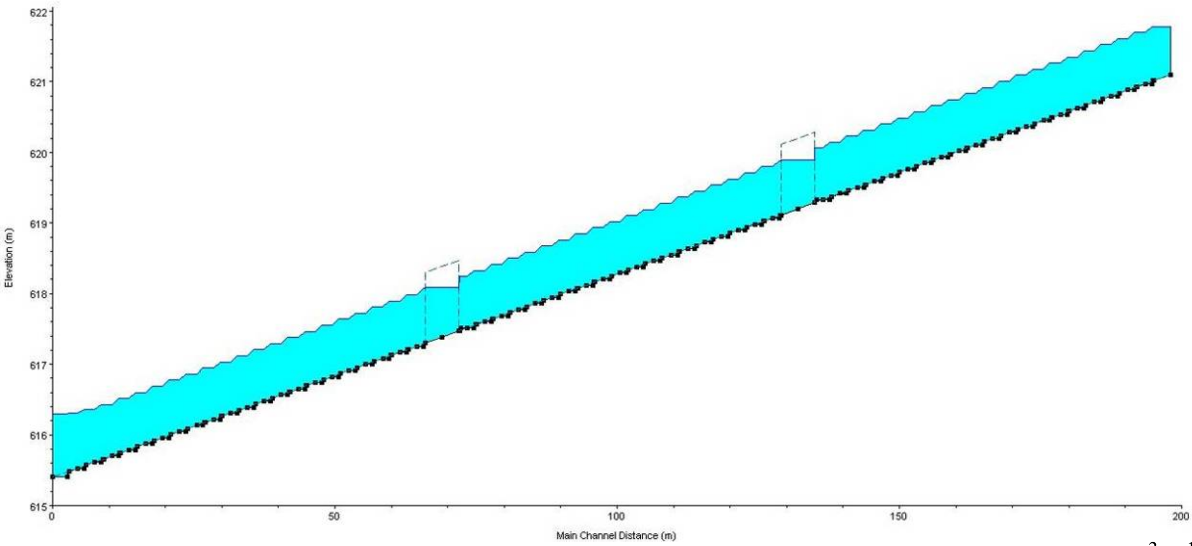

Fig. 5 River bed of the resultant variant simulated in the HEC-RAS model (for Q = 1,5 m^3.s^{-1}: depths 0,72 – 0,77 m, velocities 0,62 – 0,67 m.s^{-1})

Consequently was selected part of the object river bed with the rest pool and this segment was modelled also in the two dimensional model. This approach was chosen because of the repeating sections with barriers/slots (constant longitudinal slope, constant distance between barriers).

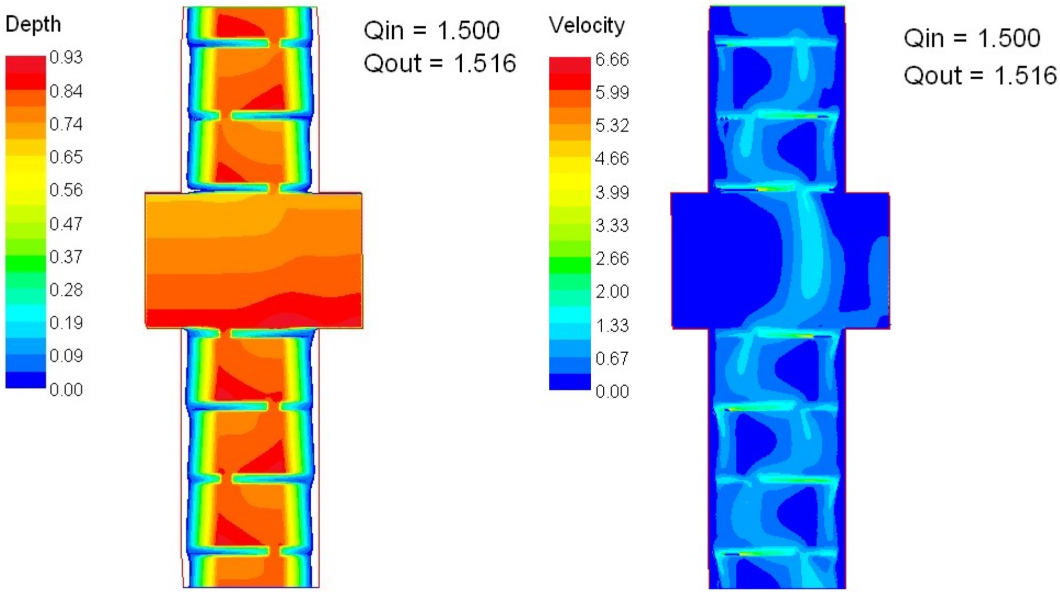

Fig. 6 Selected segment of the river bed with the rest pool and barriers under and above the pool in the River2D model (depths, velocities for Q = 1,5 m^3.s^{-1})

From the simulations in the 2D model concluded the same behaviour of the water level regime (backwater behind barriers) as well as of the velocity regime like in the 1D model. Reached values are bigger than in the 1D model. But above the barriers were small depths for boats passing (min. required depth is 0,5 m). The biggest values of the velocity were occurred on the barrier (in the slots velocities were still suitable for fish migration) and probably they were caused by the mesh insufficiency.

Afterwards I increased discharge to see if the parameters, especially depths above the barriers, will be appropriate for the boats transport.

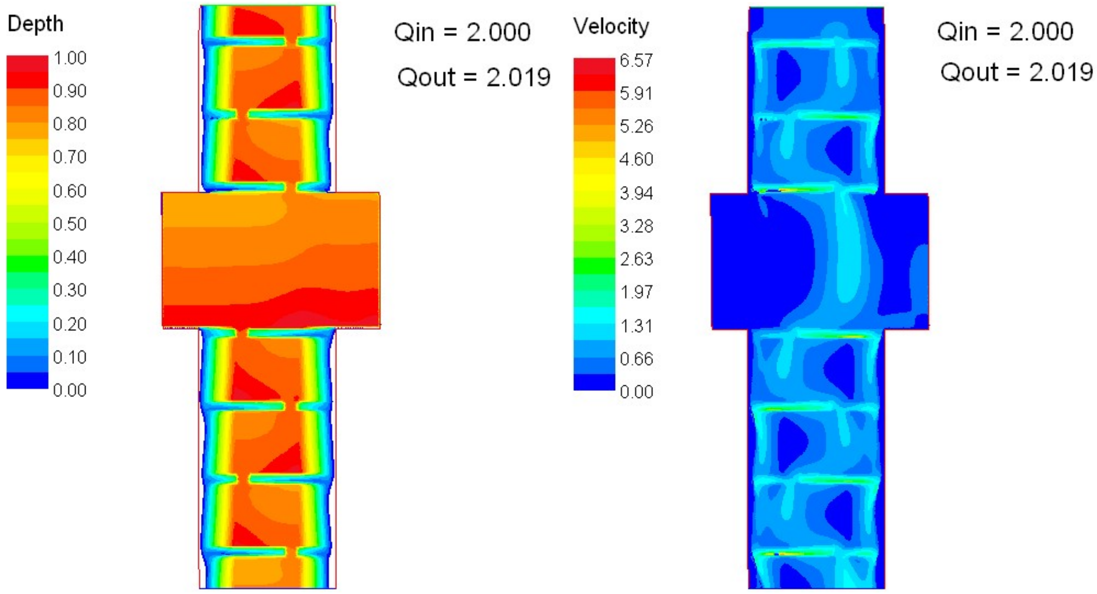

Fig. 7 Selected segment of the river bed with the rest pool and barriers under and above the pool in the River2D model (depths, velocities for Q = 2,0 $m^3.s^{-1}$)

Above the barriers there were achieved max. depths 30 cm and it is not enough for the boats transport. Additional discharge increasing is not economic because of the cost benefit analysis of the designed SHPP.

4 DISCUSSION

For the comparison possibility of the designed object with some recommendations, which were available for the slide and slot fish pass design, I mention hydraulic and geometrical characteristics of the slides for sport and recreational boats and fish passes for grayling zone.
Open slide has unregulated discharge, which minimum value is between 1,5 to 2,5 $m^3.s^{-1}$ according to the slide width. The slide cross section is rectangle, with constant width 2,3 m or 1,3 m. Slide has to be design to secure safety of the sportsmen, flow has to be calm and stabilized and construction of the slide must be simple and easy to build. The most important parameter by the slide design is its river bed slope. Recommended optimum slope is 6 % - 8 %. Side walls of the slide is suitable (from the psychological point of view) to design low because of the smooth passing with paddle across the boat. It is necessary to care about walls arrangement, they must be smooth and without pilasters. By the small depths in the river bed downstream weir it is possible to leave slide outflow nonextended. In the case of the insufficient depth it is possible to deepen river bed bottom under the slide. By bigger depths it is suitable to extend (bellmouth) the outflow from the slide, eventually to built horizontal plate under the downstream water level, which will secure in the outflow the same navigation depth like in the slide. Hydraulic jump must not occur in the outflow. Recommended depth is min. 0,5 m [2], [3], [6].
The British Canoe Union (BCU) presents following conditions for multipurpose object used by fish and canoe passage [7]:

- Are not less than 1,4 m wide

- Use minimum 20 mm thick baffles with fully rounded tops
- Carry a minimum head Ha ≥ 0,30 m
- Have rounded tops on side-walls
- The upstream ends of the side-wall(s) rake down into the headpond at ≤ 45° to be below water level at Q_{95}
- Grab chains are provided at the head of the pass
- Preferably consist of an odd number of juxtaposed baffle units, not less than three, so that the middle of the pass over the V of the baffle acts as a higher velocity 'lead'

Design parameters of the fish pass depend on the occurred fish species and on the type of the construction. For the area of interest was done ichthyological research from which resulted that grayling zone is there. Because of the dual-purpose of the planned structure, it was chosen technical type of the fish pass, so called slot.

Tab. 2 Recommended design parameters for slot fish passes [8]

Parameter	Limit values
Longitudinal slope i_o (%)	5 – 8
Water level difference (m)	0,1 – 0,15
Depth (m)	0,5 – 0,8
Pool Length (between barriers) (m)	1,9
Cross Section Width (m)	1,2
Slot Width (m)	0,15 – 0,2
Middle Velocity ($m.s^{-1}$)	0,5
Velocity at the Outflow ($m.s^{-1}$)	0,4
Discharge ($m^3.s^{-1}$)	0,14 – 0,16

Tab. 3 Recommendations for the slot fish passes in the trout and grayling zone [9]

Fish species	Min. water depth under sill h_u (m)	Min. slot high $t_{s,min}$(m)	Min. clear length L (m)	Min. clear width b (m)	Min. slot width s (m)	Min. discharge in fish pass Q ($m^3.s^{-1}$)
grayling, bream, chub (dace)	0,45	0,2	2,0	1,4	0,17 – 0,30	0,15 – 0,25
barbel, bream, pike-perch, pike, danubian salmon	0,5	0,3	2,8 – 4,0	1,8 – 3,0	0,30 – 0,60	0,40 – 1,00

Stream zone	Max. water level difference Δh_{max} (m)	Range of velocities v ($m.s^{-1}$)
Hypo-Rhithral	0,15	0,3 – 0,9

Epi-Potamal	0,13	0,3 – 0,8

It can be summarized that proposed object is suitable for the fish migration (geometrical as well as hydraulic parameters fulfilled the recommendations), but for the boats transport there are some conflicts, e. g. cross section type (rectangle should have been, but trapezoidal cross section is designed), longitudinal slope, rest pools, big nonflexible barriers, rough river bed (because of benthos), routing (slide – straight, fish pass – straight as well as sinuous), etc. And it was not considered inflow and outflow areas, e. g. for fish pass is better to have controlled inflow and some device to prevent debris entering, but in the case of slide is mostly used uncontrolled inflow without any obstacle; in the outflow area there is designed additional flow to attract fish in the case of fish pass construction, for slide such device can be confusing.

5 CONCLUSION

The needs of canoeists and fish may not coincide too well e.g. conflict between a location favourable for porterage and one that is easily found by the fish, attraction exit jet velocity of a joint canoe & fish pass is low. Accommodating canoes in technical fishways other than the specific joint facilities, especially in bottom-baffle passes, may entail compromising their efficiency for fish passage. There may be conflicts temporally between the use for canoes and the peak times for fish passage (often dawn and dusk). Such caveats have meant that it has not been a common practice to try to accommodate both fish and canoes. While it may therefore remain more appropriate to construct separate facilities for fish passage and canoeists in many instances, each project will have its own merits and careful consideration should be given to providing joint facilities where possible.. [7].

Fish passes can be a significant attraction to canoeists and indeed in some circumstances may be specifically designed (Bristle Ramp Canoe/Fish pass) or else modified for conjunctive use. However, there is considerable scope for a conflict between the two uses. The hydraulic characteristics are intended to be the most ideal and efficient for the passage of the fish. This means choosing a pass that is the most efficient at dissipating energy and lowering water velocity, and frequently also one that operates over the widest range in headwater level. In turn this leads to structures that use fairly aggressive means to accomplish this, with relatively narrow gaps, small head differences across them, or relatively thin and sharp baffles [7].

Normally, substantial efforts are made to avoid blockage of the pass by trash since clearly this would compromise pass efficiency, if not prevent fish migration altogether. Accumulations of trash also represent a potentially very expensive maintenance problem. Fish pass facilities are therefore usually provided with a form of protection that precludes the entry of canoes or kayaks [7].

While some types of pass, like very large pool and traverse passes or chevron bottom-baffle passes, may be suitable for canoes or similar small boats, and some types adapted eg. Larinier, the choice of one type over another may well compromise the efficiency of the structure as a fish pass (higher water velocity, smaller operating range with head). On an individual site basis it may or may not have significant consequences, however, if compounded by the use of a number of similar facilities along the river continuum then for migratory fish (especially diadromous species) the cumulative effect would almost certainly be significant [7].

Fish may use fish passes at different times of the day and night depending on flow, season, local conditions and the nature of the passage facility. Migrating fish often exhibit crepuscular activity during low flows (i.e. active at dusk and dawn). In general, activity patterns are not

very predictable and may be very specific to a particular site. There is clearly a potential conflict for the disturbance of migrating fish if any one facility was in constant or very frequent use by boats at a time when fish wanted to migrate [7].
For the reasons given above a facility designed for conjunctive use is not normally recommended, and it is best to provide separate facilities. While conjunctive use is not discounted altogether, a careful assessment needs to be made on an individual site basis and quite possibly in a catchment context. There are likely to be more opportunities for conjunctive use where `natural ` by-pass channels can be employed [7].

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HYDRAULIC ASPECTS OF THE VEGETATION GROWTH IN THE CANALS

T. Dadić[1] and L. Tadić[2]

Abstract

Surface drainage, as an engineering system, must sustain designed dimensions during long time period and fulfill its function. It is based on hydrologic and hydraulic calculation and on the hydropedological characteristics of agricultural land that is being drained. Sustaining of the designed dimensions of the canal and functioning of the system at different flow rates require regular maintenance. Effect of irregular maintenance of canals is overgrowing which reduces the flow rate. The aim of this paper is to show the impact of vegetation in drainage and irrigation canals through various quantitative and qualitative, both positive and negative aspects. Emphasis is placed on the complexity of the flow calculations, taking into account any grassy or woody vegetation.

Keywords

Canals, flow rate, maintenance, roughness, surface drainage system, vegetation growth.

1 INTRODUCTION

Drainage canals can easily become overgrown with vegetation due to lack of maintenance. Result is reduction of flow rate so floods can occur. Vegetation is main obstruction, but it has some positive effect like erosion reduction or improving quality of water which can be significant because lot of fertilizers and plant protection agents from the fields come with water into canals [1]. Never the less, hydraulic aspect of vegetation in canals must be determined.

For determination of hydraulic roughness in open channels, four different methods, which are based on four different roughness coefficients, are used [2]: Chézy C ($m^{1/2}/s$), Darcy-Weisbach λ (-), Manning n ($m^{1/3}/s$) i Strickler K_{st} ($s/m^{1/2}$) and Nikuradse k_N (m). These coefficients can be associated with equation [2]:

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$$K_{st} = \frac{1}{n} = \sqrt{\frac{8g}{\lambda R^{1/3}}} = \frac{C}{R^{1/6}} = \frac{18}{R^{1/6}} \log(\frac{12R}{k_N}) \quad (1)$$

where g is gravity acceleration [m/s^2]and R hydraulic radius [m]. Different methods are used in different parts of world. Although various advantages are listed in the literature, they are not that significant because all coefficients can be correlated with Eq.(1) and thus comparable [2, 3].

Mentioned coefficients are related solely to the frictional resistance of the prism bed with flat bottom. Flow resistance is influenced by numerous other factors such as nonprismaticall and meandering riverbed and vegetation. The above mentioned coefficients can capture these effects, but then they get a broader meaning and become resistance coefficients of the observed section. Roughness coefficient is a parameter of a profile and the resistance coefficient is a parameter of a section [4].

2 DIFFERENT APPROACHES OF THE VEGETATION RESISTANCE

Vegetation in the surface drainage canals can be flexible, as it is grassy vegetation, or solid, such are bushes and trees. Vegetation parameters that most influence the change of velocity and resistance are its density and distribution, height, flexibility and immersion [3]. Calculations differ with regard to flexibility (grass or trees) and immersion (flooded and no flooded vegetation).

Analysis of the interaction between vegetation and water flow, as well as analysis of increased head losses may be based on the concept of continuity, energy and momentum conservation [5, 6].

The examples below show calculations for the grass and woody vegetation.

3 CALCULATION OF CANALS OVERGROWN WITH WOODY VEGETATION

It is known that trees and shrubs suppress the growth of other vegetation beneath the canopy. If there is woody vegetation on slopes of a drainage canals, it prevents with its shadow the development of grassy vegetation, which is particularly important because then hydrophytes do not develop at the bottom of the canals. For this reason, many scientists recommend afforestation slope canals [7]. In the surface drainage canals in the fields, backfilling of a bottom occurs 2.7 times slower if woody vegetation is located on the slopes of canals [7]. This is particularly important from the standpoint of maintenance of those canals.

Erosion of the canal slopes is prevented by penetrating of roots of the trees into soil. Because of that, slopes become reinforced and stabilized. In general, vegetation does, especially roots of woody vegetation, decrease erosion of canal slopes. But, there are some investigations which prove that decreasing of erosion is only local within a static vegetation patch. Dynamic vegetation patch obstruct the flow and cause flow concentration and erosion between those patches [8]. Humus, which is produced by leaves falling from the trees, also prevents erosion and act to increase the infiltration capacity of the soil. Infiltration capacity is affected by roots of the trees also. Trees keep canal slopes from strong runoff [7].

If there is no woody vegetation on the slopes, sediment must be removed once every 5-10 years. In this period, sediment layer can increase up to 0,2-0,3 m depending on soil type and longitudinal gradient [7]. Croatian regulations [9] stipulates, according to canal importance and catchment area, sediment removing every 3 to 4 years or when the sediment layer is

greater than 20 cm for small canals, and for bigger ones every four to five years or when the sediment layer is greater than 30 cm.
Hydraulic calculations of canals overgrown with woody vegetation are more complicated than the ordinary ones with trapezium form because influence of vegetation must be taken into consideration. Under the assumption that all tree stems are located in a row and all of them have same diameter d, hydraulic resistance force ΔT can be calculated as follows [7]:

$$\Delta T = \gamma C_f K_i d \frac{v^2}{2g} \Delta H \tag{2}$$

where γ-specific gravity of water [N/m^3], v-average flow velocity [m/s], C_f-coefficient estimating the shape of stems [-], K_i-multiplier estimating stems density [-], ΔH-stem height [m].
Average velocity in overgown canals can be calculated [7]:

$$v = \sqrt{\frac{2gi}{\left(\frac{1,15\xi}{l}\right) + \left(\frac{\lambda}{4R_{tr}}\right)}} \tag{3}$$

where i-hydraulic gradient [-], ξ-hydraulic resistance of stem [-], l-distance beetwen stems [m] λ-hydraulic friction coefficient (0,10-0,15), R_{tr}-hydraulic radius [m].
In parts which are overgrown with trees, flow velocity decreases and flow velocity gradient occur at the contact with non-overgown parts. Because of that gradient and the stop power which is produced by shear stress, velocity is increased near the borders of overgrown strips so Eq.(3) can be used to calculate velocity in the middle of wide overgrown strip. Velocity in actually overgrown stripcan be expressed from the balance equation of the forces: resistance of tree stems, friction on the slope, shear stress and the horizontal component of the gravity power causing water movement [7]. Expressions for velocities in overgrown and non-overgrown parts (Fig.1) can be obtained by balancing those forces.

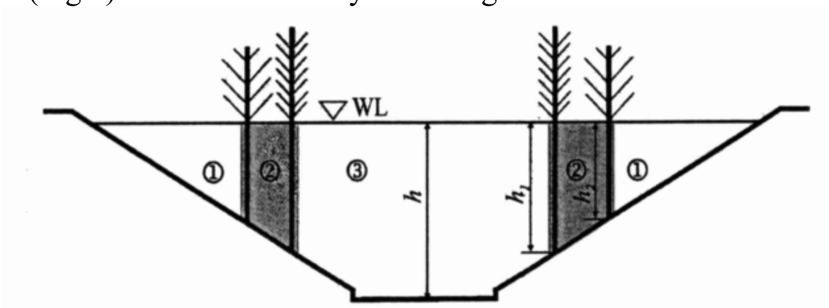

Fig. 1 Different flow strips occurring in canal cross-section:1-upper non-overgrown strip, 2-overgrown strip, 3-central non-overgrown strip [7]

4 LINE RESISTANCES DUE TO GRASSY VEGETATION

4.1 Iterative method

The influence of vegetation on the line resistance is very important when flow in inundations is considered in a period of high water, or when the flow capacity of vegetated drainage canals is calculated.
Manning's roughness coefficient can be determined for relatively small canals using Fig. 2. The degree of the flow retardation (A-E) is empirically formulated depending on the height

and density of the grass where A stands for very high degree, B-high, C-moderate, D-low and E-very low. As the values of Manning's roughness coefficient, velocity *v* and hydraulic radius *R* are inter-dependent, the calculation is performed by iterative procedure [4].

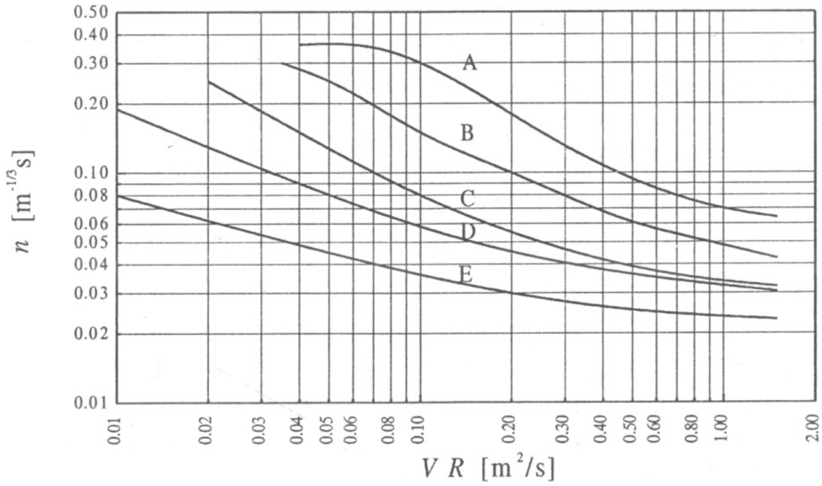

Fig. 2 Manning’s roughness coefficient depending on the degree of flow retardation [4]

4.2 Hydraulic equivalent thickness of grass layer

Another approach is based on roughness coefficient *n* which exists on the surface of grass layer and the bottom of the channel is raised to a certain thickness of a grass layer. This concept has been used when artificial bottom roughness was used in the stream modelling. Hydraulic calculations were more accurate when the water depth was assessed from the level of equivalent hydraulic bottom and obtained formulas were correct in both laboratory and natural conditions [10].

The equivalent thickness of the grass layer was determined by measuring the depth of water from hydraulic bottom plane. This depth is calculated using Manning's formula based on actual roughness coefficient *n* of grass and conditional roughness coefficient n_1 obtained by measuring the depth of water from the surface of the grassy area.

Using this approach the thickness of laid or gently inclining grass can be determined. The degree of inclination of the grass and the water depth depends on the water discharge. The higher the discharge, the lower is the equivalent thickness of the grass. At low discharge rates, the grass is lying less than at higher ones and then roughness coefficient is higher. More details about this method can be found in literature [10].

4.3 Friction coefficient for flooded vegetation

Modern computational models, which take into account vegetation influence, are based on defining the relative roughness in the function of the biomechanical properties of the vegetation cover, which will be briefly described below.

The analysis shows that the tangential stress at the bottom of streams with a depth that is significantly greater than the height of the vegetation cover mainly depends on three dimensionless numbers [4]:

$$C\tau = f_3\left\{\frac{h}{\left(MEI/\tau_0\right)^{1/4}}, \frac{k_v}{h_v}, \frac{k_v}{h}\right\} \tag{4}$$

where C_τ-coefficient of shear stress [-], k_v-absolute roughness of grass [m], h_v-average height of the grass in a vertical position [m], h-normal depth of the water [m], *MEI*-vegetation stiffness [Nm^2], τ_0- shear stress at the bottom [Pa].

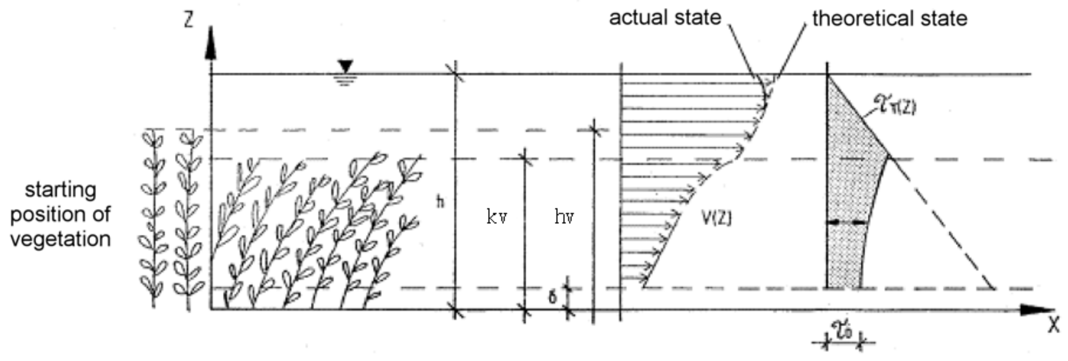

Fig. 3 Parameters for flooded vegetation [11]

Vegetation stiffness MEI depends on the type of the grass [4, 11]:

$$MEI = \begin{Bmatrix} grass : 319 h_v^{3,3} \\ dead \quad grass : 25,4 h_v^{2,26} \end{Bmatrix} \tag{5}$$

It is evident that the global stiffness increases with the height of the grass, although the rigidity of individual plants decreases. The reason for this is that the bending of leaves increases the density of vegetation cover.

The vegetation height under flow is approximated by the formula using the vegetation stiffness MEI [4, 11]:

$$k_v = 0,14 \mathrm{h_v} \left[\frac{\left(\mathrm{MEI}/\tau_0\right)^{0,25}}{\mathrm{h_v}} \right]^{1,59} \tag{6}$$

Darcy-Weisbach friction coefficient can be expressed in relation to vegetation height under flow k_v, vegetation height in starting position h_v, flow depth h and the vegetation stiffness *MEI* of the vegetation elements [4, 11]:

$$\frac{1}{\sqrt{\lambda}} = \mathrm{A_v} + \mathrm{B_v} \log(\mathrm{h/k_v}) \tag{7}$$

The parameters A_v and B_v vary with the bending of the vegetation, which is expressed by the relationship of the critical shear stress velocity v_c to the actual shear stress velocity v [4, 11]:

$$\mathrm{v_c} = \min \begin{Bmatrix} 0{,}028 + 6{,}33\mathrm{MEI} \\ 0{,}23\mathrm{MEI}^{0,106} \end{Bmatrix} \tag{8}$$

$$\mathrm{v} = \sqrt{ghI} \tag{9}$$

where *I*-longitudinal slope. In Eq. (8) first term refers to a low grass that returns entirely in the upright position (elastic deformation), and the other on the vegetation with a long and solid leaves, which will not be returned to its original upright position (residual, plastic deformation) [4]. The parameters A_v and B_v are listed in the Tab. 1.

Tab. 1 Value of parameters A_v and B_v [4, 11]

Leaves position	Bending parameter	Parameters	
		A_v	B_v
upright	$u/u_c \leq 1{,}0$	0,15	1,85
bended	$1{,}0 < u/u_c \leq 1{,}5$	0,20	2,70
bended	$1{,}5 < u/u_c \leq 2{,}5$	0,28	3,08
bended	$2{,}5 < u/u_c$	0,29	3,50

5 CASE STUDY: MANNING'S ROUGHNESS COEFFICIENT FOR VUKA RIVER

Calculation procedure described in Chapter 4.3. is applied on Vuka River from 34+442 rkm to 28+929 rkm (Fig.4). Even though it is not a drainage canal, cross section of its main riverbed is adequate dimensions.

In its lower parts it is typical lowland river with very small slope (less than 0,2‰), small velocities and intensive meandering. River is recipient for many artificial surface drainage canals. In the past, Vuka River very often flooded surrounding agricultural fields. Longitudinal (Fig.5) and cross section (Fig.6) geometry data of river bed were taken from [12].

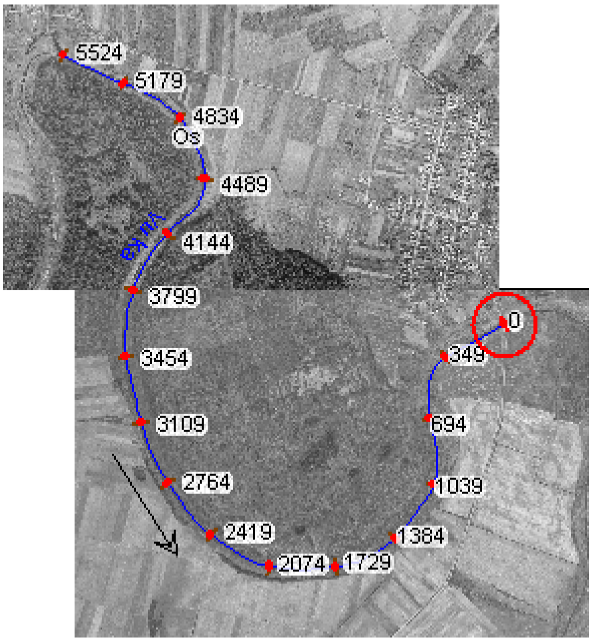

Fig. 4 Vuka River from 34+442 rkm to 28+929 rkm

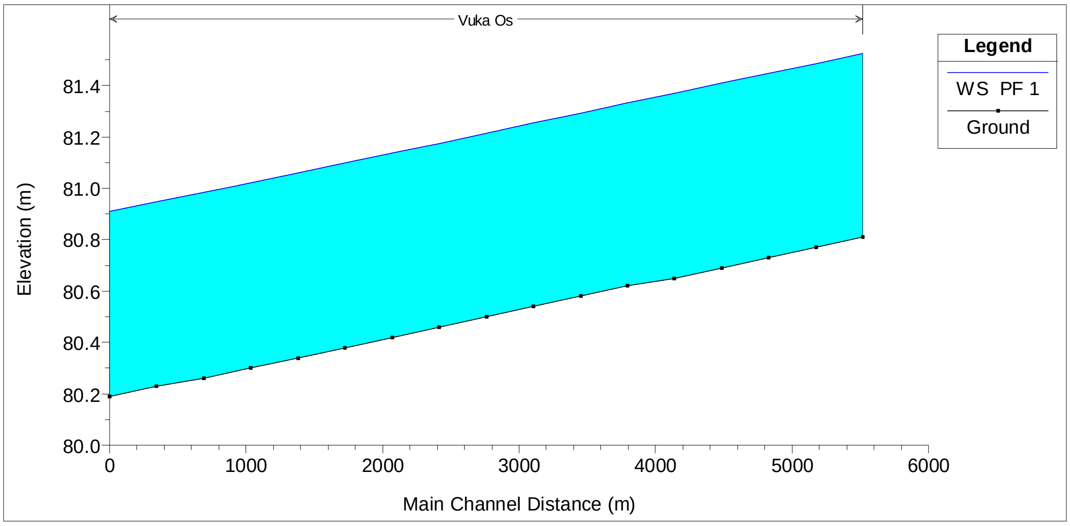

Fig. 5 Longitudinal profile of Vuka River from 34+442 rkm to 28+929 rkm

Eq. (4)-(9) were used to calculate Manning's roughness coefficient for Vuka River at the 28+929 rkm. This station was used because flow rates and water levels were measured there in a period from year 2005 till year 2011. Only medium discharges or flow rates which remained in the main riverbed were used because calculation is intended for trapezoidal formed cross sections. Cross section of Vuka River at the 28+929 rkm is shown in Fig.6.

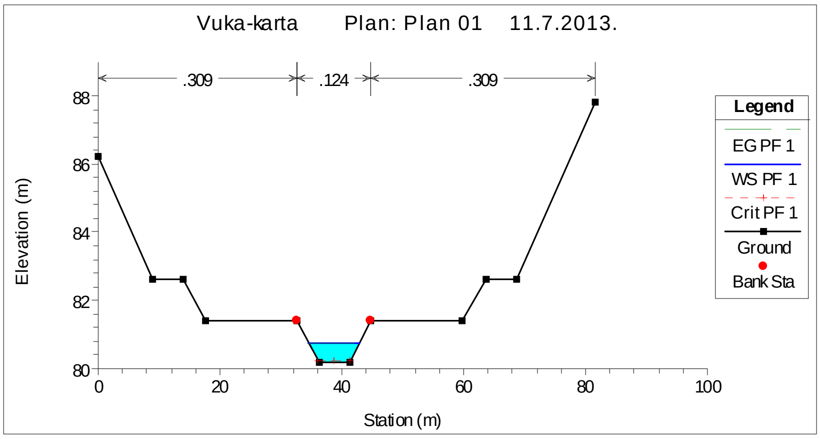

Fig. 6 Cross section of Vuka River at the 28+929 rkm

For known flow rate and water level, height of the grass was assumed. Based on the water level, geometry data of cross section, and assumed grass height, Manning's roughness coefficient was calculated. Using HEC-RAS software, new water level was obtained which was compared with measured one. The procedure was repeated until measured and obtained water level didn't meet. Results of calculation are shown in Tab 2.

Tab. 2 Obtained results

Date	Flow rate [m^3/s]	Measured water level [ma.s.l.]	h [m]	h_v [m]	v_c [m/s]	τ_0 [Pa]	k_v	λ	n	Obtained water level [ma.s.l.]
8.2006.	0,26	**80,89**	0,71	0,2	0,220	0,597	0,379	2,330	0,156	80,96
				0,13	0,199	0,597	0,332	1,722	0,133	80,90
				0,12	0,195	0,597	0,323	1,639	0,130	**80,89**
9.2006.	0,22	**80,88**	0,7	0,12	0,195	0,590	0,325	1,702	0,132	80,83
				0,15	0,205	0,590	0,358	1,977	0,143	**80,86**
10.2006.	0,16	**80,83**	0,65	0,12	0,195	0,554	0,333	2,116	0,146	80,76
				0,185	0,216	0,555	0,380	2,971	0,173	**80,82**
9.2007.	0,171	**80,92**	0,74	0,15	0,206	0,617	0,342	1,684	0,133	80,75
				0,30	0,243	0,617	0,423	2,788	0,171	80,84
				0,47	0,270	0,617	0,486	4,205	0,210	**80,91**
7.2008.	0,18	**80,71**	0,53	0,12	0,195	0,468	0,356	4,545	0,208	80,93
				0,02	0,051	0,468	0,205	1,120	0,107	**80,70**
12.2008.	0,19	**80,77**	0,59	0,05	0,158	0,511	0,262	1,559	0,124	**80,76**
8.2009.	0,075	**80,62**	0,44	0,06	0,165	0,400	0,306	5,142	0,216	80,66
				0,055	0,161	0,400	0,298	4,674	0,205	**80,64**

Obtained Manning's roughness coefficients for this cross section are significantly higher than the ones recommended in literature. In most tables where Manning's roughness coefficients are listed, the higher value is 0,15 which refers to very weedy state, deep pools or floodway with heavy stand timber and brush [13]. In some tables [14], value 0,04 is for canals in very bad shape with increased amount of rocks and sewed. In this study, high values of coefficient were needed in order to calibrate water levels. Another reasons is very small longitudinal slope (0,11‰) and meandering river section. The highest value obtained Manning's roughness coefficients is 0,210. It refers to vegetation height of 0,47 m and water depth of 0,74 m.

Fortunately, periods of high discharge rates occur during winter and spring, when vegetation is not so developed. Otherwise, flow resistance would be much greater than in usual hydraulic calculation procedures and could cause flooding.

6 CONCLUSION

Irregular maintenance of a canals leads to increased vegetation growth. Result is reduction of flow rate and change of designed cross-section dimensions. However, vegetation has several positive impacts such as stabilisation of canal slopes and improving water quality.

Impact of vegetation in drainage canals on flow rate must not be neglected. There are several types of calculations which refer to different types of vegetation and conditions of immersion. By appliance one of this calculation and taking into account measured data from field, obtained results may differ from theoretical ones. Especially when calibrating.

Further investigations are needed in order to match theoretical values of roughness coefficient with the ones which take into account real measured data.

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THE SMALL HYDRO POWER PLANTS IN SLOVAKIA - THE PREPARATION OF THE PROJECTS

Prof. Ing. Peter Dušička, PhD.[1], Ing. Peter Šulek, PhD.[2], Ing. Ján Rumann, PhD.[3]

Abstract

The concept of using hydropower potential of rivers in Slovakia until 2030 was approved by the Slovak government in 2011. This concept should influence the further development of hydropower in Slovakia. This paper describes the experience of the Department of Hydraulic Engineering, which participated in the preparation of specific projects of small hydro power plants in fields of technical preparations, elaboration of project documentations, environmental impacts assessment processes for small hydro power plants and permits processing.

Keywords

Small Hydro Power Plant, Hydro Potential, Renewable Energy

1 INTRODUCTION

In the Slovak Republic (SR), the hydropower is the most used of all renewable energy sources (RES). Its use contributes also to reducing the greenhouse gas emissions and reducing the dependence on the import of fossil fuels, it contributes to the diversification of energy sources and it is consistent with requirements of environmental acceptability and principles of sustainable development.

The European Union (EU) requires the increase of the share of RES in the final energy production. Based on the conclusions of the EU Council from March 2007, the EU 2020 targets set to achieve a reduction in greenhouse gas emissions by 20% and the share of RES in final energy production by 20%. These objectives have been set for the entire EU.

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As a part of the Climate and Energy Package (January 2008), a directive of the European Parliament and of the Council 2009/28/EC on the support of energy from RES was adopted. A national target for SR results from the directive to achieve the share of RES in final energy production of 14% by 2020. (For example, in 2005 this share was 6.7%)
The main objectives and priorities of the energy policy and energy security of SR are characterized by the Energy Policy of SR, which defines the basic objectives and the framework for the energy development in a long term.
The Government Resolution no. 732 of 15 October 2008 approved the Energy Security Strategy of the Slovak Republic. For RES it indicates the following estimated increase of the share of electricity produced in RES in total electricity consumption (not including large hydropower plants)

- from 1% in 2005 to 7% in 2015,
- to 9% in 2020
- to 11% by 2030.

Further it is stated that at present the potential suitable for small hydropower plants (SHPP) is used only to 25%.

2 THE CONCEPT OF HYDROPOWER POTETIAL UTILIZATION IN SR UNTIL 2030

To support the increase of the share of small hydropower plants in electric energy production, a Concept of hydropower potential utilization of rivers in SR until 2030 (CoHEP) was in the field of hydropower elaborated and consequently approved by the Government Resolution of SR no. 178 of 9 March 2011.
The CoHEP should have mapped the current state of utilization of the hydropower potential of rivers in SR, it should have detected potential and environmentally acceptable possibilities of its further utilization and is should have created a basic material for its further development. Given that the sites suitable for the locations of large hydro hydropower plants have been already hydro-energetically utilized, respectively the sites are already planned for such utilization; the CoHEP does not solve their further development and is focused only on SHPP.
As a technical basis for the mapping of the hydropower potential of rivers in SR in the CoHEP (the current status of its utilization and the perspective possibilities of its further utilization) a database of the Water Research Institute in Bratislava (WRI) was used. The complete database of technical hydropower potential (HEP) of the rivers in SR includes:

- A total of 625 profiles,
- of which:
- 227 profiles represent used HEP,
- 398 profiles represent unused HEP,
- 24 profiles represent functional large hydropower plants,
- 203 profiles represent build functional SHPP,
- 26 profiles represent build but not functional SHPP,
- 4 profiles are reserved for the construction of large hydropower plants and
- 368 profiles are technically usable for the construction of small hydpropower plants.

The total amount of the HEP 6 700 GWh/year, while the used HEP is 4 732 GWh/year (70.6 %), the unused HEP is 1 968 GWh/year (29.4 %).
Large hydropower plants are currently producing 4,448 GWh of electric energy per year (66.4% of the HEP), the SHPP produce 284 GWh of electric energy per year (4.2% of the

HEP), built SHPP that are not functional represent an energy potential of 12 GWh/year (0.2% of the HEP), planned large hydropower plants produce 1,159 GWh/year (17.3% of the HEP). For the HEP utilization in SHPP remains 797 GWh/year (11.9% of the total HEP).

According to the CoHEP is the distribution of the HEP according to river basins in SR as follows:

Hydrologic basin	**HEP [GWh/year]**
Moravia	29
Danube	2 511
Váh and Small Danube	2 985
Nitra	72
Hron	427
Ipeľ	34
Slaná	96
Bodva	3
Hornád	262
Bodrog	138
Poprad a Dunajec	143
Together in SR	**6 700**

For potential investors of SHPP were the "practical results" of the CoHEP the tables with profiles of SHPP according to rivers in Slovakia, where the basic parameters of the SHPP in each profile are defined: the river kilometer, installed power capacity and average annual production of electric power. These profiles were then assigned to particular candidates and the rivers management, the Slovak Water Management Enterprise (SWM), should have concluded a contract with the candidates of the future lease of a part of a river bed and the use of HEP.
A great interest had the profiles on the Hron River, which has after the Danube and Váh Rivers the third largest HEP, but compared to these two rivers it is minimally energetically utilized.
The following describes the experience of the employees of the Department of Hydraulic Engineering with the preparation and planning of SHPP in the middle section of the Hron River between Banská Bystrica and Hliník nad Hronom.

3 THE EXPERIENCE OF SHPP PROJECT PREPARATIONS

Preparation of the projects of SHPP in the concerned stage of the Hron River has been elaborated for investors to who the profiles of SHPP were allocated according to the database of CoHEP. The preparation started in years 2009–2010. Presently these projects are at the stage of environmental impact assessment (EIA), or territorial proceedings.
Although the basic technical parameters of the SHPP were quantified in the CoHEP, we advised the investors to develop a technical study of the particular SHPP (preferably with variants of technical solution) based on following information:

- a current geodetic survey of the riverbed and adjacent area,
- current hydrologic data and
- an overall disposition of the territory including infrastructure.

The technical study included mainly:

1. Calculations on an 1-dimensional mathematical model of the riverbed and adjacent section of the river for determination of:
   - current river bed capacity,
   - water level regime during flood discharges,
   - backwater reach upstream the SHPP,
   - heads at the SHPP.
2. Hydraulic calculations of particular structures at the SHPP (design of dimensions and capacities of a weir, design of a stilling basin, hydraulic calculations of a fishpass, etc.).
3. Layout and technical design of SHPP structures.

Technical studies were recommended to be discussed with the river management - SWM in Banská Bystrica, which is responsible for the management of the Hron River. Only after the commenting of the studies we recommended to start the planning of the project for the territorial processing.

The technical studies thus served for approving and making technical designs of the SHPP accurate by the river management and also served as a basis for assessment processes of the EIA. The technical studies were elaborated in the form of a project for territorial processing to make it easy to develop this project for territorial processing, after implementing the comments.

Experiences form this phase are following:

- The discharge capacity of the riverbed was mostly sufficient for 2 – 5 year discharge. It is obvious that higher discharges currently flood the adjacent area (usually inundation). We mention this fact, because in further permitting processing in several cases the participants of the permitting processes required along with the construction of the SHPP also a full flood protection of concerned areas against 100 year discharge. In the case of externals of villages, where inundation areas are intended for flooding, such a requirement is economically and technically impossible and liquidating for the construction of particular SHPP
- In the comments to the technical design the river management recommended maximal cleaning of sediments in the riverbed and was strongly against massive digging in riverbed downstream the SHPP and the consequent disruption of fortification of the riverbed and banks.
- Practically in all cases there was a huge difference between the average annual power generation stated in the CoHEP (about 9~10 GWh) and average annual power generation given by the calculations based on the current survey of particular locality, current hydrological data and recommendations of the river management to the SHPP design (about 5~6 GWh).

The next phase was the EIA assessment process. The main experiences form this process are as follows:

- The State Nature Protection (SNP), Slovak Fishing Association (SFA) and others mostly civil associations have always dissenting opinions.
- In the case of the Hron River mostly the SNP and SFA enforce tolerable spacing of SHPP in the range of 20-30km. Given the existing SHPP and already permitted ones, the fulfilling of this requirement practically means the end of construction of SHPP or in other cases this requirement cannot be fulfilled.
- The SFA representatives often declared the opinion that all fishpasses are dysfunctional. Doing so they did not present any example of how an optimal design

of such a structure should look like. Unfortunately in the SR there is not any systematic research in this field, while the corresponding knowledge from abroad is usually rejected by SFA representatives as unsuitable for Slovak rivers.

- The real time lengths and processes of the EIA assessment show that time period for achieving at least the territorial permissions (usually 2 years), which was the condition of the SWM for valid contracts for future lease of a part of the riverbed and utilization of the HEP. Many applicants for the SHPP construction are in stage where after 2-3 years of investments mostly in engineering, planning and EIA documentation they only have got an unfinished EIA assessment process with the perspective of non-continuation or not signing the contract for future lease of a part of the riverbed and utilization of the HEP.
- An interesting finding from public hearings on the EIA is also the fact that practically no one (not even the state administration authorities) was interested in the technical design of the SHPP and their future operation, even though this determines the impacts on the environment. The SHPP have been principally seen as constructions destroying the environment. Program listings or program commands in the text are normally set in typewriter font, e.g., CMTT10 or Courier.

4 CONCLUSION

The situation around the permitting process for SHPP has proved to be extremely difficult and almost impassable for investors. On the one hand, the state declared an interest in using the hydropower potential. On the other hand, the state administration authorities, especially at the regional level, when it comes to a specific project in their fields of work are against it. And that's not even mentioned the different civil associations.

Other serious facts can yet be added to the previously mentioned experience in permitting procedures, as follows:

- If municipalities do have prepared and approved land use-planning documentation, this documentation does not include the utilization of the HEP and the SHPP construction. Changing such documentation is becoming "a tool" to obtain various "benefits" from the investors for the municipalities.
- The ownership of the land on river banks – not all investors, who have got an assigned SHPP profile, have "under control" the land on the concerned river banks, which are necessary for the construction of SHPP. It appears that this aspect have not been taken into account in the allocation of profiles. In practice, this means that there are sites that are allocated to investors but other potential investors (who did not get the allocations) own the land, thus the construction of SHPP in such locations is so far in stalemate.

In conclusion we confine ourselves to stating that the CoHEP actually does not guarantee anything – not even the SHPP parameters. Any investor who got assigned a SHPP profile must on his own go through the permission process, including the EIA assessment. This also applies to the strategically important locations stated in the CoHEP.

Acknowledgements

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IMPACT OF THE SMALL HYDRO POWER PLANT IN ŠALKOVÁ ON THE GROUNDWATER LEVEL REGIME

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Abstract
In connection with the use of hydro potential a plan of the small hydro power plant (SHPP) Šalková on the Hron River was proposed. In the process of assessment of the anticipated effects on the SHPP is required to determine the impact of the proposed activity and the level of the groundwater adjacent mode. The aim of the work was a model for surface water and groundwater before and after implementation of the SHPP Šalková. This model will be used for the design and evaluation of possible negative impact of the proposed measures which would minimize the effect of SHPP construction and operation on groundwater level regime. HEC-RAS and TRIWACO modelling software were used for the simulation.

Keywords
groundwater level, HEC-RAS, hydro power plant, model, TRIWACO

1 INTRODUCTION

In connection with the use of hydropower potential a plan has been proposed to build a small hydro power plant in Banská Bystrica, part Šalková. Hydroelectric project is proposed in the original bed of the Hron River in rkm 180,7 in the village Šalková. In the process of assessing the expected impacts of small hydropower plant (SHPP) on the environment it is necessary to determine the impact of the proposed activity on the groundwater level regime in adjacent areas. The aim of this work is a mathematical model flow direction and groundwater regime before and after the implementation of SHPP Šalková on the Hron River designed by HCI

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HYDROCONSULTING, Ltd. [1], and to consider the proposal bilge drain and its functionality on the project [1].

2 MATERIAL AND METHODS

Part of the investigated area is located in the sub-basin of river Hron. This is a hydrogeological region built by quaternary sediments SK1000700P (intergranular groundwater points of Quaternary sediments of the Hron River watershed) (Fig. 1). Base quaternary sediments mentioned throughout the study area consist of pre-quaternary formations SK200390KF (Dominant fissure-karst groundwater body of the Muran Plateau in the Hron watershed) (Fig. 2).

In terms of regional geological division of the Western Carpathians, the area of interest lies in Hron synclinorium in the western part of Veporian zone. According to the geological map of the northern part of the Zvolen basin are the surroundings of the area of interest in the valley of the river Hron built with Quaternary fluvial formation of Hron. Fluvial sediments of Hron consist of layers composed mainly of fluvial sandy loam and loamy sand, loamy fluvial gravels, fluvial sandy gravel and loamy fluvial sandy gravels [2], [3].

After gathering the necessary supporting documents I continued with the modelling of surface water levels using the software package HEC-RAS [4], [5], [8]. The outputs of the modelling were curves of water surface for different levels of design flow. The longitudinal flow profile is plotted for water level in the river for operational flow $Q_p = 30\ m^3 \cdot s^{-1}$ (Fig. 3).

Course at an operational flow (Fig. 4) is significantly affected by backwaters caused by a dam.

Fig. 1 Bodies of groundwater in Quaternary sediments [10]

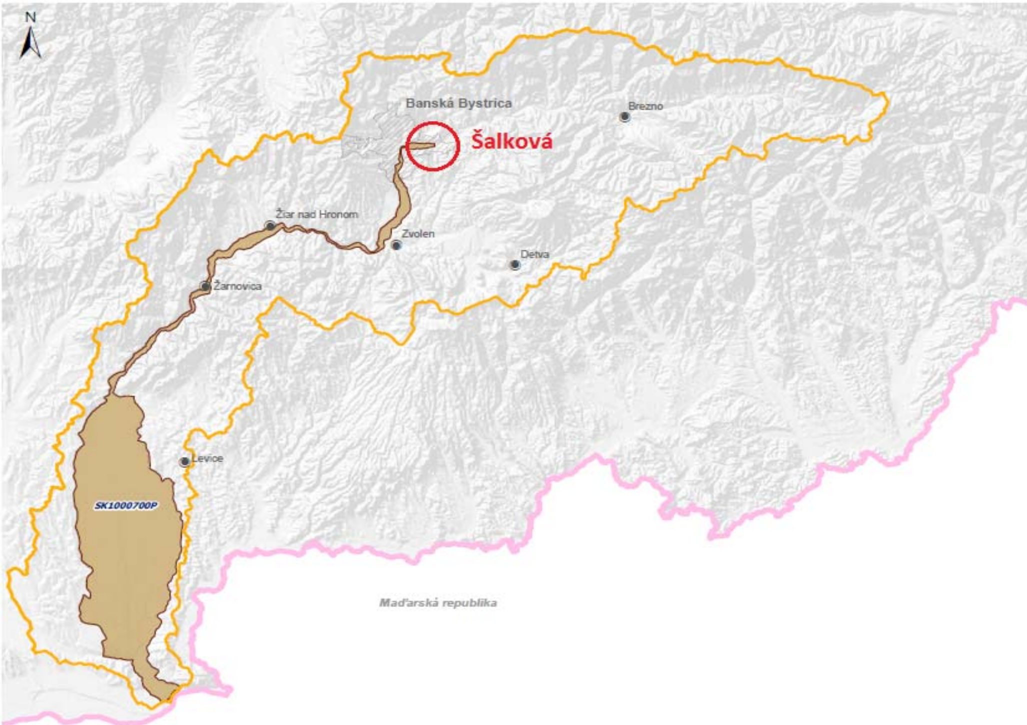

Fig. 2 Bodies of groundwater in Quaternary sediments, detail [11]

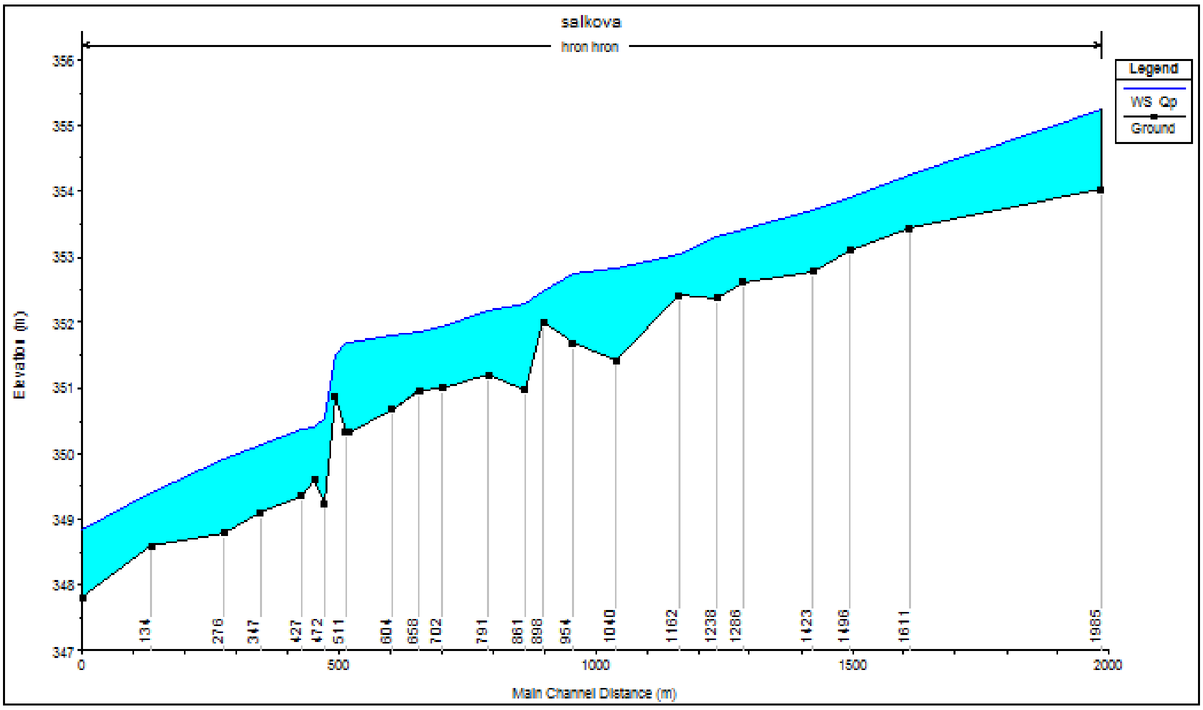

Fig. 3 Progress of water surface levels for operational flow Q_p

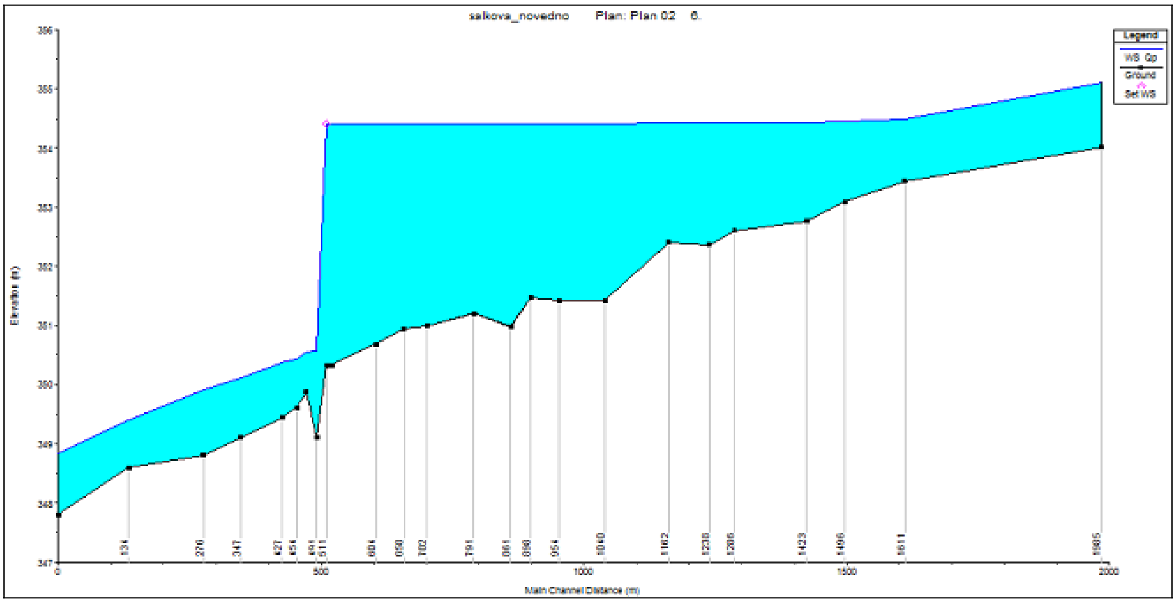

Fig. 4 Progress of water surface levels for operational flow Q_p after building the hydroelectric project - small hydro power plant

The next step is to create finite element mesh as the mathematical basis for calculating the course of groundwater levels (GWL) and other parameters [6], [7] of hydrogeological layer. Graphical output of the program Triplot shows an interpretation of finite element mesh of triangles (Fig. 5), together with the refinement in the area of proposed network of underground sealing walls.

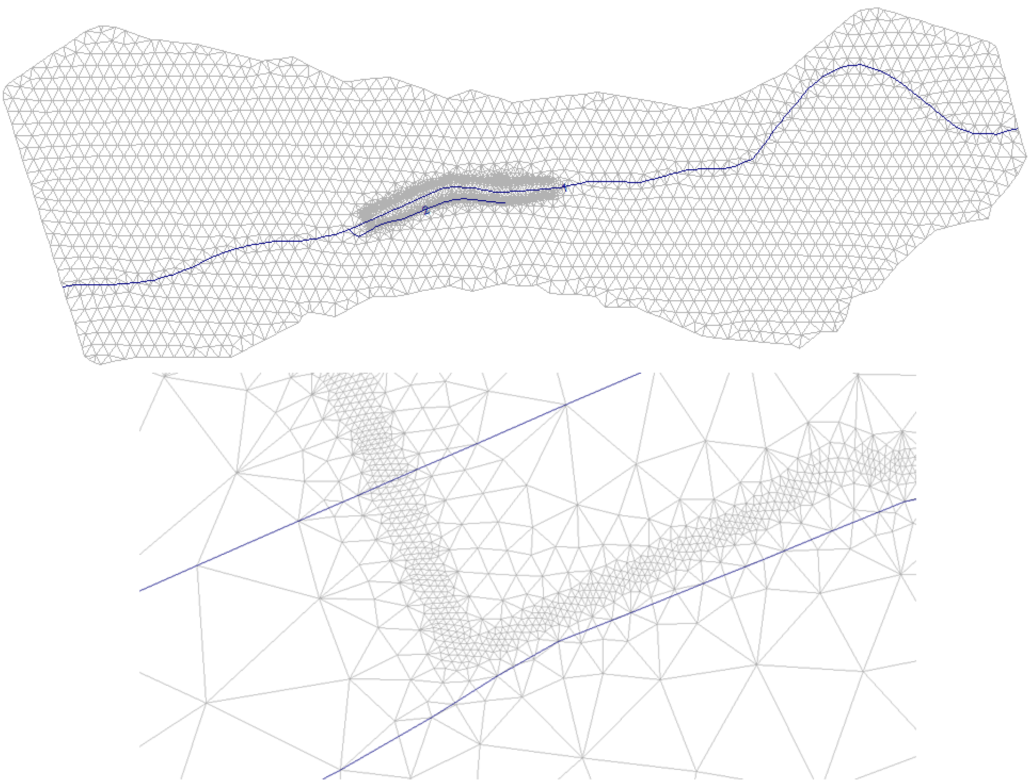

Fig. 5 Visual interpretation of network elements in the program Triplot, detailed refinement of networks in USW

Another necessary part of modelling is defining boundary conditions of the study area (Fig. 6), which was introduced as Dirichlet boundary condition, i.e. using groundwater level, represented in blue, and the Neumann boundary condition defined by the inflow to the area, shown in red.

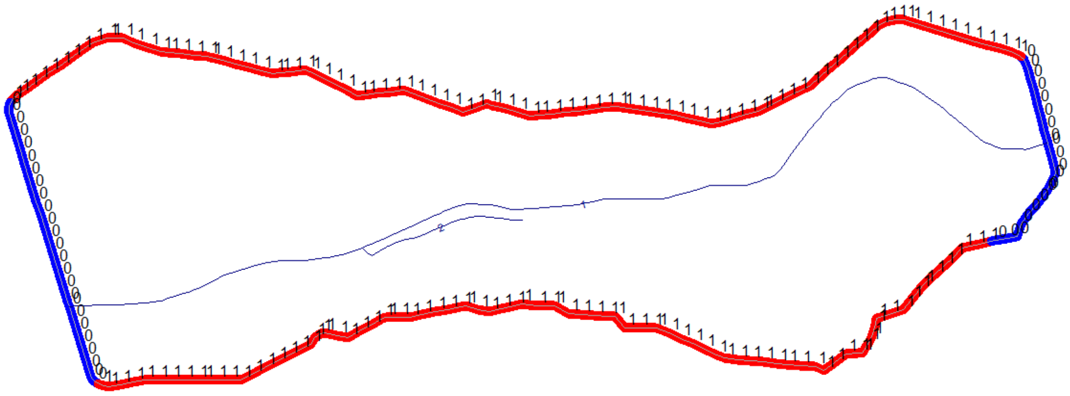

Fig. 6 Boundary conditions

3 RESULTS

After a launch and subsequent successful finalization of calculations software package Triwaco provides the resulting data model as a graphical interpretation in the graphic interface called Triplot. To create a concept of the original course of the groundwater level before the construction of small hydro power plant it is necessary to create a graphical interpretation of the current state of the area. The current state indicates the progress of groundwater levels prior to construction of water works Šalková, without backwater raise in level of surface water and level regime for normal surface water flow in the river Hron for average flow Q_p = 30 $m^3.s^{-1}$. Groundwater level in the urban area ranges from 352 to 354.5 meters AMSL (Fig. 7).

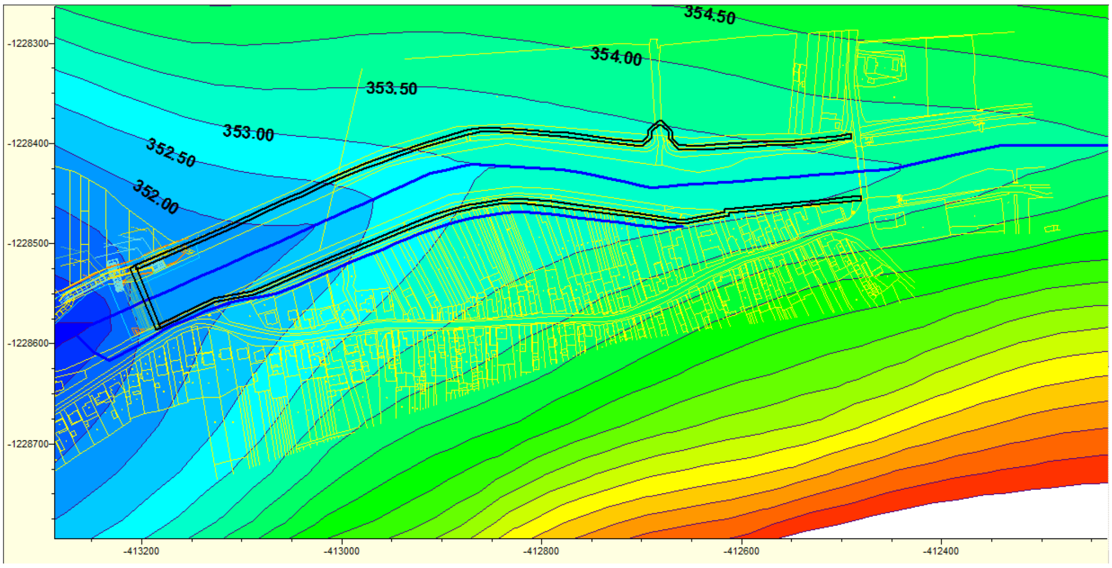

Fig. 7 Simulated GWL course (m a.s.l.) in the vicinity of the planned SHPP for the current state

Construction of small hydro power plant and putting it into operation results in impoundment of water level, which is also reflected in the impoundment of groundwater level, which may negatively affect the urban area of the village. It is therefore necessary to take measures to minimize the impact of backwater on the groundwater regime. According to the project it’s done by building underground sealing wall and the filter drain. The resulting regime of groundwater is visible in situation (Fig. 8).

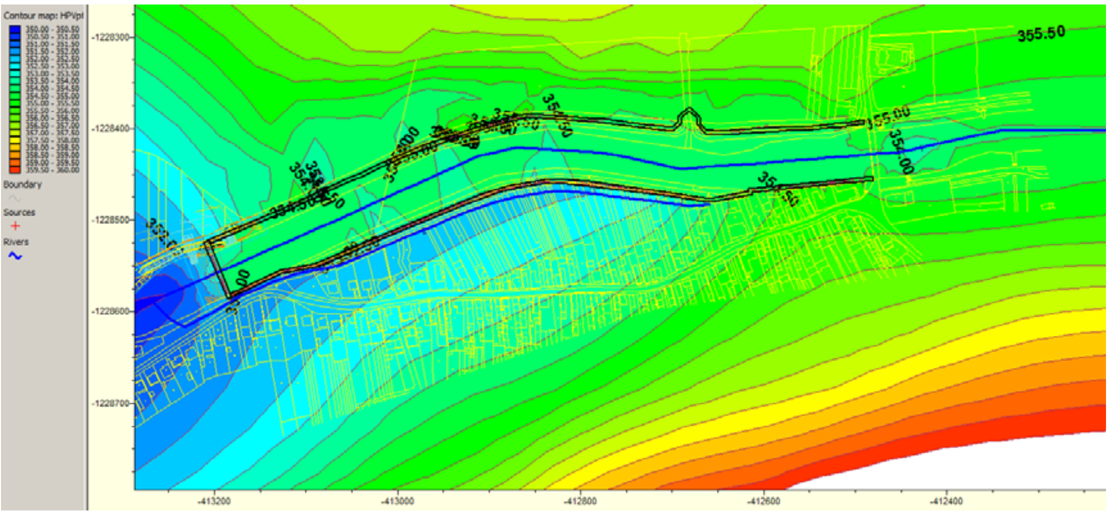

Fig. 8 GWL course (m a.s.l.) after the implementation of SHPP and control measures.

In situation (Fig. 9), the resulting change in groundwater level regime is visible simultaneously for the construction of water works as well as the implementation of regulatory measures, an underground sealing wall and filter drain. In the southern part of the site (left-hand side in the flow direction) the groundwater level will decrease in the range of 0.30 to 0.90 meters. In the northern part of the site (right-side flow) there is a noticeable rise in the water table. The area of right bank is mostly composed of fields and pastures and there are no urban areas or amenities present, so this increase in groundwater level can be considered as not negative and that the rise won’t affect the incident site or area on the left bank side of the river.

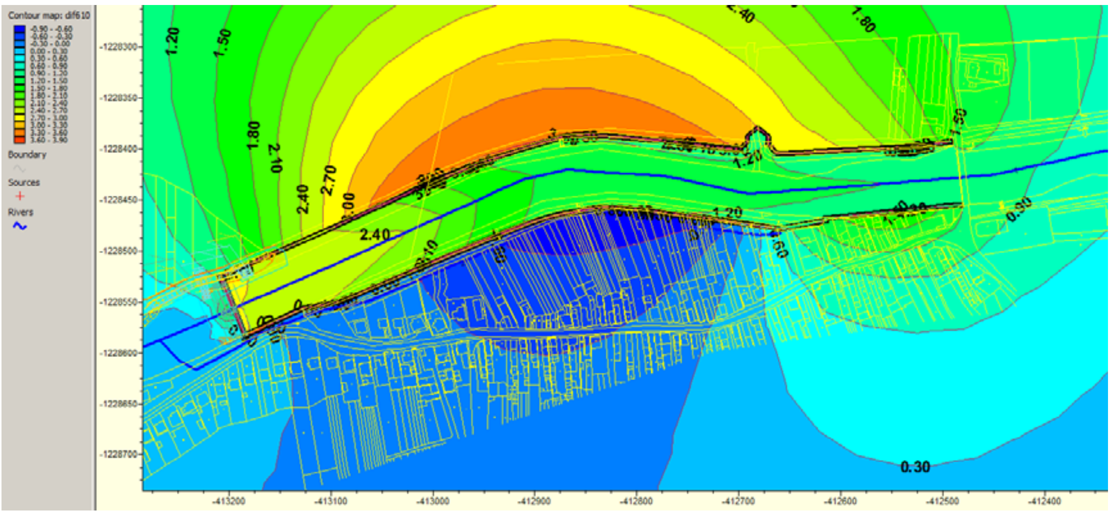

Fig. 9 GWL changes isolines (m) in view of the current state (differences before and after the implementation of SHPP and control measures)

4 CONCLUSION

Proposed measures for minimizing the impact of backwater water structure can be considered as functional and efficient. From the outcomes of modelling based on supplied materials can be assumed that the condition of post-construction groundwater regime did not worsen and there is a partial reduction in groundwater levels which appears to be beneficial, since the site is present in urban area. At the same time the groundwater level did not drop significantly, what could adversely affect the supply wells and other features that are closely related to the yield of groundwater.

Acknowledgements
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HYDROLOGICAL ANALYSIS OF FLOW VARIATIONS ON SHPP SITE

Gordon Gilja[1], Eva Ocvirk[2] and Boris Beraković[3]

Abstract

The use of renewable energy resources is greatly encouraged by many governments, with aim for achieving sustainability whilst satisfying ever-growing demand for energy consumption. Among the renewable resources of energy in the world, hydro power plants produce the most electric energy. Small hydro power plants (SHPP) partake in overall hydro power production with around 7% of installed power (approx. 50 GW). Of special interest for small hydro power plants are sites where in the past water power and energy was used, particularly from standpoint of environment acceptability and regional and country planning. According to the regulations in the energy sector: "Besides the Croatian national electricity company (HEP), persons, companies and other legal persons can also produce and distribute electric power". Currently third of the small hydro power plants in Croatia are in private ownership, and trend of private investments is showing continuous growth as investors see possibility for good rate of investment return. Installed power for 95 % of available locations is less than 1 MW. On these watercourses variety of flow conditions can occur which are poorly recorded because of monitoring network scarcity. This paper covers hydrological analysis of the Lika River for location of small hydro power plant Miškulin near the city of Gospić. The Lika River is torrential stream with zero-flow period during dry season and is influenced by considerable backwater from reservoir Kruščica during rainy season. Hydrology at SHPP site is under influence of the Lika River's four tributary streams. This example illustrates method for analysis of sites under extremely temporally and spatially variable hydrological conditions.

Keywords

Renewable energy resources, small hydro power plant, the Lika River

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1 INTRODUCTION

The hydro power plants can be categorized in many ways – by type of the turbine, size, installed power, etc. Small hydro power plants (SHPP) are plants with an installed power smaller than the limit, which varies from state to state (in Croatia small hydro power plants are those from 500 kW to 10 MW of installed power). In the European Union and in the most countries of the world upper limit for SHPP is 10 MW. Small hydro power plants are becoming increasingly important in the power systems of the developed countries [1]. There are number of plants built all over the world and because of the good experiences in building and working the interest for them is increasing, as shown in table (Table 1). There were more than 60000 small power plants in the world in 2002.

Table 1. Summary of SHPP use in Europe.

Country	***Installed power (MW)***	***No of SHPP***
Austria	670	1720
Czech rep.	200	1200
France	1972	1717
Germany	1300	6000
Italy	2000	1510
Norway	950	550
Spain	1540	/
Sweden	1050	1615
Switzerland	750	1000
Croatia	73.2	32

Among the renewable sources of energy in the world, hydro power plants produce the most electric energy. The potential for the total installable power of SHPP is estimated to be about 180 GW, which is about 6 % of the estimated total installable power in hydro power plants [2]. Europe ranks second in energy contribution from SHPP on global level, following Asia. In year 2001 SHPP contributed approximately 2 % in total energy production, and 9 % in energy production from renewable resources [3, 4]. In 1985 Republic of Croatia published *Croatian Register of Small Streams* which gave the first assessment of possible SHPP locations on 134 streams. More detailed study declared 63 streams from afore-mentioned 134 as suitable for construction of small hydro power plants. On those 63 streams 699 locations were identified, with total installed power about 177 MW (Table 2).

Table 2. Summary of potential SHPP locations in Croatia by installed power.

Installed power (MW)	***No of SHPP sites***	***Total installed power (MW)***
1.5 - 5	20 (3 %)	50.2 (29%)
1 – 1.5	17 (2 %)	21.7 (%)
0.5 – 1	42 (6 %)	28.7 (%)
0.1 – 0.5	296 (42 %)	55.7 (%)
< 0.1	324 (47 %)	20.7 (%)
Total	699	177

Number and availability of potential SHPP sites is inversely dependant of installed power, because smaller natural drops are more common than bigger ones [5]. Croatian watercourses are characterized by large number of small drop sites, with more suitable sites located in upper part of basin – remote and populated areas with insignificant power consumption and scarce distributive network, as well as natural landscape areas. Most of the small hydro power plants are in private ownership –restored water mills built and operated throughout history and abandoned during early 20th century. This paper presents hydrological analysis and energy production determination for SHPP Miškulin situated on Lika River in upper part of reservoir Kruščica which is a part of HPP Senj. Water stage oscillations in reservoir are between 30 m and 60 m what are reflected in upper part of reservoir. The idea is to better use of Lika river energy with SHPP, which will operated in specific circumstances and what cause specific hydrological and hydraulically analyses.

2 HYDROLOGICAL DATA

SHPP planning consists of determining tailwater elevation, headwater level and backwater effect on upstream river reach and floodplains. Since SHPP sites are rarely located at gauging stations these values are calculated using 1D numerical flow model of range of hydrological events. Flow model is defined with 37 cross-sections, with downstream boundary condition set on gauging station GS Budak and upstream boundary condition set on GS Bilaj (both on Lika River). Hydrological regime on defined river reach is influenced by several factors: discharge of Lika River, inflow from tributaries Jadova River (upstream of SHPP site) and Novčica River (downstream), as well as backwater generated from Kruščica reservoir situated 50 km downstream (**Fig. 1**). Discharge at SHPP site is sum of Lika River flow (GS Bilaj) and flow from tributary Jadova (GS Barlete).

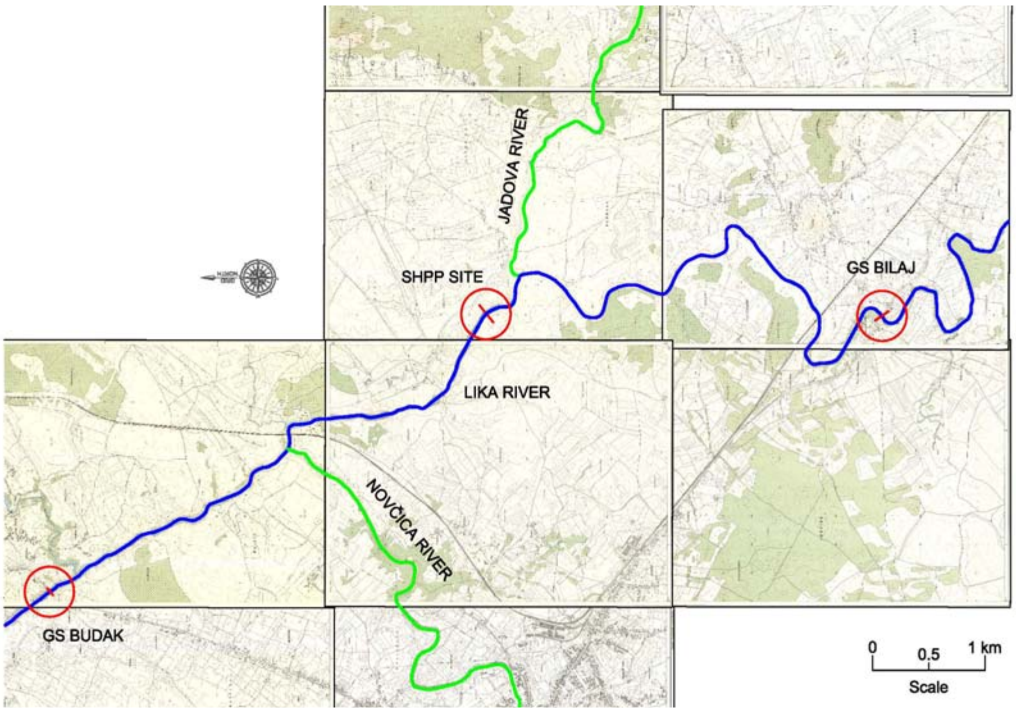

Fig. 1 Modelled river reach with location of boundary condition profiles, tributaries and SHPP site.

Downstream of SHPP site flow conditions are unfavourable because of inflow from tributary Novčica River (calculated from GS Lički Novi on Novčica River and GS Kolakovica on Bogdanica River - Novčica's tributary). Downstream boundary condition of flow model is GS Budak on which only stage hydrograph is recorded, discharge curve cannot be established because of significant backwater effect from Kruščica reservoir (**Fig. 4**). Discharge on GS Budak is thus calculated as sum of Q_{BILAJ}, Q_{JADOVA}, $Q_{NOVČICA}$ *and* $Q_{BOGDANICA}$. Next figure (**Fig. 2**) shows hydrographs for all GS in year 2004. Hydrological regime of rivers with small watersheds usually results in low median discharges with extremely large ratios of peak and low discharge. Mean and maximum discharges are given in following table (Table 3).

Table 3. Characteristic discharges for Lika River and its tributaries.

	Q_{LIKA}	Q_{JADOVA}	$Q_{NOVČICA}$
Q_{MEAN}	5.6	3.4	4.7
Q_{MAX}	145	103	117.3

There is visible dry season from June till October, with no flow during August and September, while peak discharges occur during winter and spring.

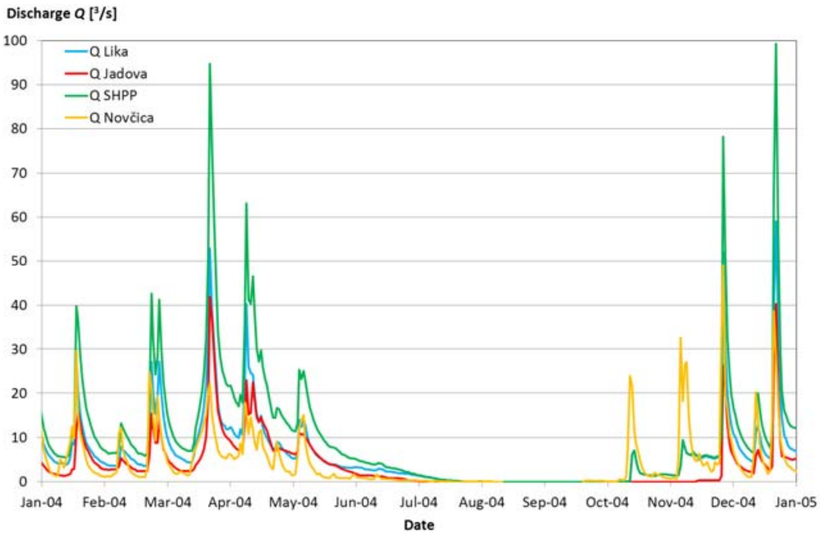

Fig. 2 Flow hydrographs for Lika River and its tributaries in year 2004.

3 HYDROLOGICAL ANALYSIS

In order to establish boundary conditions for analysis of hydrological regime, correlation between discharges of Lika River and its tributaries must be defined for range of discharges. For discharge correlation analysis base station used is GS Bilaj (upstream boundary condition) and time period from 2003 to 2011. Discharge on all GS is correlated with GS Bilaj discharge Q_{BILAJ} with goal to define relationship between them. Since seasonal variation of discharge on defined river reach is pronounced (**Fig. 2**) there is no significant correlation between Q_{BILAJ} – $Q_{NOVČICA}$ (R^2 = 0.77) In order to define discharge correlation more accurately year is divided in 6 periods: period G1 (January and February), period G2 (March through May), period G3 (June and July), period G4 (August), period G5 (September and October) and period G6 (November and December).

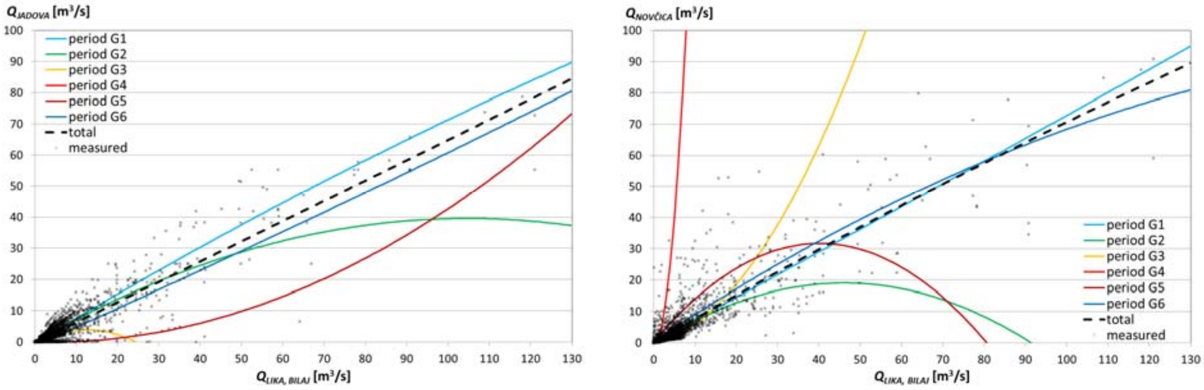

Fig. 3 Discharge correlation between Q_{BILAJ} and tributary: a) Jadova River; b) Novčica River.

Scatter plot given for discharge correlation shows that correlation strength for different periods varies (Table 4).

Table 4. Coefficient of determination from discharge correlation analysis.

Period	R^2 $Q_{JADOVA} = f(Q_{BILAJ})$	R^2 $Q_{NOVČICA} = f(Q_{BILAJ})$
G1	0.92	0.80
G2	0.70	0.64
G3	0.74	0.58
G4	0.04	0.77
G5	0.76	0.57
G6	0.84	0.78
total	0.85	0.77

There is visible strong correlation between discharge Q_{BILAJ} and Q_{JADOVA} throughout all year except during dry season that occurs in August (**Fig. 3**a). Winter and spring periods show similar trend of correlation, which corresponds with relationship for total time period. Periods G3 and G4 characterized with dry season and small number of observations show distinct pattern. Period G5 also shows weak correlation because of high flow oscillations in autumn. For hydrological analysis purposes relationship between discharges Q_{BILAJ} and Q_{JADOVA} is strong enough for periods in which inflow encourages energy production. On the other hand, correlation between discharge Q_{BILAJ} and $Q_{NOVČICA}$ shows no significant relationship (**Fig. 3**b). Two winter periods (G1 and G6) show same relationship pattern which corresponds with relationship for total time period. During spring period G2 relationship between Q_{BILAJ} and $Q_{NOVČICA}$ shows similar pattern for most of the data, with only extreme discharges deviating from this pattern. Periods G3, G4 and G5 show distinct pattern, because of the same reasons already determined for afore-mentioned Jadova River. There is visible significant dispersion on scatterplot Q_{BILAJ} and $Q_{NOVČICA}$ for discharges under 50 m^3/s (**Fig. 3**b), which is very important for determination of tailwater level and energy output.

Beside discharges, water surface elevation must be known on downstream profile of defined river reach in order to establish resulting water surface profile for given hydrological conditions. Downstream profile of defined river reach is GS Budak on Lika River (**Fig. 1**), and measured pairs of *Q-H* data for this profile are given on figure below (**Fig. 4**) for period from 2004.

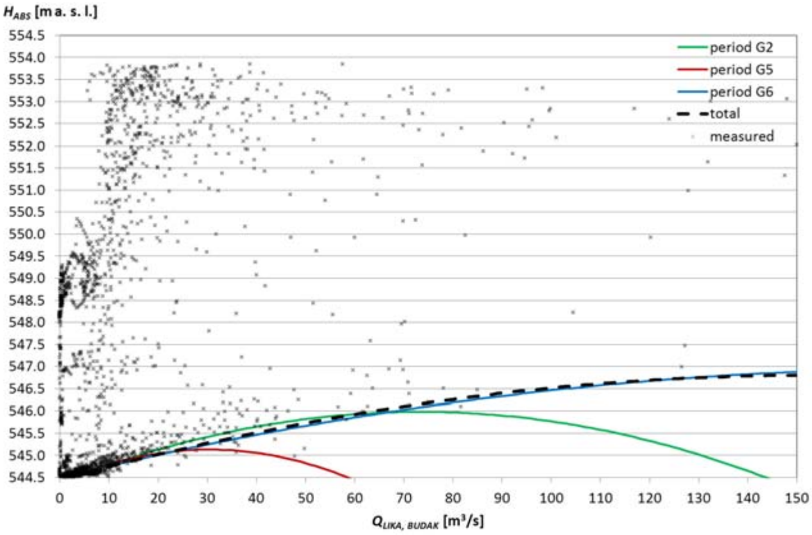

Fig. 4 Pairs of *Q-H* data for GS Budak.

From scatter plot of GS Budak (**Fig. 4**) is visible that its water levels are under influence of backwater flow from Kruščica reservoir. On figure is shown idealized discharge curve without backwater effect (**Fig. 4**, hidden line) and discharge curves for periods that are not influenced by Kruščica's backwater (winter period G6, spring period G2 and autumn period G5). These periods without backwater effect coincide with ones defined to have strong relationship between tributary discharge and Q_{BILAJ} (**Fig. 3**a, **Fig. 3**b). On following figure (**Fig. 5**) is shown hydrograph during year 2004 for Q_{SHPP} and absolute difference in water levels of two boundary condition profiles of defined river reach: GS Bilaj (upstream, **Fig. 1**) and GS Budak (downstream, **Fig. 1**).

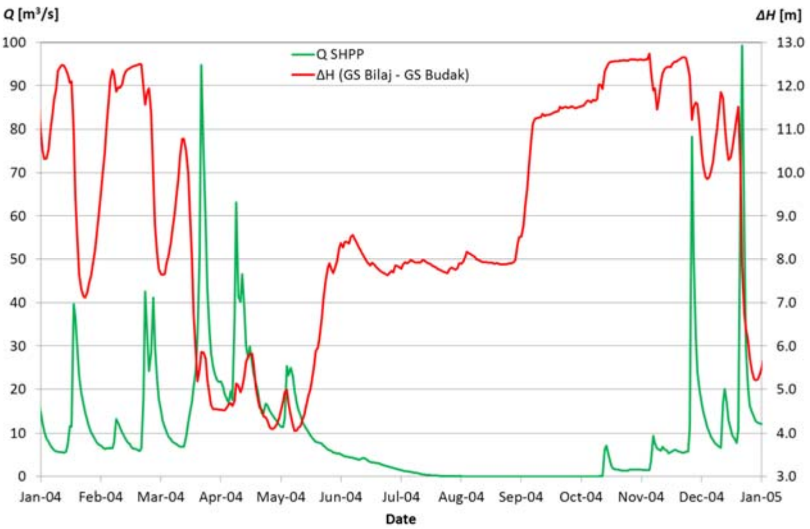

Fig. 5 Hydrograph for Q_{SHPP} and absolute difference in water levels on model boundary conditions in year 2004.

Hydrograph shows that during spring (period G2) when there is abundant inflow upstream backwater from reservoir Kruščica lowers available water head for energy production. During winter (periods G1 and G6) when there is reasonable inflow backwater influence oscillates and highest backwater effect coincides with peak discharges (**Fig. 5**). Period with most head available is during June through November (periods G3, G4 and G5) which is drought season. Backwater effect can cause increase in water elevation up to 10 m (**Fig. 4**). In such stochastic hydrological conditions there is no exact method to determine “stationary” hydrological boundary conditions for numerical modelling of flow through defined river reach. Therefore, characteristic values of discharge and water surface elevation cannot be used as reliable for numerical modelling of water surface profile and determination of tailwater at SHPP site.

4 RESULTS OF NUMERICAL MODELLING

Defined river reach is characterized with numerous submerged weirs which disturb flow field locally, creating pools and chutes. Calibration of roughness coefficient of such riverbed is usually impossible because weirs create local backwater effect which disturbs gravitational flow. Therefore, more emphasis must be given on calibration of coefficient of flow over weir crest and higher discharges which are less influenced by weirs than lower ones.
Because no deterministic pairs of *Q-H* points can be established on downstream boundary condition, standard procedure for energy production calculation cannot be used. Therefore, instead of using discrete values from discharge duration curve, all of the data from long-term observations must be used. Duration of period for which production is calculated must be long enough to include extremes with higher return period. For this analysis time period selected ranges from 2003 until 2011, with 2556 simulated daily hydrological events. Maximum discharges in this period are $Q_{max,\ Lika}$ = 145 m^3/s, $Q_{max,\ Jadova}$ = 103 m^3/s and $Q_{max,\ Novčica}$ = 105 m^3/s. When these values are compared with absolute extremes (Table 3), it shows that they reflect entire range of discharges that occur on this reach and that selected time period is reliable for calculation of tailwater elevation as input for energy output calculation. In this paper two variants of headwater elevations are defined: 555 m a. s. l. and 557 m a. s. l. Since energy production is not possible during restrictive hydrological conditions, *e.g.* during drought period or during very high backwater flow, not all given data can be included in calculation. Therefore, calculated pairs of SHPP tailwater data were filtered out to exclude ones that cannot be used. Filtered data included discharges 2 m^3/s < Q < 22 m^3/s and available head ΔH > 2 m (visible on **Fig. 6**).

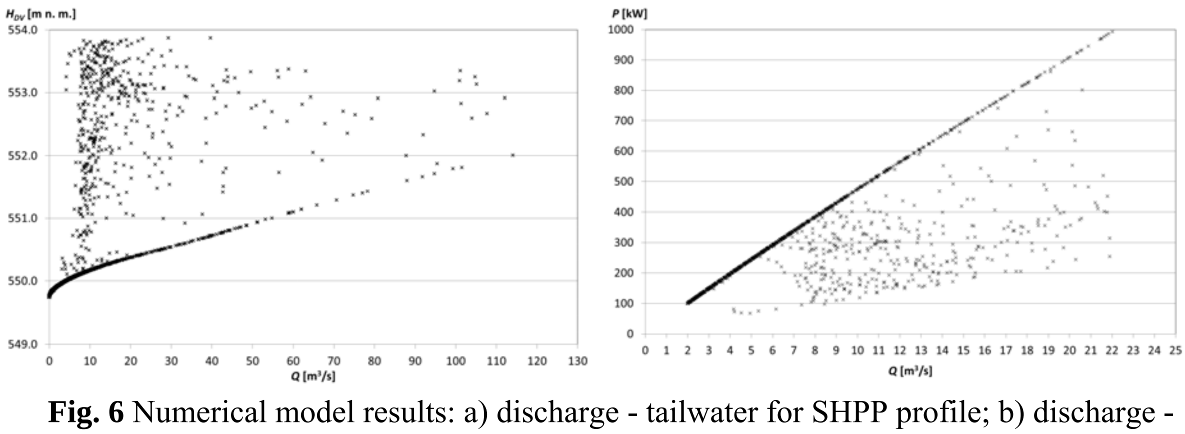

Fig. 6 Numerical model results: a) discharge - tailwater for SHPP profile; b) discharge - power

On next figure (**Fig. 7**) is given comparison of power duration curves for two defined headwater elevations.

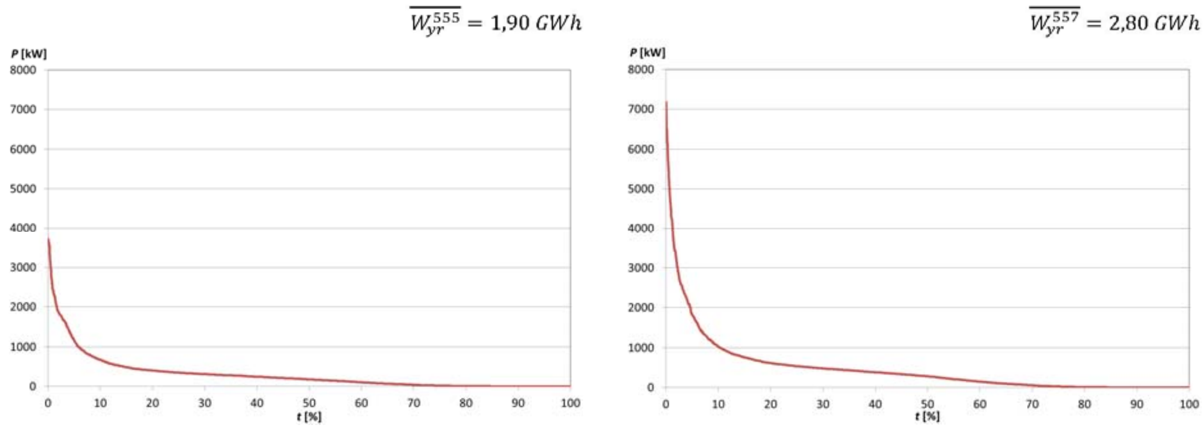

Fig. 7 Power duration curve for headwater elevation at: a) 555 m a. s. l.; b) 557 m a. s. l.

5 CONCLUSION

This paper presented overview of complex hydrological regime on Lika River. Established relationships between discharge of Lika River and its tributaries Jadova and Novčica enabled determination of boundary conditions for 1D numerical model. Numerical modelling of flow was used for description of hydrological regime and determination of energy output for SHPP which showed that potential for energy harvesting of small watercourses at far end of hydro power plant reservoirs exists.

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COMPUTER ALGORITHM FOR ANALYSIS OF BEDFORM GEOMETRY

G. Gilja[1], N. Kuspilić[2] and B. Brckan[3]

Abstract

Sandy riverbeds are covered by periodic bedforms of different scales, from ripples to antidunes. Under certain flow conditions simultaneously more than one type of bedforms can occur - smaller bedforms are superimposed on bigger ones. The superposition of bedforms causes difficulties for determining individual bedform parameters. In flume experiments bedform data is averaged for entire flow field - sum of total lengths/heights is divided by number of present bedforms. This method has limited applicability to bedform field with uniformly shaped dunes over relatively mild sloped riverbed. If more accurate description of bedform geometry is required other methods for bedform description have to be utilized. This paper presents comparison of two methods for separating different scales of bedforms from Multibeam Echo Sounding (MBES) data: dune geometry components from the mega ripple component. One method determines manual decomposition of the MBES signal, and the other method uses computer algorithm developed for this purpose to calculate signal decomposition. Both methods are described and tested on multiple MBES data sets. As a first application of the separation method individual bedform parameters of bedforms are identified: more particularly wavelength and wave height of bedforms. Results from both methods are then compared in order to validate implemented algorithm logic in computer calculations. The described algorithm represents a more versatile option for accurate description of the shape of the complex bedform geometry, compared to conventional approach.

Keywords

Bedforms, computer algorithm, Multibeam Echo Sounding

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1 INTRODUCTION

When a unidirectional turbulent flow acts on a flat bed of non-cohesive sediments, complex interactions between turbulent flow, sediment transport and bed morphology give rise to various types of river bed configurations. A wide variety of bedforms is known to develop at an assortment of scales under unidirectional flow in rivers. There is general agreement in the literature that there are at least two distinct bedform scales formed in sand under unidirectional, lower-regime flow; relatively small scale ripples and relatively large scale dunes. Criteria to distinguish these two distinct bedform scales include (1) sediment caliber, (2) hydraulic roughness, (3) bed form shape or aspect ratio, (4) relevant length scale, (5) dimensionless excess shear stress (transport stage), (6) dimensional length, and (7) scaling with hydraulic system parameters, e.g. velocity, depth, sediment size, Shields' nondimensional shear, stream power, Froude number [1-4]. Dunes are the most common bed configuration in sand-bedded streams, forming in a range of sediment sizes from silt and sand through to gravel. Dunes in river flows determine hydraulic resistance, sediment transport, channel morphodynamics and hydraulic habitat for biota. They also often present a major problem for engineering structures (e.g., water intakes or discharges, pipelines, groynes, etc.) and may introduce severe restrictions to navigation. Knowledge of such phenomena, and of their effects on the flow characteristics, can greatly benefit the design of rivers and canals, estuarine and coastal modelling studies, flood studies, the estimation of the depth of erosion around structures and rates of sediment transport, etc. [1, 5].

Fig. 1 Dune field.

Experimental studies with medium to fine sand reveal that a number of dune states exist that are stable only between certain values of flow velocity/bed shear stress and sediment-size. For this reason, dunes developed in sand beneath steady flow have been well studied in laboratory conditions. In controlled laboratory conditions observation of dune field is relatively easy: after steady flow conditions have been established for given time period and dune field has formed water is drained from flume and measurement of dunes takes place. When measurement of dune field takes place in estuary or river, it is conducted in highly adverse environment: large areas covered in deep water have to be surveyed which is time consuming and eventually with time hydrological conditions change and exert forces on riverbed which change its morphology. Although dunes play significant role in river engineering and management their characteristics are not part of standard hydrological monitoring. Their presence may be noted during discharge measurements or occasional fathometer profiles, but bedform-mapping requires specialized equipment and data processing. In small hydrologic studies detailed channel topography can be measured using small boats with no built-in navigational hardware. Advances in development of measurement equipment led to invention of hydroacoustic equipment designed for data bathymetry data collection in riverine environment: multibeam echo sounders (MBES). This equipment has ability to collect data through the swath of acoustic beams which provide large coverage of the riverbed during mobile surveys (**Fig. 3**).

Data collected with multibeam contain hundreds of thousands of elevation points and that makes its analysis time consuming. Field datasets also contain large number of noise data due to vegetation cover, man-made trash, remains of constructions, etc. Noise data has to be identified and filtered out which can be done in post processing, but not to absolute amount. Also, rivers and estuaries have sloped beds which influence dune placing - reference plane cannot be established for entire dune field (*e.g.* as flume bottom) and has to be redefined for different parts of surveyed area. All of the above results in numerous man-hours needed for visual recognition of dune field in order to describe characteristics of dunes in natural environment. Purpose of this paper is to develop computer algorithm which uses pattern approach to decompose recorded riverbed profile into individual dunes. Proposed separation method identifies individual dune parameters: more particularly wavelength λ and wave height Δ. Results obtained from algorithm are then compared to conventional, visual pattern recognition method in order to validate implemented algorithm logic in computer calculations and identify its weaknesses and strengths.

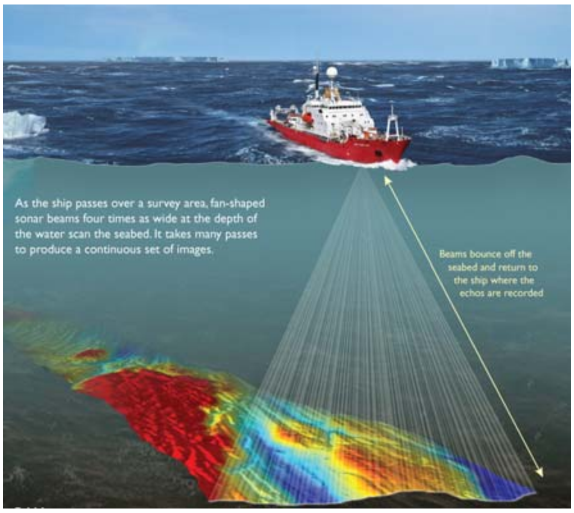
2 FIELD SURVEY

A reach of the Drava River at Nemetin was chosen for study because it provided: (1) "pseudo-steady" flow conditions for duration of data collection of 10 to 15 hours; (2) natural flow environment undisturbed by presence of river training works; (3) extensive database of flow and bathymetry data collected since 2006; (4) good logistics with a boat launch adjacent to the data collection reach (**Fig. 2**).

Fig. 2 Surveyed Drava River reach with defined longitudinal profile (magenta).

In 2012 3D riverbed morphology of dunes was collected on longitudinal profile with MBES (**Fig. 3**). Longitudinal profile was selected in such way that passes through high velocity filaments of river cross-sections where dunes of highest magnitude arise. Survey was conducted on 2 km long river reach of which upper section is in natural conditions and lower section in man-made river cutoff (**Fig. 2**). This reach is suitable for development of computer algorithm because of this various flow conditions - lowland natural flow in upper section with low mean flow velocity, accelerated flow on entrance in cutoff and high flow in narrowed profile of lower section. Distinct flow conditions of these two sections ensure that riverbed is not going to be covered with uniformly shaped dune, but with complex forms which are suitable for validation of logic programming introduced in developed algorithm.
Multibeam unit used for bathymetry survey was ODOM ES3 with an array of transducers that simultaneously transmit pings (sound pulses) at a specified frequency to cover a large area in short time. Multibeam ODOM ES3 uses swath of 420 acoustical beams, with up to 3° width each, transmitted towards bottom in direction perpendicular to boat orientation (**Fig. 3**).

a)



b)

Fig. 3 Multibeam echo sounder: a) boat mount, b) operation scheme.

Transmitted signal reflects from hard bottom and returns in active transducer sensors which calculate distance traveled through equation:

$$l = \frac{t}{2} \cdot v \text{ [m]}, \tag{1}$$

where: l - distance traveled by acoustic signal [m], t - elapsed time [s], v - measured sound velocity in water [m/s].
Width of region ensonifed by swath of beams is dependent of water depth, with maximum of 80 m at water depth of 60 m and sector size of 120°.

3 PATTERN RECOGNITION METHODOLOGY

Bathymetry data collected with MBES consist of numerous points defined in wide band along boat track. In order to analyze profile of dune covered riverbed this data has to be defined as longitudinal profile perpendicular to dune crests. Therefore, section through collected data was defined with vertical plane through defined idealized longitudinal profile (**Fig. 2**). Since

defined profile isn't straight, longitudinal profile is straightened in a way that distance between consecutive points is drawn following course of longitudinal profile.
Generally, dunes have an asymmetrical shape, with a long stoss side slope, sharp crest and short steep lee side slope causing a flow separation zone (**Fig. 4**). These descriptive characteristics were used as guidelines for definition of dunes geometrical characteristics. Both visual pattern recognition method (VM) and algorithm pattern recognition method (AM) used three characteristic points for dune description: stoss toe P1(X1,Y1), brink point P2(X2,Y2) and lee toe P3(X3,Y3).

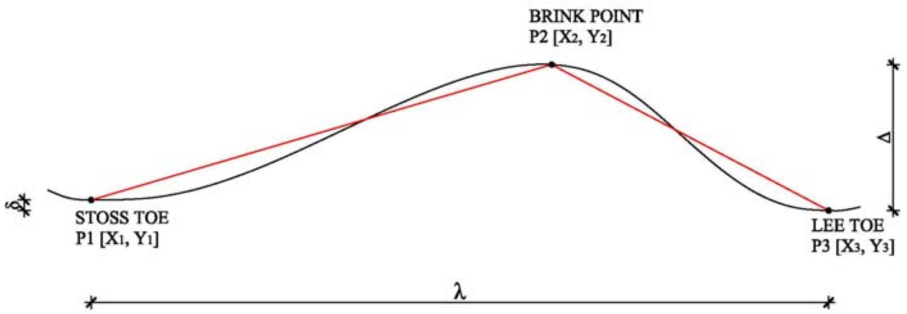


Fig. 4 Idealized dune profile with characteristic points.

VM method consisted of manual definition of dune geometry with drawing of individual polylines through points P1, P2 and P3 successively. These points are defined through visual inspection of distorted longitudinal profile with regard to defined boundary conditions. Set of defined boundary conditions included minimum dune length λ_{MIN}, minimum dune height Δ_{MIN}, and maximum vertical difference δ_{MAX} between two toes, P1 and P3. Dune characteristics were calculated from geometrical characteristics of lines describing dune. Dune length was calculates as absolute distance between points P1 and P3, and dune height as distance from point P2 to its orthogonal projection on line P1P3.
AM method used logic programming algorithms introduced by authors to define dune geometry. Dunes were also described with three points as in VM method. Input in AM method is extended to include minimum number of points defining stoss slope n_U, and lee slope n_D (**Fig. 5**). Minimum number of points that define slopes is used for elimination of noise data that have small number of points that reflect triangular geometry similar to dune.

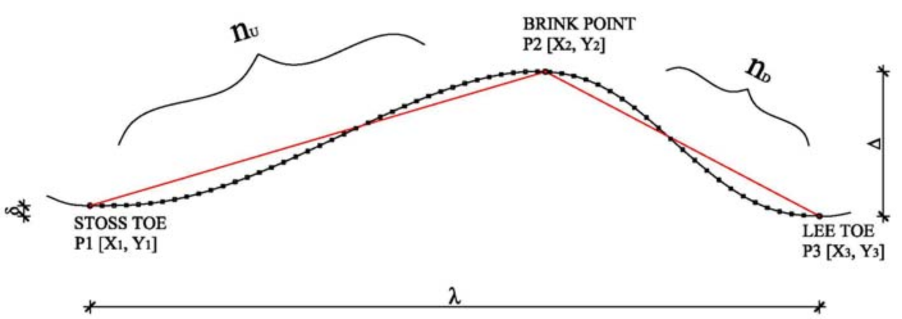


Fig. 5 Dune profile as "seen" by AM method.

4 RESULTS

Characteristics of dune field are determined with VM and AM method for two surveys, S1 and S2. Comparison of determined characteristics of dune field with VM and AM method is shown graphically. Histogram of recognized dune lengths is given on figure (**Fig. 6**), with S1 values in top row and S2 values in bottom row. Algorithm recognized significantly more dunes than visual method for both surveys: 362 to 107 on survey S1 and 320 to 82 for survey S2. Next figure shows histograms of recognized dune lengths (**Fig. 6**). Recognized dune lengths for both methods are found in same span (3 m to 25 m for survey S1 (**Fig. 6**a; **Fig. 6**b) and 3 m to 10 m for survey S2 (**Fig. 6**c; **Fig. 6**d). AM method defined more dunes for both smaller and longer dune lengths, though this difference is more pronounced in range of dunes shorter than 11 m.

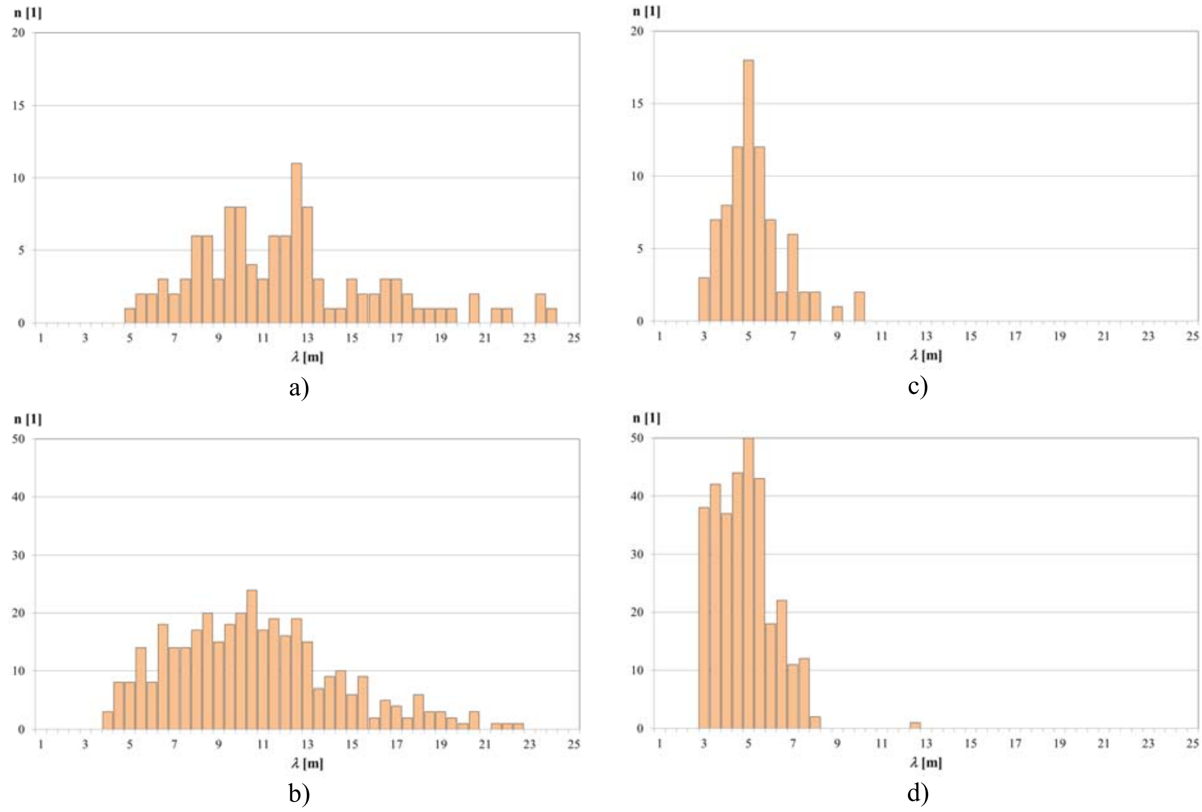


Fig. 6 Histogram of dune lengths for survey: S1 [a) VM; b) VA] and S2 [c) VM; d) VA].

Second survey (S2) is conducted in conditions of lower discharge and smaller water depth, which resulted in significantly shorter dunes recorded than ones on survey S1, while total number of dunes is approximately the same. Next figure (**Fig. 7**) gives comparison of dune lengths defined by two methods, VM and AM. There is visible good alignment of data for second survey, while first survey has more scatter in data. Generally, significant scatter is present for dune lengths that weren't recorded on second survey, while best alignment is present for smaller dune lengths that weren't recorded on first survey. For situation when VM method defined longer dunes there is significant gap between outlier values and ones placed next to line of agreement (**Fig. 7**). When VA method defined longer dunes outliers are placed closer to line of agreement.

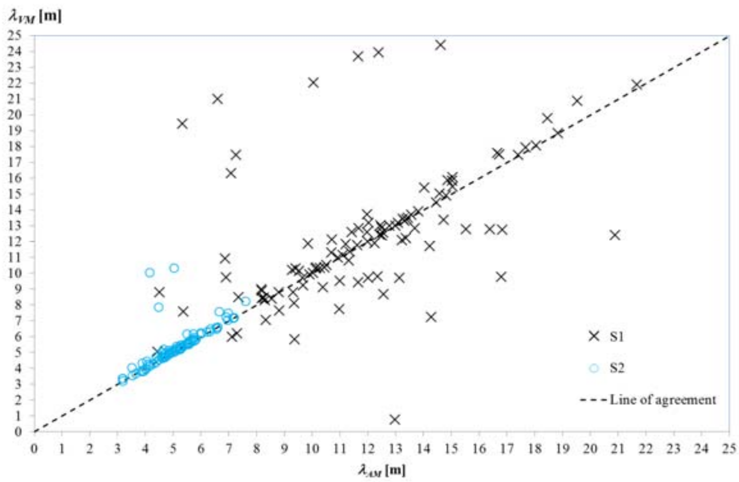


Fig. 7 Comparison of dune lengths for surveys S1 and S2 recognized by VM and VA method.

Identified outliers positioned above the line of agreement on scatter plot (**Fig. 7**) are isolated on next figure (**Fig. 8**). Discrepancy between AM and VM method occurs when dune has two (or more) peaks. If both peaks are defined with more than minimum number of points n_U and n_D AM method defines both of them as dunes and takes their dimensions into calculation. VM method, on the other hand, in these cases recognizes only one dune, dismissing the smaller one (pointed out with arrows on **Fig. 8**). In such cases dune defined with VM method is longer than the two dunes which defined VA method.

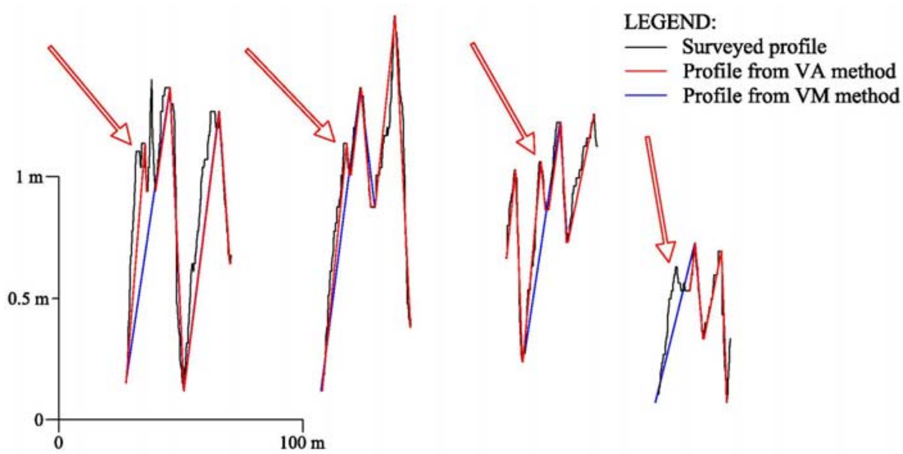


Fig. 8 Identified outliers in profile.

Outliers positioned below the line of agreement on scatter plot (**Fig. 7**) originate from similar discrepancy in pattern recognition approach between VM and AM method when lee toe of dune is located on long slope that ends in a depression. AM method then calculates this geometry as for normal dune while VM method filters this dune so that stoss toe and lee toe are in same level, resulting in shorter dune lengths. In this case difference between calculated lengths by two methods is not as large as for first group of outliers.

Next figure shows histograms of recognized dune heights, which are found in same span for both surveys (0.15 m to 1.20 m for survey S1 (**Fig. 9**a; **Fig. 9**b) and 0.10 m to 0.60 m for survey S2 (**Fig. 9**c; **Fig. 9**d). AM method defined more dunes for both smaller and longer dune lengths, though this difference is more pronounced in dune range smaller than 0.40 m.

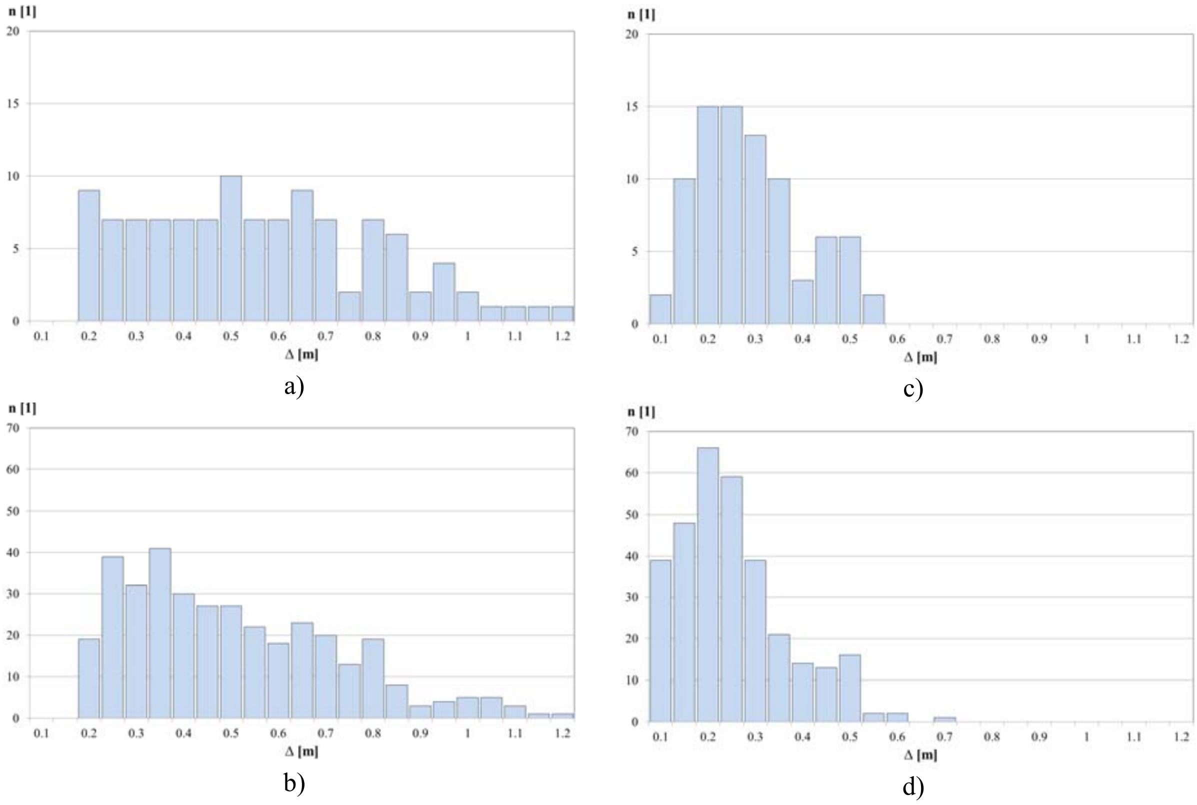


Fig. 9 Histogram of dune heights for survey: S1 [a) VM; b) VA] and S2 [c) VM; d) VA].

Next figure (**Fig. 10**) gives comparison of dune heights defined by two methods, VM and AM.

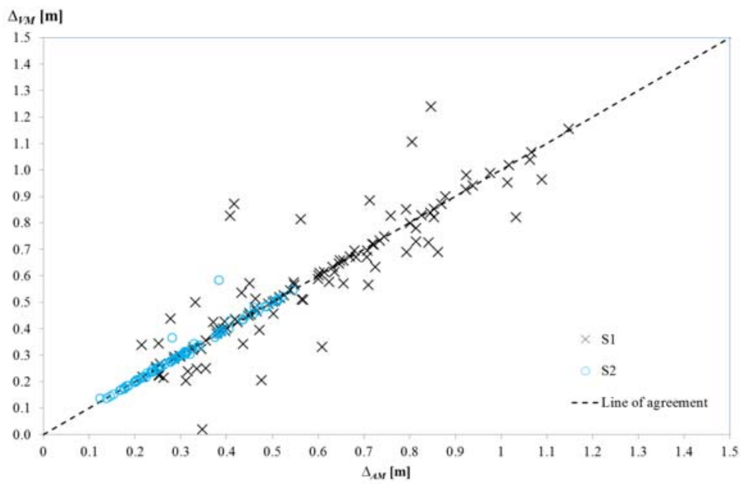


Fig. 10 Comparison of dune heights for S1 and S2 recognized by VM and VA method.

Discrepancy of dune height data around line of agreement (**Fig. 10**) is smaller than for dune lengths (**Fig. 7**). Better agreement between data is for survey S2, as for dune lengths. Origin of identified outliers is same as ones described for dune lengths: outliers positioned above the line of agreement on scatter plot (**Fig. 10**) occur when dune has two (or more) peaks; outliers positioned below the line of agreement originate when lee toe of dune is located on long slope that ends in a depression. Number of outliers above and below line of agreement and their distance is of same order of magnitude.
Next figure shows scatter plot of dune height to length ratio for both surveys (**Fig. 11**). There is no visible relationship between them, and threshold for dune steepness is approximately 0.12 (red line).

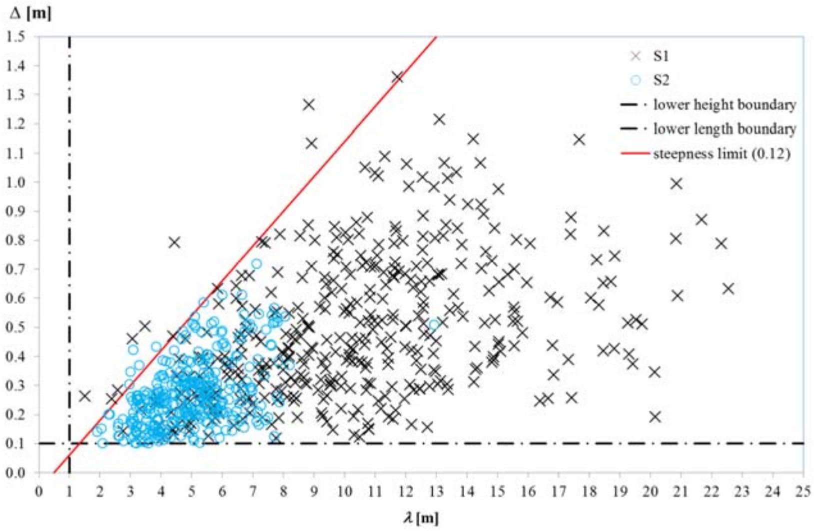


Fig. 11 Relationship between dune lengths and heights for S1 and S2.

Lower limits for dune heights and lengths are drawn in dash-dot lines. Recognized data for heights lies on this boundary, which implies that there is possibility of existence of smaller dunes. Recognized dune lengths are well above set limit of 1 m, *i.e.* there are no dunes shorter than 2 m in both surveys. Since pattern recognition with lower limits set at λ_{MIN} = 1 m and Δ_{MIN} = 0.1 m describes longitudinal profile well (**Fig. 12**) it can be assumed that there is no dunes with Δ smaller than 0.1 m and that smaller forms are ripples or dunes in beginning of forming.

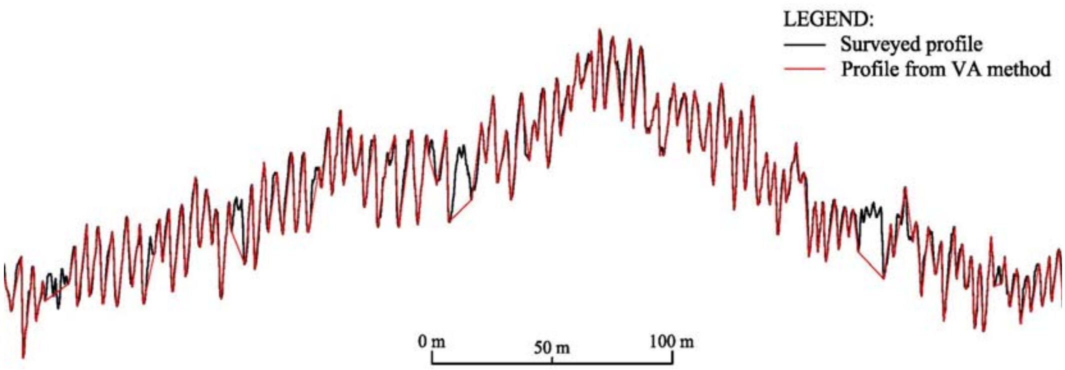


Fig. 12 Surveyed longitudinal profile vs. profile pattern from algorithm method.

5 CONCLUSION

Conducted research has shown applicability of developed algorithm for description of dune geometry from longitudinal riverbed profile. Dunes recognized with both visual and algorithm method show strong correlation for both dune lengths and heights. Limitation of algorithm are dunes with two brink points distant enough to have large number of points between them which cause algorithm to describe it as two smaller dunes. Visual method is heavily influenced by biased judgment of researcher outlining the dune profile. Pattern approach in visual method is has to be done on distorted profile which makes recognition of single dunes more difficult. Developed algorithm can be used as first iteration in pattern recognition of dune profile, because it can filter out noise data quickly and produce reliable and fast approximation of riverbed profile. Results from algorithm method then have to be supplemented by visual inspection in order to correct recognized dunes with two peaks.

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RESEARCH ON STANDARDIZED FUNCTIONAL OBJECT OF THE POLDER IN SCALE 1:20

M. Gramblička[1] and M. Orfánus[2]

Abstract
The paper deals with the construction of the physical model of polder Oreské, whose research was conducted at the Department of Hydraulic Engineering at Faculty of Civil Engineering of the Slovak Technical University. After the floods on the river Myjava there were proposed several polders to stabilize the situation. The proposals should prevent the flash floods. As a first one, after the proposals, was selected the polder near the village Oreské. The physical model should give answers to some questions about the hydraulic and operational issues. The facts should then be taken into account in the design and preparation of the other objects in the basin of Myjava.

Keywords
Polder, physical model, functional object, flood protection, Myjava

1 INTRODUCTION

In recent years were the flood situations occurring frequently on rivers in the Slovakia. Their common characteristics were the particularly rapid increase of the discharge with high peaks and not excessively big volume of flood wave as well as a higher number of culminations in the short term. Almost all of them were caused by heavy precipitation activity. Slovak Water Management Enterprise, Banská Štiavnica found the solution in the construction of several

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dry fluvial polders. Their essence is to build a dam across the valley, interrupted in the place of the functional object. Natural landscape with dam retention creates a volume that is during the flood situation filled to reduce the peak discharge in the river bed below the object. Unlike in the past, proposed objects have been designed as polders with unattended functional objects i.e. uncontrolled culvert as well as emergency spillway. Overall, the situation in the upper Myjava basin could be satisfactorily resolved by seven dry fluvial polders whose basic parameters are presented in Tab. 1.

2 GENERAL PARAMETERS OF THE EXAMINED OBJECT

Polder Oreské is created as earth-filled dam with the crest at elevation of 265.00 meters above sea level with a width of 5.0 meters. The dam is designed in profile, which landscape in the valley rises at 255.00 meters above sea level. Normal operating conditions as well as conditions during the flood will be provided by functional object, which will be incorporated into the body of the dam.

Tab. 1 General parameters of polders dry fluvial polders

Name of the polder	Flooded area for maximum water level	Dam height	Volume of the polder for maximum water level	Area of permanent soil occupation
	[ha]	[m]	[10^3 m^3]	[m^2]
Polder Myjava	5,881	4,80	130	1 386
Polder Malejov	16,392	3,65	284	2 445
Polder Podbranč I	20,266	6,50	587	3 448
Polder Podbranč II	18,301	6,50	504	2 800
Polder Prietrž I	43,389	6,50	1 264	4 224
Polder Prietrž II	60,504	4,50	1 442	6 024
Polder Osuské	56,387	6,50	1 033	5 472

Requirements for functional object can be summarized in the following characteristics:

- Without major backwatering in the riverbed of Chvojnica river up to a maximum value Q = 5.0 $m^3.s^{-1}$. This feature will be provided by the bottom culvert of the functional object,
- In the case of increased discharges the village Oreské cannot be flooded by the water from the river Chvojnica. Its capacity was set at Q_{cap} = 9.0 $m^3.s^{-1}$, and this value may be reached in a time when the water level in the polder reaches the edge of the emergency spillway,
- Carry over the “100 year” flood Q_{100} = 21.0 $m^3.s^{-1}$ through the emergency spillway even in the situation when the unregulated bottom culvert will be silted during the flood,
- Provide adequate control of energy of the flowing water through the spillway and culvert in the stilling basin so that the river section below the object will be without any bed distortions.

Modelled object had proposed a bottom culvert of rectangular shape with dimensions H x W = 1.0 x 0.8 m. It was a hole that had its bottom at the bottom of the channel at an elevation of 251.00 m n. m. and the remaining three edges were rounded as cylindrical with a radius of 0.3 m. This opening is part of an emergency spillway, which was located directly above it.

Emergency spillway is in plan view formed as semicircular area with a radius of curvature 6.0 m, which adjoins either side of a straight section with a length of 7.4 m. Spillway edge is at the top of the object, and is formed by a portion of the cylindrical surface of radius r = 0.25 m. The stilling basin was designed as a divergent with divergent side walls at an angle of 3.8°. The stilling basin is deep 3.5 m and ends up on the downstream side slope of which reaches a length of 32 m. The project contemplated with three baffles of trapezoidal shape. Those with its height of 4.0 meters with its upper edge protrude 1.5m above the level of the original riverbed. A plan of the layout has been designed so that the middle was slightly shifted downstream, and the other two are in front of the first one just for the culvert. Flow over the weir section of about 72 m was adapted to the single-slope 4.3% and below the dam to 100 m long with a slope of 0.75%. The channel has a trapezoidal shape with a width of 5.0 m and at slopes of 1: 2.
The calculation of the conversion to the physical model scale was based on the similarity of the curves of high water repetition to allow the model verification the effect of the discharge of more than a hundred year flood, for the profile of Oreské where $Q_{100} = 21.0\ m^3.s^{-1}$.

3 PHYSICAL MODEL

Given the possibility of hydraulic laboratory, we chose to build a model in scale of M = 1: 20. The model itself was built as an area of 10.5 x 3.0 m. To build the model we have built a waterproof pool with glass side walls. The front wall made of the metal was the down part of the tank at the inlet of the model and the back wall was gradually joined to the pool with the water supply. The dimensions of the area used to build the model are represented as 8.450 x 2.650 m. In this area could be an appropriate scale model of the river Chvojnica of the area 169 x 53 m along with part of the dam and its functional object. Dam with a functional object was located at the middle of watertight tank.
Due to the expansion and proposals were used to construct the model by precise shaped brick blocks embedded in concrete. The individual river cross sections were modelled from large-scale plates. Profiles were selected to be able to fit between them and pour concrete blocks. Blocks were embedded in 10 cm thick layers of concrete and were selected to either side were still embedded at least 5 cm of concrete (Fig. 1). The whole model was completed by pouring about 3-5 mm layer of self-levelling compounds for last layer of concrete. Finishing the model has consisted of coating, the entire model, waterproof paint (Fig. 2). The area downstream of functional object was constructed differently (Fig. 3).

Fig. 1 The modelling of the river bed and dam

Fig. 2 Coloured model finishing

Also the gabionade of the functional object was examined as the quarry stone divided into two later modelled tested fractions. 1 aggregate fraction contained 16 to 20 mm after calculating the similarity of the model corresponding to real aggregates from 32 to 40 cm and in aggregate 2 with fraction of 20-26 mm grain diameter corresponding to 40 to 52 cm in reality (Fig. 3).

Fig. 3 Downstream part of the model

4 THE MEASSUREMENTS AT THE PHYSICAL MODEL

Water supply to the model was ensured by pipe with diameter of 200 mm, with discharge regulating fitting. The pipe was flowed into the damping reservoir from which water overflowed through Thomson spillway, which served to control the discharge of the water at the upstream of the model. Pressure sensors mounted at three locations where two at the bottom of the object at the culvert and one at the downstream bottom of the object of the polder. A sensor in the drain served mainly to set lower water levels and according to documents obtained by the in situ measurements. The pressure sensors have been set to the sensitivity of 1 mm in the range of 0-1 m. In addition to the position of the water levels measured at three locations also the velocities were measured by micro-wing. First place was 120 cm from functional object, while this velocity we considered as the inlet velocity and we used it to determine specific energy profile. In extreme discharges were further measured the water velocity at the culvert opening for the determination of the fortifications. Third place was the space at the functional object and dam connection at the upstream. This was mainly measured at the extreme flows. Other observations were recorded on photo or video media, while we focused mainly on monitoring of aeration and vortices at the inlet of the functional object, character of hydraulic jump in stilling basin and stability of the riverbed stone fortifications at the connection to the original riverbed.

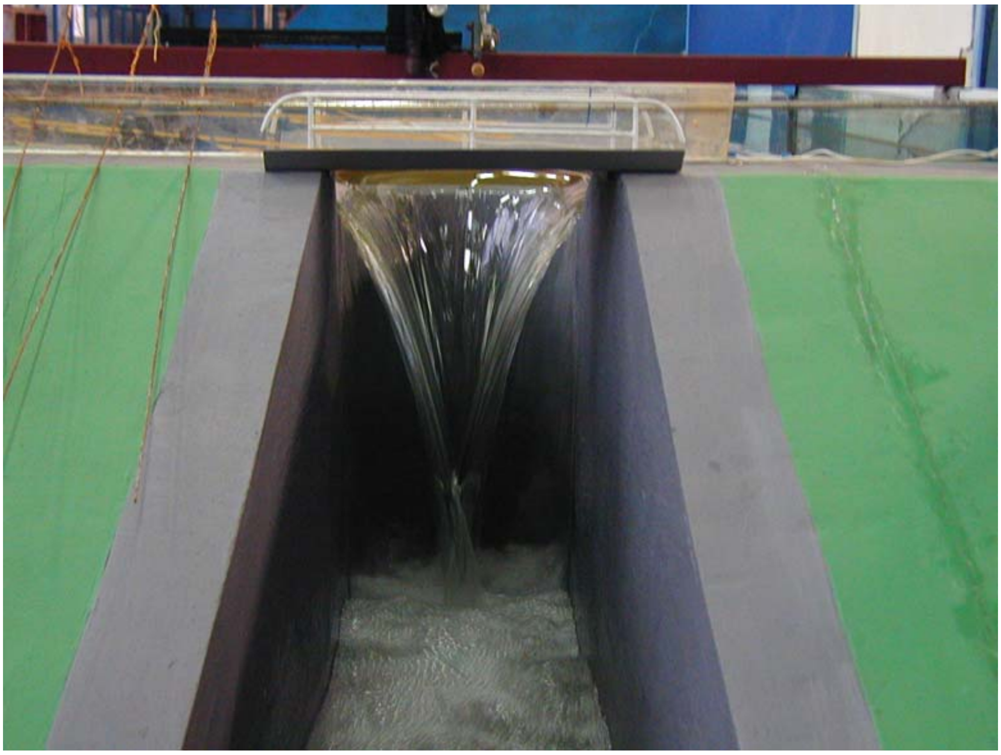

Fig. 4 The measurements at the functional object of the polder

5 COCLUSION

Clearly it can be concluded that the research model was justified and deliver results on the basis of which suggest some modifications to the initial project. The early experiments and observations have demonstrated the unsuitability of location of the functional object itself. The construction was located at a curve, as in the case of smaller discharges cause improper inlet to the culvert as well as the creation of the swirling areas. Recommendation resulted from the research was the displacement of functional object and straightening the river bed at the upstream. Velocity measurement showed that the flood conditions may cause velocities at the culvert over 13 $m.s^{-1}$, which requires adequate protection of concrete. Monitoring the energy decay in stilling basin showed the great functionality in stilling basin with depth of 2.5 m, which is about 1 m less than was proposed in the project. Designed baffles also confirmed its relevance and appropriateness of the deployment. When using an emergency spillway we measured water velocity of about 2.0 ms^{-1}. Whereas these ranges depending on the amount of overflowed water occurred only at a distance of 5.0 m from the object, it is necessary to propose embankment protection on the upstream side in the area. It also confirmed the suitability of the downstream channel protection solutions for stilling basin. Average aggregate of stone at the bottom must not fall below 40-50 cm, because this fraction showed sufficient stability.

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THE EFFICIENCY OF SMALL VARIABLE SPEED PUMPING SYSTEMS

Ivan Halkijević [1], Dražen Vouk [2] and Igor Tadić [3]

Abstract

Pumping water supply systems, especially closed water supply systems are usually characterized by a variation of flow and pressure. Since pump efficiency is a function of flow, for varying demand values a pump can run fairly inefficiently. To deal with this inefficiency the usual way is to change the performance of a pump by applying the pump speed control. For varying system requirements variable frequency drives recently became most common method used to control the motor speed. The equations relating pump performance parameters of flow, head and power to the motor speed are known as the affinity laws. However, affinity laws assume that the efficiency practically does not change with the speed. For most pumps the affinity laws can be used to predict the performance of a pump but only to a certain level, usually not less than 40 to 50 percent of full speed. At lower speeds affinity laws may yield false predictions. The aim of this paper is to verify how well the theoretical aspects work for a small variable speed pump regarding efficiency.

Keywords

Affinity laws, pump efficiency, pump speed control, VFD efficiency

1 INTRODUCTION

Water supply system demand is usually characterized with high variability. For direct pumping system or closed water supply system, pump is usually sized to meet the greatest output demand with the duty point is in the high-efficiency area of the pump. Since the maximum demand occurs in a relatively short period of time during the day, for most of the

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time pump is characterized with lower efficiency. To maintain pump efficiency as high as possible the pump performance must be adjusted. Efficient way to vary the operating point of the pump to match demand is to control the speed of the pump (shaft). The most commonly used device to change pump speed is variable speed drive (VSD). Variable frequency drives (VFDs) are the most common type of variable speed drive that meet varying process requirements by adjusting the frequency and voltage of power supplied to an alternating current (AC) motor. In this manner pump operates over a wide range of speeds. As a result the pump curve is adjusted for different operating conditions with the possibility of maintaining high efficiency.

2 AFFINITY LAWS

The relationships that link the pump characteristics (flow, head, power) operating at different speeds (N_x) are described by the affinity laws. However, affinity laws assume that the efficiency practically does not change with the speed. In the case of speed reduction efficiency curve will be only shifted to the left, Figure 1.

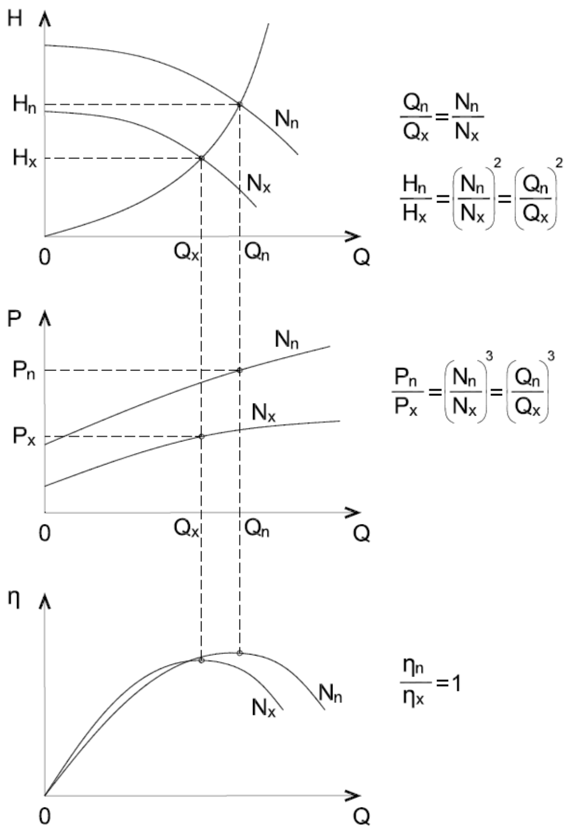

Fig. 1 Affinity equations. The units are arbitrary [8]

Pump efficiency is defined as the ratio of the power delivered on the fluid by the pump in relation to the power supplied to the pump motor. Pump efficiency is a function of the discharge and head and tends to increase with flow rate up to a best efficiency point (peak efficiency) and then declines as flow rises further. Pump efficiency performance is usually supplied by the manufacturer for a given pump.

For a fixed pump speed the operating point is forced to move along the pump curve corresponding to the fixed nominal speed leaving the area of the high efficiency.

Equation (1) given by Sarbu and Borza (1998), [3], deals with an analytical relationship between two different speeds (N_n and N_x) and the corresponding efficiencies (η_n, η_x).

$$\eta_x = 1 - (1 - \eta_n) \cdot \left(\frac{N_n}{N_x} \right)^{0.1} \tag{1}$$

where η_n [%] is the efficiencies for shaft rotational speed N_n [rpm] and η_x [%] is the efficiencies for shaft rotational speed N_x [rpm].

The speed of a pump, N [rpm], in relation to the current frequency and the number of pole pairs is given by a synchronous motor speed formula:

$$N = \frac{120 \cdot f}{p} \tag{2}$$

where f [Hz] is alternating current frequency and p [1] the number of pole pairs.

The base speed of a standard "4-pole" motor for 50 [Hz] current frequency is 1500 [rpm].

Varying the motor speed therefore has a direct effect on the pump's performance. The equations relating pump performance parameters of flow, Q, to motor speed, N, and head, H, and power, P, to motor speed are known as the Affinity Laws, Figure 2:

· Flow is proportional to the motor speed; $Q \alpha N$

· Head is proportional to the square of the motor speed; $H \alpha N^2$

· Power is proportional to the cube of the motor speed; $P \alpha N^3$

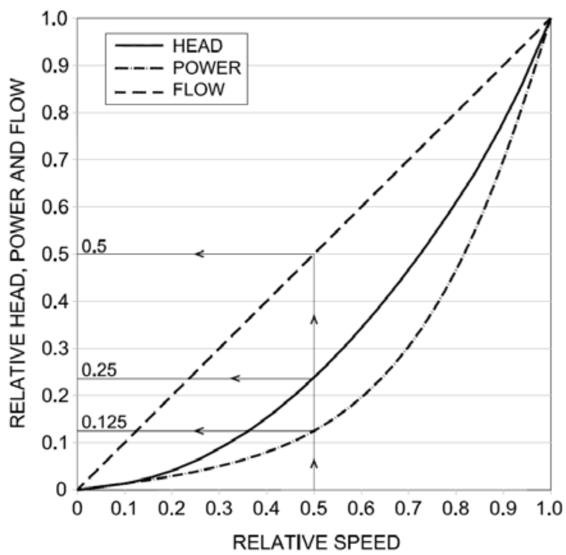

Fig. 2 Affinity Laws [4]

As can be seen doubling the motor speed will increase the power consumption by 8 times. Conversely a small reduction in motor speed will result in a very large reduction in power consumption.

3 VFD PUMPING SYSTEM EFFICIENCY

The total efficiency of a VFD pumping system consists of motor efficiency, pump efficiency and VFD efficiency. This efficiency is also called “wire to water efficiency”.

$$\eta_{total} = \eta_{pump} \cdot \eta_{motor} \cdot \eta_{VFD\ drive} \tag{3}$$

Elements affecting the efficiency are the rotor and stator in the case of the motor and the impeller and volute (diffuser) in the case of the pump. The friction produced by bearings and other mechanical components also affects pump efficiency, but the impeller and volute have the greatest influence. Many medium and larger centrifugal pumps offer efficiencies of 75 [%] to 93 [%] and the smaller ones usually fall into the 50 [%] to 70 [%] range, [1].

Large AC motors can approach an efficiency of 97 [%] and any motor, above 5 [hp], can be designed to achieve the 90 [%] efficiency, [7]. The motor efficiency is generally affected by the use of a VFD and the efficiency of the VFD controller depends on the particular motor.

VFD are not 100 [%] efficient and the efficiency decreases with decreasing motor load [5], [9]. The efficiency decline is more outlined with smaller horsepower drives. Efficiency reduction is particularly outlined among older VFD devices. Later there have been some significant improvements, especially for lower horsepower drives and at the lower load portion of the drive efficiency curve, Figure 3.

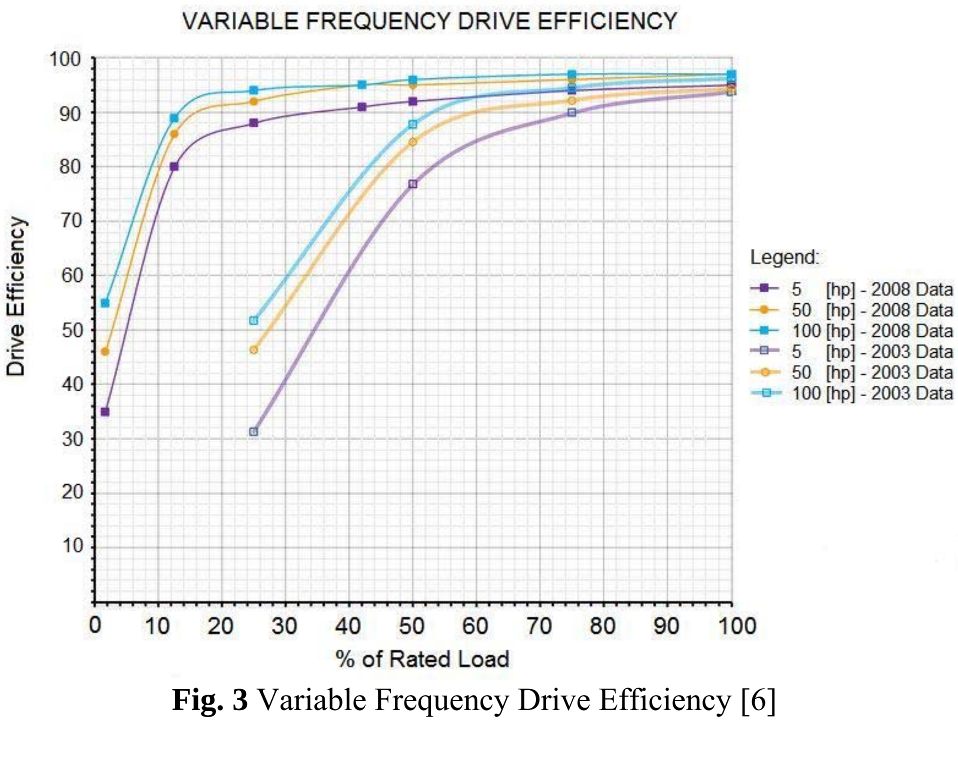

Fig. 3 Variable Frequency Drive Efficiency [6]

4 EXPERIMENTAL PUMPING SYSTEM

The efficiency of a small VFD drive was examined using small-scale centrifugal pump demonstration unit (Armfield FM 20) with integrating wattmeter (SWA1) for variable speed examination. One single-stage pump is driven by close-coupled 180 [W] AC motor. Maximum head is 10 [m], and maximum flow is 1.0 [l/s] at 50 [Hz]. A 2-pole motor has 2880 [rpm]. The unit is equipped with measurement sensors for pressure head across the pump, flow rate, input power, water temperature at the inlet and rotational speed of the impeller, Figure 4.

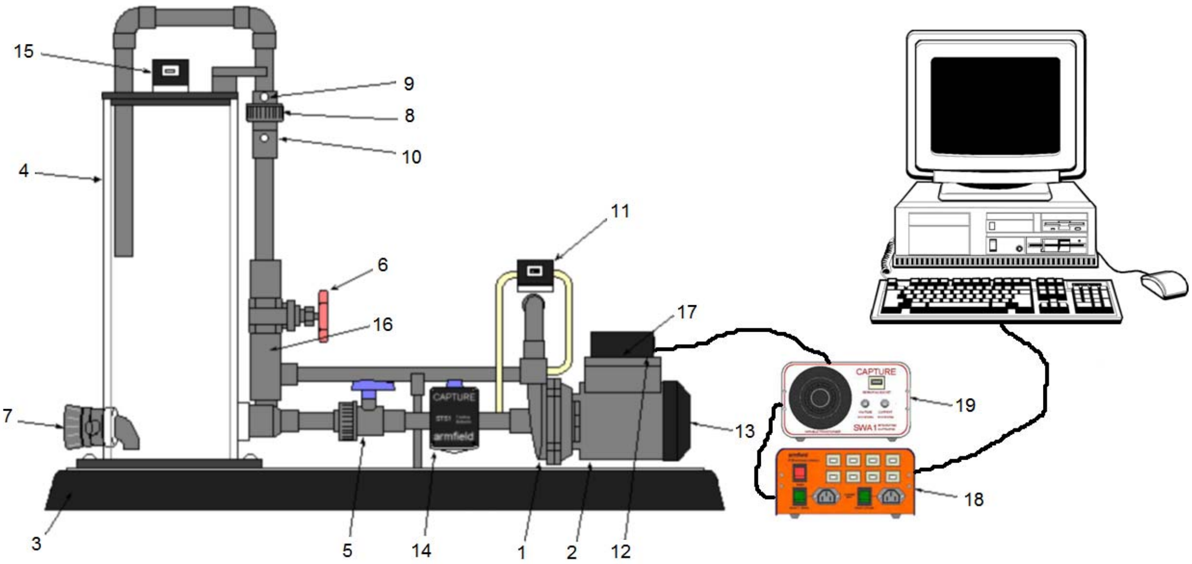

Fig. 4 Armfield FM 20 demonstration unit

The equipment contains centrifugal water pump (1) driven by electric motor (2) which are mounted on a support plinth (3) together with a clear acrylic reservoir (4) and associated interconnecting pipework for continuous circulation. Flow operation is controlled by using the appropriate valves. The inlet (suction) valve (5) controls the flow and pressure entering pump. The flow of water through the centrifugal pumps is regulated by a flow control valve (6) installed in the discharge side (16) of the unit. Adjustment of this valve allows the head/flow produced by the pumps, to be varied. Clean distilled water is used as the operating fluid and a drain valve (7) at the base of the reservoir allows the water to be drained after use.

Appropriate sensors are incorporated on the unit to facilitate analysis of the pump performance when connected to a suitable computer via an interface console. In addition to the tappings lines required by the pressure sensors, additional tappings are included in the pipework at the pressure tapping points to allow appropriate calibration instruments to be connected.

Differential pressure sensor (15) is used to measure pressure developed across the orifice plate (8) installed in the discharge pipework of the pump. The volume flow rate of water can be calculated using this measurement. The sensor is connected to the appropriate tappings in the pipework using flexible tubing (9, 10). Differential pressure sensor is used to measure the difference in pressure between the inlet and outlet of centrifugal pump (11). Rotational speed sensor (12) comprises of a reflective infra-red opto switch (13) used to measure the rotational speed of impeller (motor). A temperature sensor (14) is inserted through the wall of the pipe using a waterproof gland and is used to measure the temperature of the water entering the pump. The motor of the centrifugal pump is connected to the integrating wattmeter and the interface, IFD6 (18), is used to transfer data to a computer by using USB cable.

In addition to the above sensors, which are all permanently attached to the pump unit, an integrated wattmeter, SWA1 (19), is connected to measure the electrical power supplied to the electric motor. The dial on the wattmeter is used to vary the power to the pump. This changes the pump rotational speed and flow rate. The wattmeter is connected between the mains lead (17) from the pump and the power supply. This facilitates the measurement of the electrical power supplied to the motor.

For the input motor power of 260 [W] and output motor power of 180 [W], motor efficiency is 69.2 [%]. The pump output power of 60 [W] results with pump efficiency of 33.3 [%] causing the overall efficiency to be 23.0 [%].The operating characteristics of a pump are shown by plotting head H, power P, and an efficiency E against discharge flow Q. Nominal characteristics of the concerned pumping unit are shown in Figure 5.

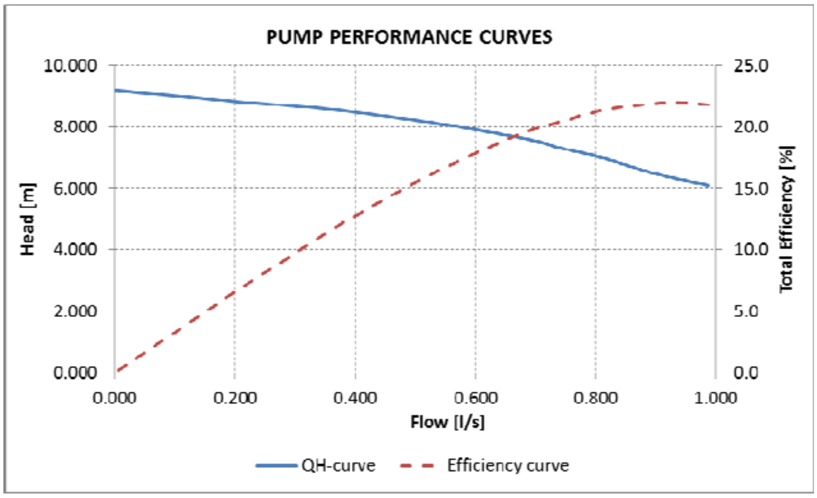

Fig. 5 Pump nominal performance curves

Since the efficiency of the variable frequency drive (SWA1) at lower speeds is not known due to the different operating mode, its efficiency will be determined relative to the full speed efficiency.

Measurements were carried out over a wide range of flows and speeds. At first, the head curve and the efficiency curve for the pump running at full speed were developed. In the next step, the pump speed is adjusted to a new, lower value at which the measurements were carried out. According to the obtained results for the new, lower speed, new set of curves were developed. These steps are repeated for each 10 [%] speed reduction as long as the sensors were able to read data.

The overall results are shown in Figure 6.

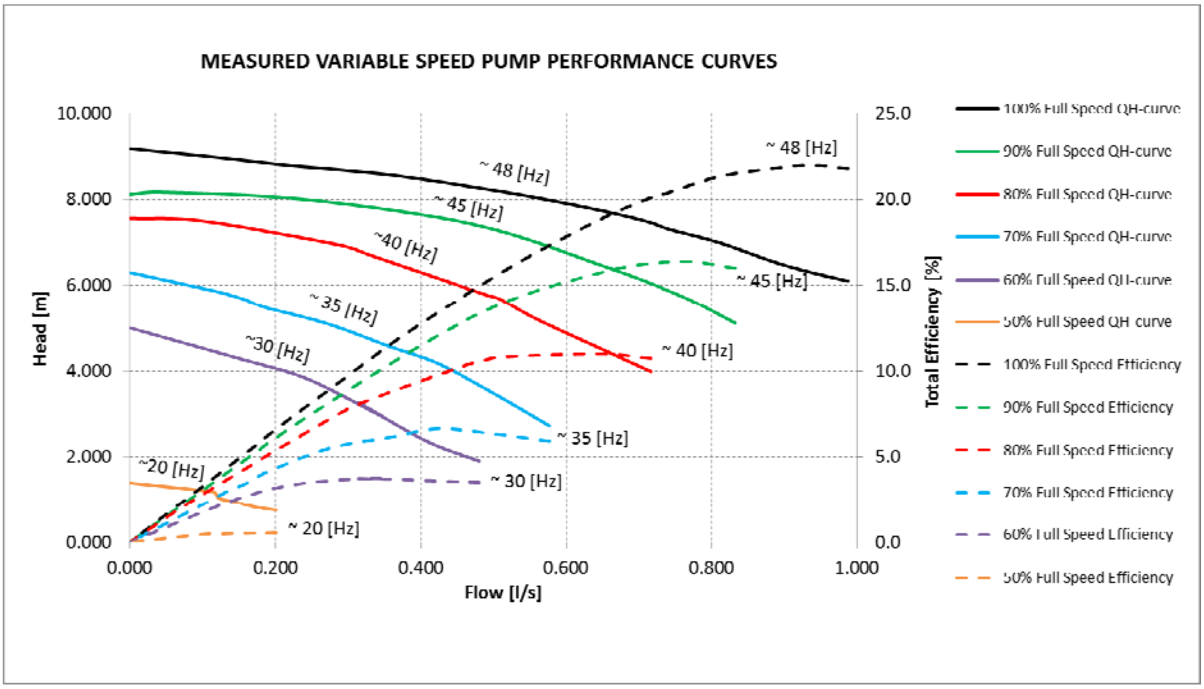

Fig. 6 Measured pump performance curves at different speeds

Using the affinity laws and Sarbu - Borza equation pump heads and efficiencies curves were developed for lower pump speeds. Theoretical and measured heads and efficiencies for different percentages of full speed are compared in Figure 7.

In general, the pump head curves generated using the pump affinity laws agreed with the experimental data. Slightly larger difference is evident at 50 [%] of full speed which is accounted for affinity laws accuracy decline for low pump loads. While the head curves behaved in accordance with the affinity laws, the pump efficiency curves did not. As the speed decreased from full speed the efficiency differs more increasingly from the theoretical behavior.

This difference is accounted in total efficiency equation as the motor and the VFD drive efficiency loss. Since the affinity laws do not account efficiency change and due to the lack of data, motor efficiency change was neglected. The loss of efficiency was attributed to the variable speed drive. The effect of speed change on VFD drive efficiency is shown in Figure 8. The VFD drive efficiency was obtained as the ratio of the efficiency measured divided by the efficiency predicted by theory.

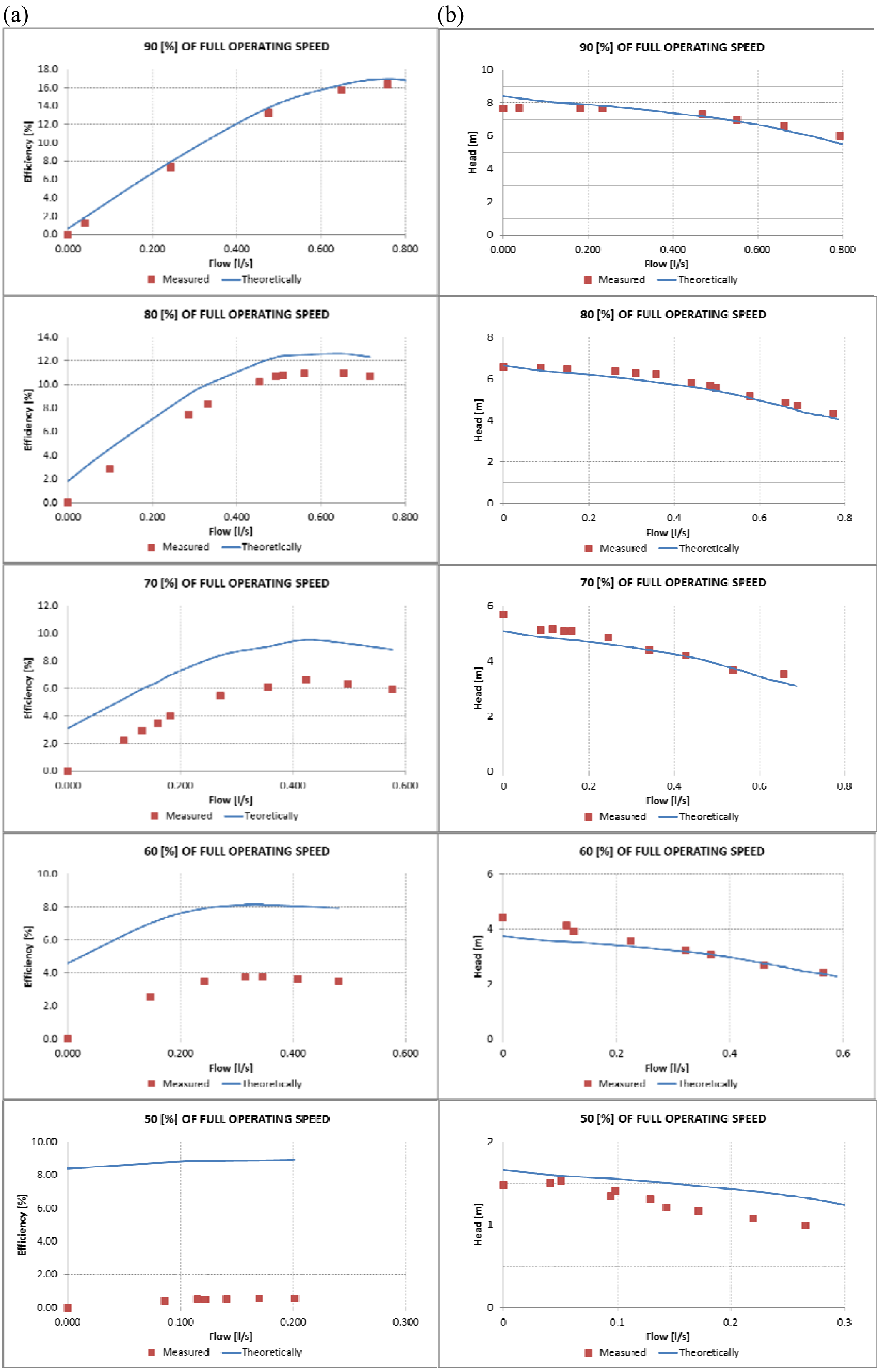

Fig. 7 Comparison of measured and theoretical values
(a) Efficiency comparison; (b) Head comparison

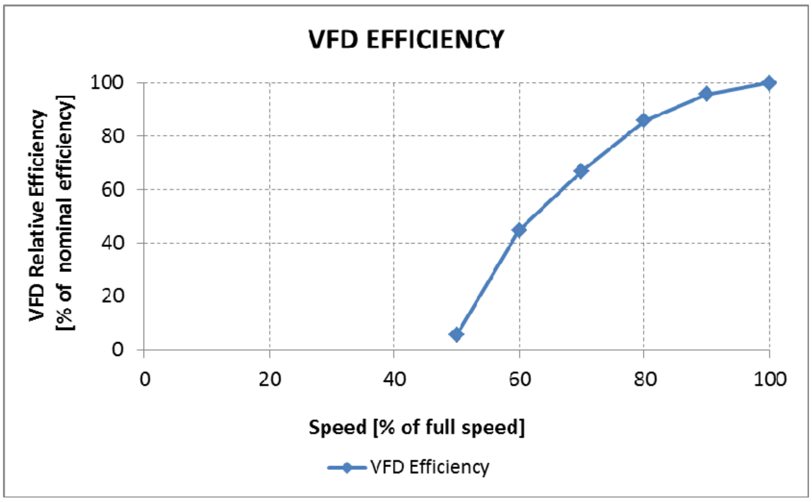

Fig. 8 Measured VFD efficiency at different speeds

5 CONCLUSION

This research was based on a single small and low power pump with small VFD unit and cannot be used to indict all variable speed pumping systems as inefficient. It confirmed that a variable speed controlled pump with a VFD drive does not maintain its efficiency over a wide range of conditions. It also confirmed that low power VFD drives lose efficiency at much higher loads than high power VFD drives. Some differences between predicted and measured head occurred at 50 [%] of full speed as stated at the beginning of the article that at lower speeds affinity laws may yield false predictions.

Since variable speed drive manufacturers do not readily provide data on loss of efficiency as the drives deviate from loads that result in peak efficiency, it can be concluded that the variable frequency drive efficiency depends on the VFD type, size and on the speed reduction, [2].

Motor efficiency depends not only on motor design, but also on the types and quantity of active materials used. The efficiency can therefore vary considerably from manufacturer to manufacturer, [7].

The affinity laws give a good approximation of how pump performance curves change with speed but in order to obtain the actual performance of the pump in a system, the system curve also has to be taken into account.

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CFD ANALYSIS OF THE INLET ZONE OF A SETTLING TANK AT THE WWTP HUMENNÉ

Michal Holubec [1] and Štefan Stanko [2]

Abstract

The paper is focused on CFD modeling (Computational fluid dynamics) of a primary settling tank at the WWTP in municipality of Humenné. The primary sedimentation tanks are structures of mechanical treatment, before biological activation. The settling tanks are of rectangular shape, which is a common sight at WWTPs in Slovakia. To achieve the most effective operation concerning the proper technological processes and cost saving, we need to find the best solution for hydraulic conditions in these facilities. The CFD offers solutions without the need of building a physical model.

Keywords

CFD, Settling tank, WWTP, Sedimentation

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1 INTRODUCTION

Computational Fluid Dynamics (CFD) is an interdisciplinary science which connects the fields of mathematics and physics and through computer sciences. The basic notion is to simulate complex engineering problems through mathematical models solved by high performance computers. The main benefits of this technology are, of course, the reduction of cost for most experiments and experimental design compared to physical modelling and lower time demand which results in faster production cycles in the industry and more effective research. A large part of the sanitary engineering field consists of wastewater transportation and treatment, which both offer a wide range of problems that could be solved more efficiently with the utilization of CFD technologies. There is a lot of waste water treatment plants (WWTP) in Slovakia which have problems with hydraulics in their technological processes, even after reconstruction. This paper is a continuation of research on CFD analysis of a wastewater settling tank located on a WWTP in the municipality of Humenné, Eastern Slovakia (1).

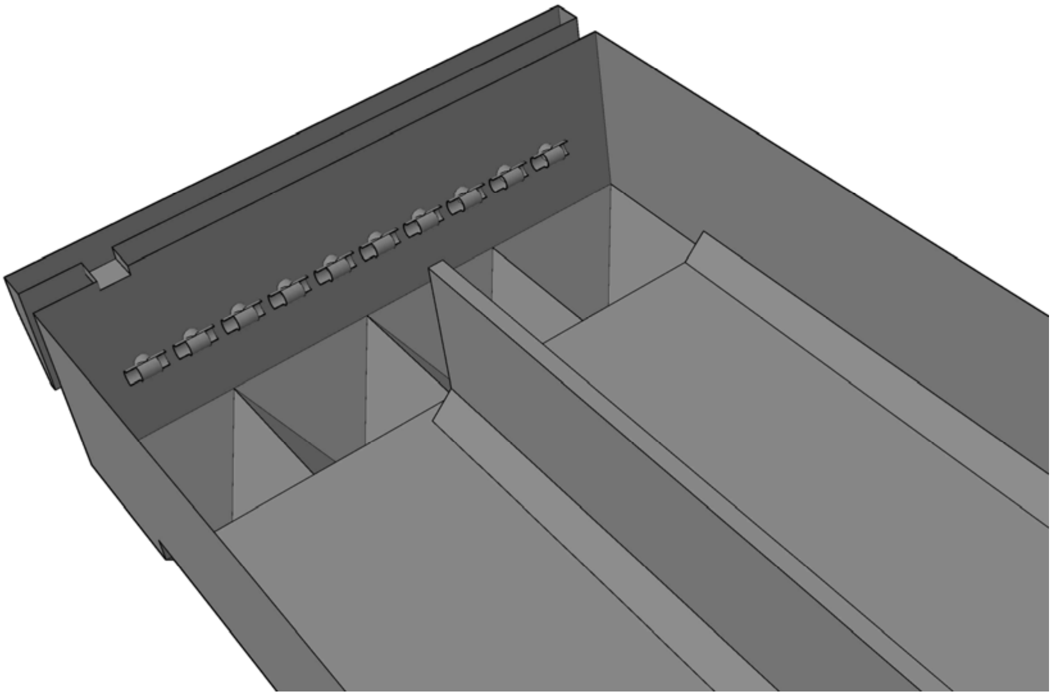

Figure 1 View of the inlet zone of the tank

The wastewater treatment plant is designed to process wastewater from 96700 equivalent residents and works in two stages – mechanical treatment and biological treatment. The wastewater is transported via a combined sewer system, with maximum inflow of 1050 l.s-1 (1922 l.s-1 during storm events). Storm water tanks and the storm water treatment process line are in place to deal with storm water inflow. The object of this analysis is a primary sedimentation tank. It is a rectangular tank with horizontal flow. The wastewater is transported from the grit chambers through a concrete conduit, perpendicular to the flow direction in the tank. The water flows from the inflow chamber through ten T-shaped steel pipes DN300, which are located on the front wall of the tank (Figure 1), 1622 mm above the

tank bed. There is also a small, 600 mm wide floodgate in the same wall. There are 4 holes for the extraction of the sludge at the bottom of the tank in the inlet zone. The tank is 36 meters long and 12 meters wide, the maximal height of water surface is 3,72 meters, according to the project documentation. The volume of the tank is divided by a 3,55 meters tall and 300 mm thick concrete wall which is situated in the middle of the tank. The tank is equipped with a moving bridge with sludge scrapers. The sludge is hauled into the holes at the inlet zone and pumped out of the tank with sludge pumps. The treated water flows out of the tank through a weir at the far end. The schematics of the tank are shown in Figure 2, Figure 3 and Figure 4.
The CFD analysis suggested negative impact of the shape and direction of inflow on overall flow in the tank. This paper focuses on a follow-up simulation of the inlet zone, with the intent to propose a possible solution.

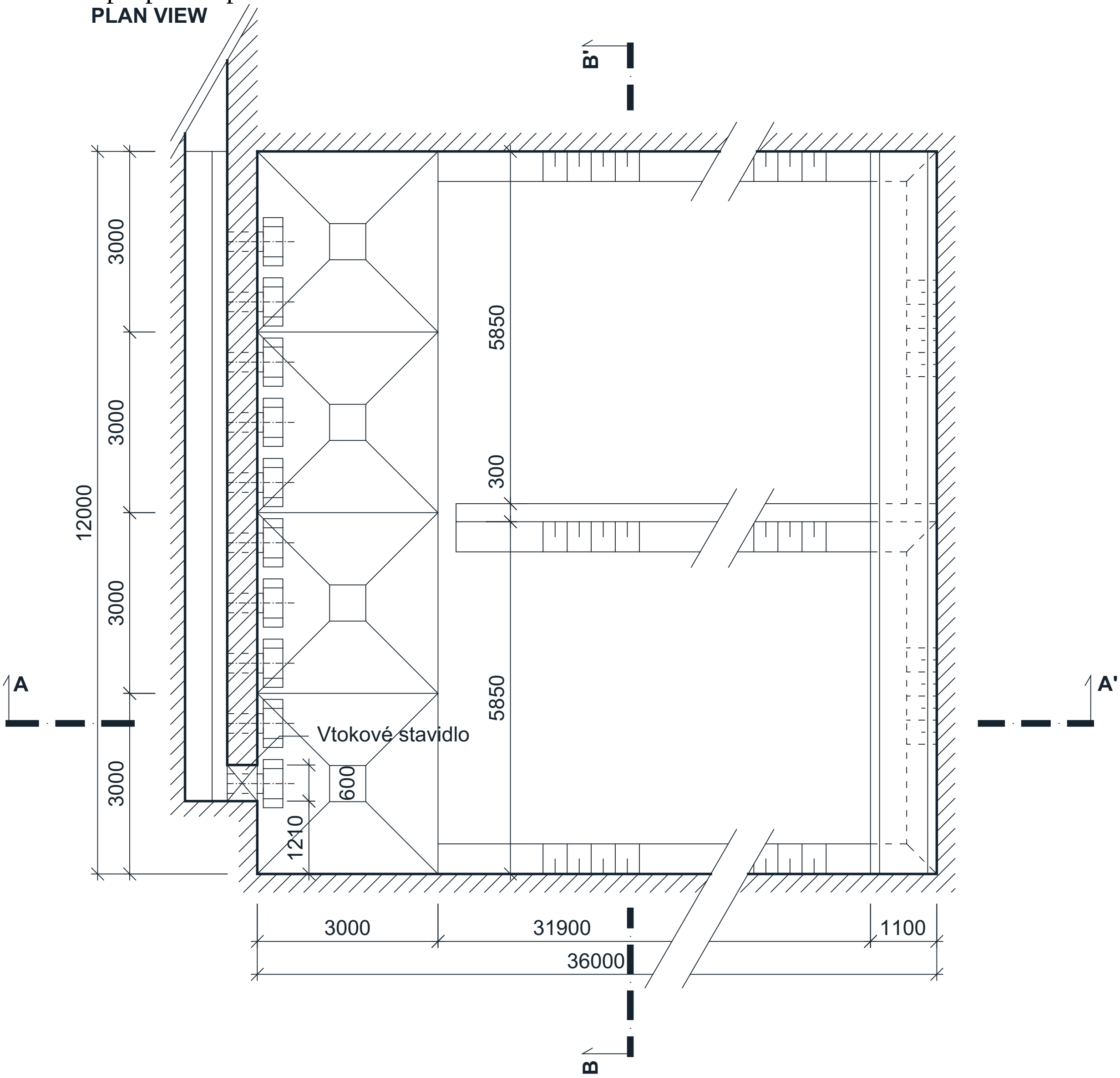

Figure 2 Plan view of the settling tank

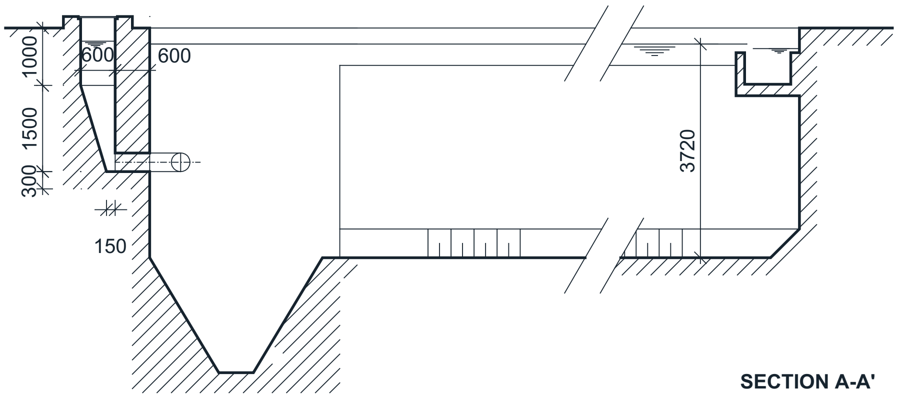

Figure 3 Section A-A’

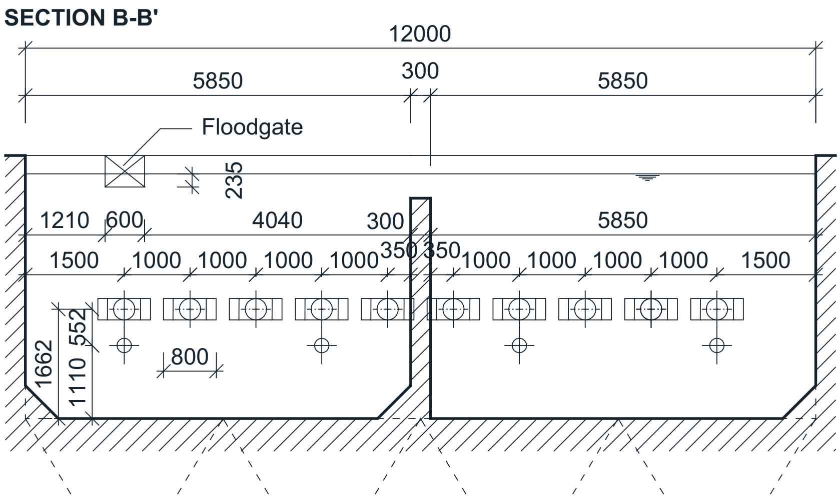

Figure 4 Section B-B’

2 METHODS

The problem was simulated in 3D space, with a zero shear stress wall condition to simulate surface of the fluid. This was done mainly due to conserve time and computational capacity. The height of the surface plane is determined based on field measurements at the site, and so was the flow rate/velocity at the inlet boundary. The inlet boundary was defined with a steady velocity of 0,369 $m.s^{-1}$. At this stage, only the inlet part of the tank was modeled, with an outflow boundary condition at the far end. There were two types of walls with different roughness coefficients to simulate differences between the concrete tank and the steel pipes. The problem was simulated with „pure water”, meaning without sediment transportation. K-omega turbulence model was used for its precision with low Reynolds number turbulent flows and relatively confined spaces in the area of interest. The whole simulation was done using applications from the ANSYS CFD software package – ICEM CFD for meshing, FLUENT for computations and CFD POST for post processing. The geometry was created in AutoCAD. For the purpose of this research, two different variants of the inlet zone were simulated. The meshes for these variants can be seen in Figure 5 and Figure 6 and their composition is summarized in Table 1.

Table 1 Mesh composition

Variant	Total Elements	Total Nodes	Triangles	Tetrahedron
Actual tank	733024	126260	46098	682004
Modification	437544	75898	30106	404763

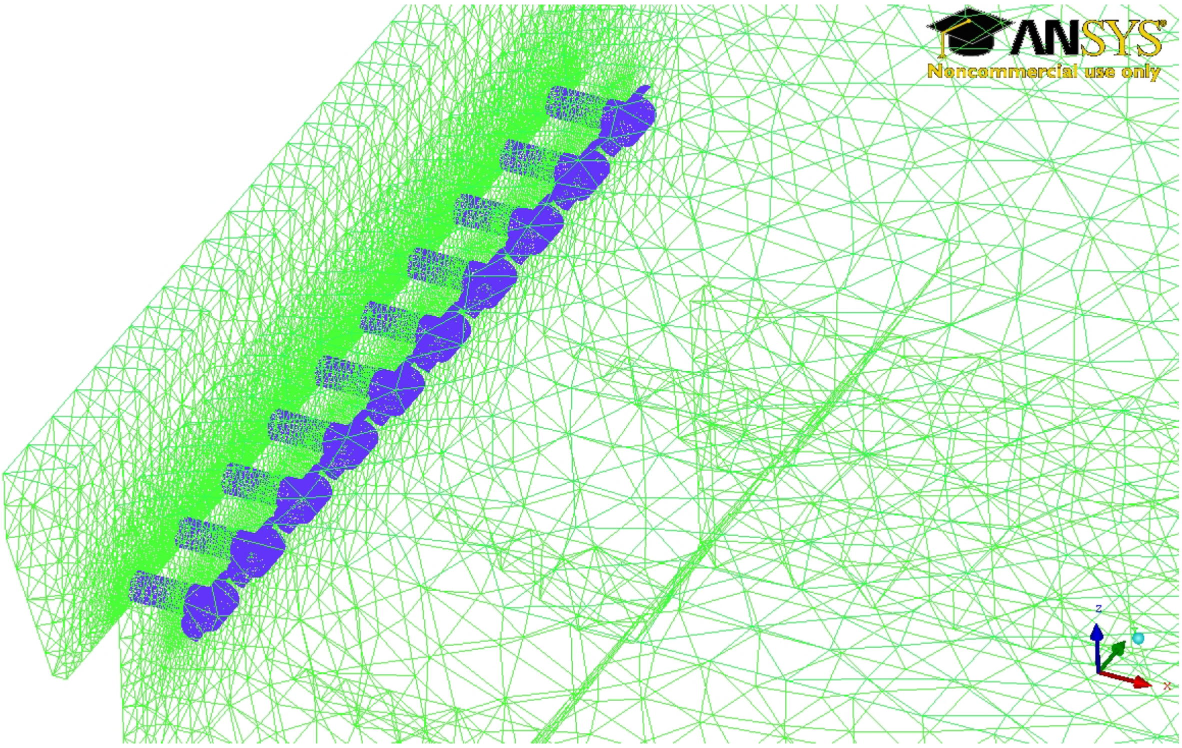

Figure 5 Inlet zone mesh for simulation of the actual settling tank

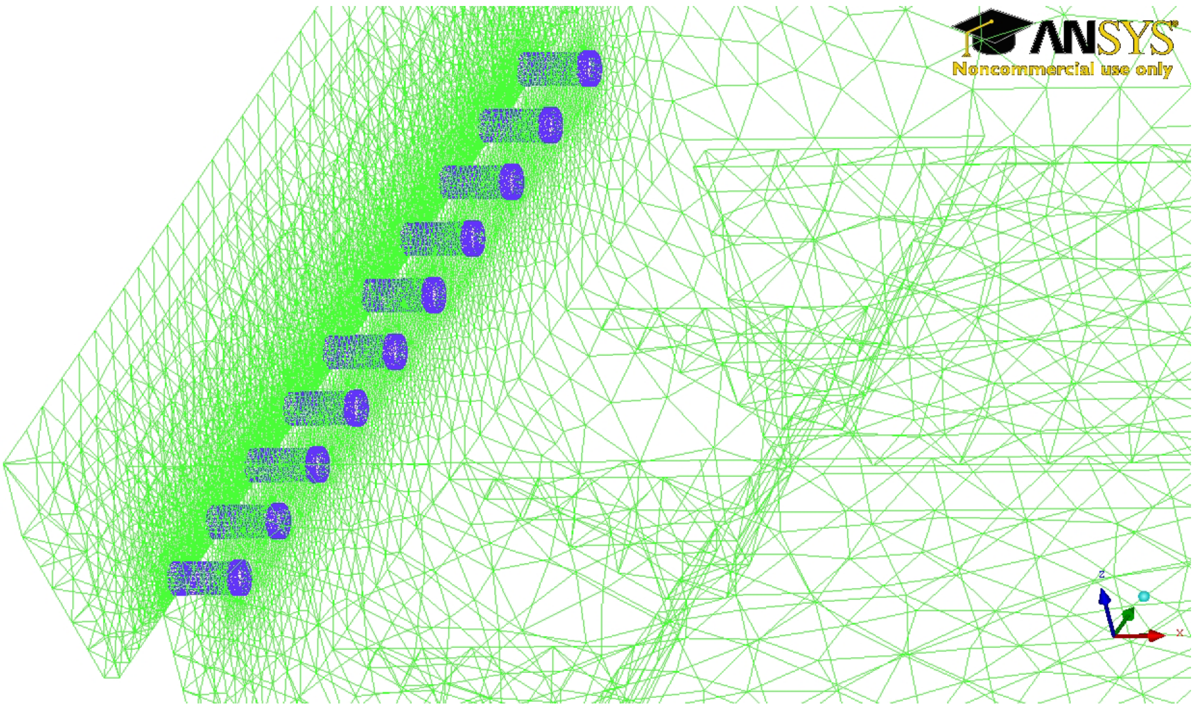

Figure 6 Inlet zone mesh for simulation of the proposed modification

3 RESULTS

Results of the simulation of the first variant can be seen in Figure 7 and Figure 8. We can observe strong tendencies of the flow towards the right side of the tank, caused by the perpendicular inflow channel. Figure 8 hints at possible operation problems caused by high velocities in the sludge pits, especially on the right side of the tank. This could lead to resuspension of collected sludge.

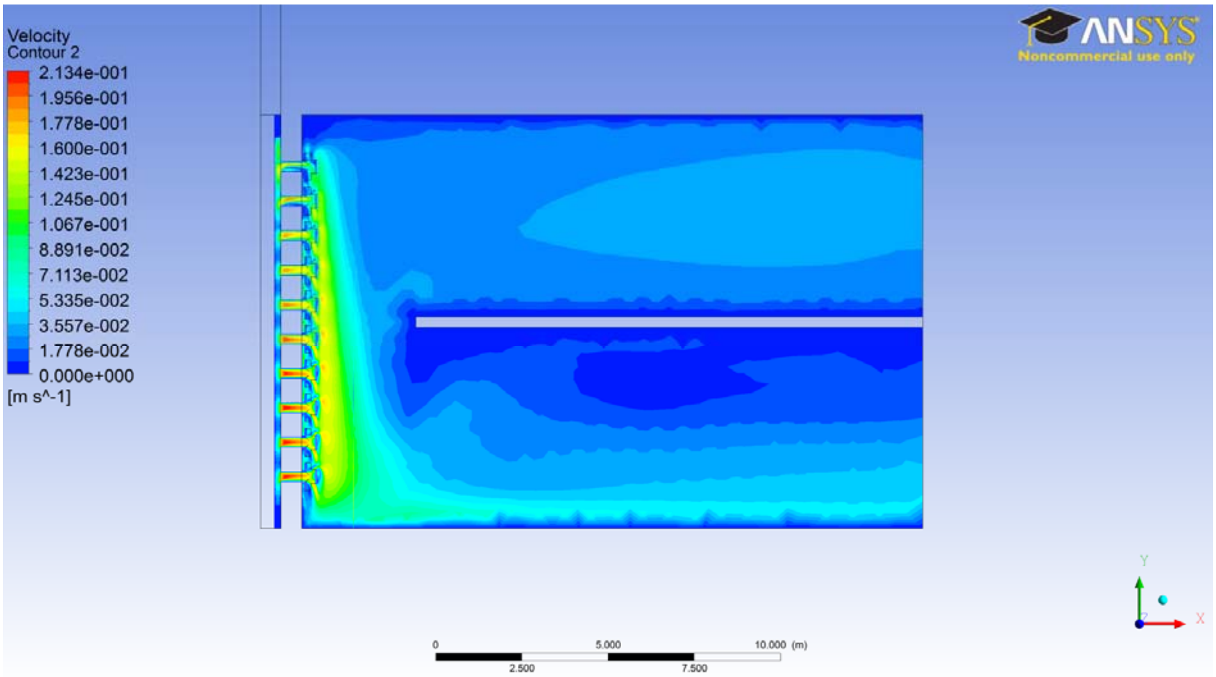

Figure 7 Velocity contours - 1st Variant (actual state)

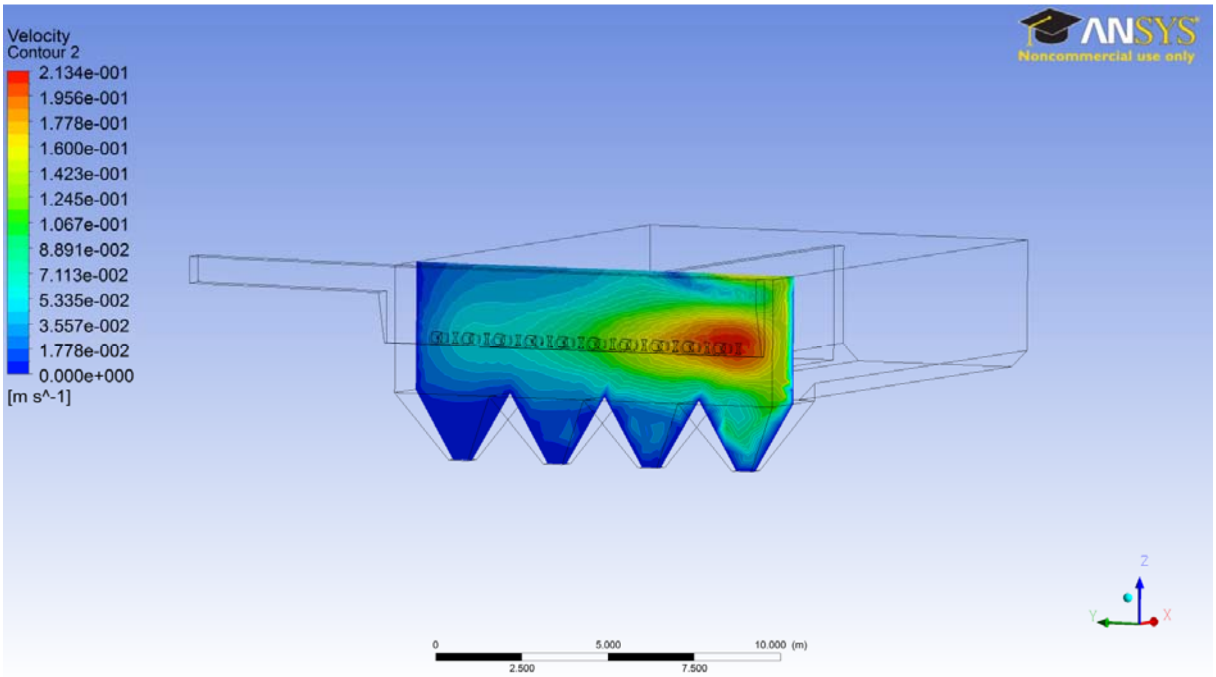

Figure 8 Velocity contours in the sludge pits - 1st Variant (actual state)

The results from the second simulation (Figure 9, Figure 10) show improvement in certain area. The strain to the right side is still present, but overall, the velocities are lower compared to the first variant. Figure 10 shows that the velocities in the sludge pits are more favourable.

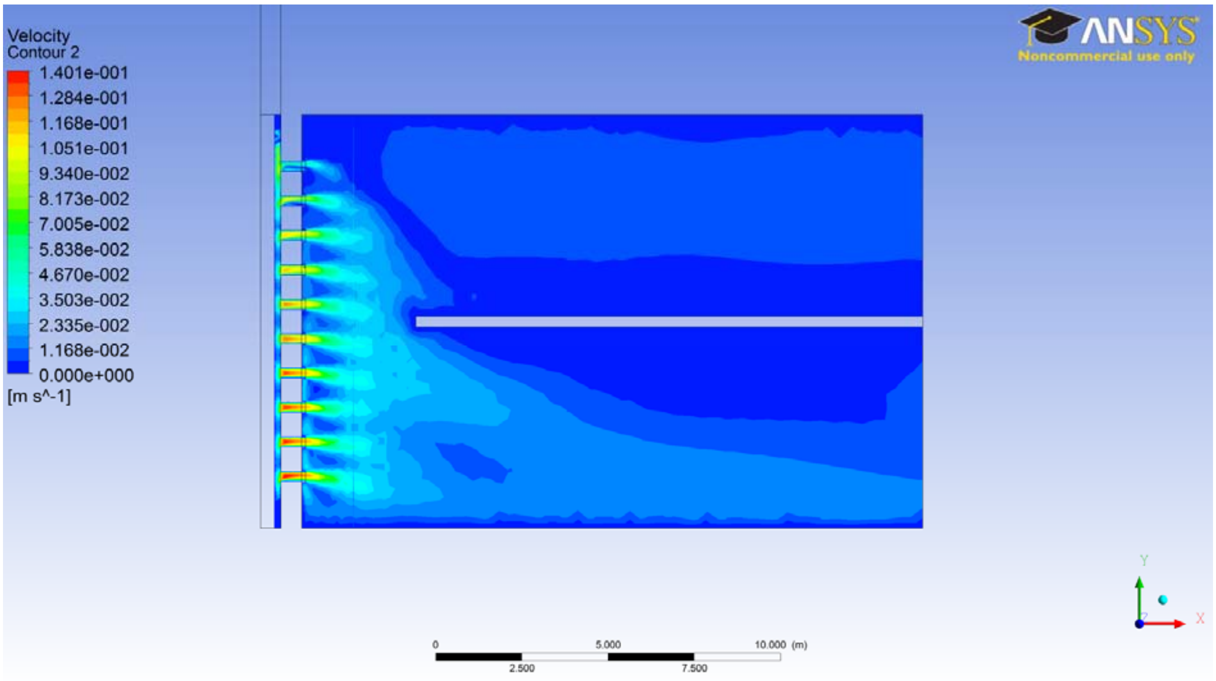

Figure 9 Velocity contours – 2nd Variant (proposed modification)

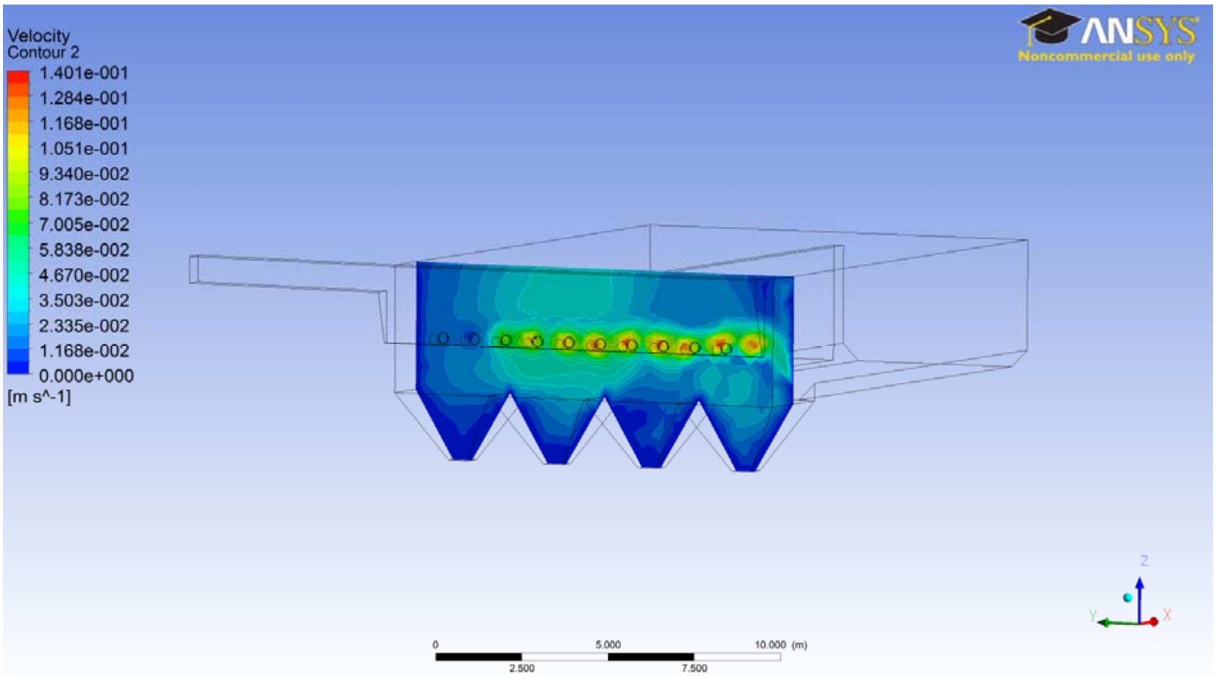

Figure 10 Velocity contours in the sludge pits - 2nd Variant (proposed modification)

4 CONCLUSION

The simulation of hydraulics of wastewater treatment settling tank. The settling tank in WWTP Humenné was selected Based on the survey of operators of wastewater treatment plants and field measurements were also performed at the site. (1)

The results show that simple pipes used in the second simulation could possibly contribute to better operation as opposed to the T shaped deflectors used in the settling tank. However, as these simulations were performed using certain simplifications (namely, without a multiphase model to simulate wastewater) another simulation series with particle transport and sedimentation would be needed to get more conclusive results.

5 ACKNOWLEDFMENT

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HYDRODYNAMIC SIMULATION OF FLOODING SCENARIOS FOR CRISIS MANAGEMENT IN PRAGUE

M. Mišík[1], J. Bajčan[2], P. Sklenář[3] and M. Kučera[4]

Abstract

Important part of flood protection measures in city Prague are mobile flood barriers along banks of river Vltava. Eventual failure of mobile flood barriers could cause flooding of vulnerable urban areas. It was decided that contingency planning and crisis management for such situations will be prepared on the basis of numerical simulation of flood protection failure scenarios. Critical places of flood protection hypothetical failures were assessed and unfavourable discharge and water level scenarios were prepared. Flooding of most vulnerable urban areas was simulated by detail 2D hydrodynamic unsteady flow modelling. On selected localities were defined also scenarios of flooding through sewer system man holes. Results of simulations were presented in form of maps showing flooding extent, flooding depth, water surface elevations, flow velocity magnitudes and directions, as well as by text description of flooding situation, all in selected time steps. Video animations showing flooding evolution in space and time were created. All results were elaborated to form of interactive graphical application which helps planners and crisis managers on city level.

Keywords

Flood mapping, flood protection mobile barriers, flood protection failure, 2D hydrodynamic modelling

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1 INTRODUCTION

Important part of flood protection measures in city Prague are mobile flood barriers along banks of river Vltava. Eventual failure of mobile flood barriers could cause flooding of vulnerable urban areas. It was decided that contingency planning and crisis management for such situations will be prepared on the basis of numerical simulation of flood protection failure scenarios.

2 DEFINITION OF CRITICAL LOCALITIES AND FLOODING SCENARIOS

Critical localities of possible failures of flood protection structures were defined. Selected were places with highest elevation of flood protection structure crest above terrain and localities where failure could lead to most unfavourable scenarios. Dimensions of failure opening in protection line were defined based on technical assumptions and type of structure (mobile barriers, earth dike).

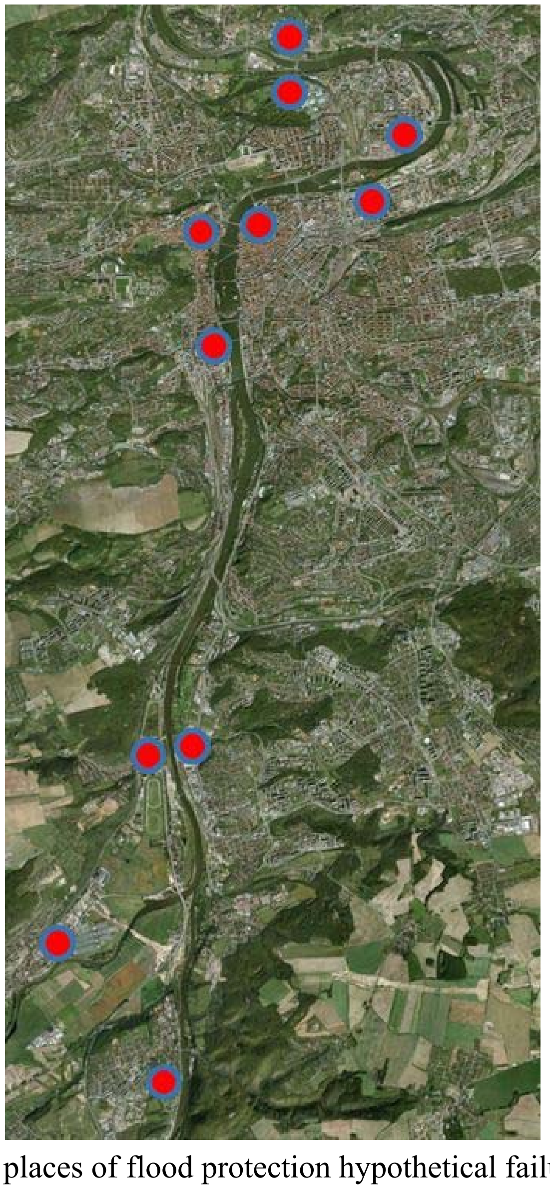

Fig. 1 Critical places of flood protection hypothetical failures in Prague

Fig. 2 One of the selected critical places with high mobile barriers – Na Kampě

3 NUMERICAL SIMULATIONS

Flooding of most vulnerable urban areas was simulated by detail 2D hydrodynamic unsteady flow modelling. Modelling tool MIKE 21 FM was applied. Detail 2D computational mesh of models combined triangular and quadrangular elements, followed directions of streets and roads and covered areas of interest in detail. Planar positions of buildings were excluded from models. Topography of the areas in models was defined on the basis of actual high resolution Digital Elevation Model.

Inflow discharge through simulated openings in flood protection lines was defined by model on the basis of water level boundary conditions in Vltava River. On selected localities were defined also scenarios of flooding through sewer system man holes. Discharge of water through man holes was defined as point source time series inflow defined on the basis of sewer system hydraulic calculations.

To achieve the required results, it was necessary to prepare and run together 33 simulations of flooding scenarios within 11 individual areas. Handling of needed computational effort in reasonable time was supported by parallel computing.

4 APPLICATION OF RESULTS AND CRISIS MANAGEMENT TOOL

Results of numerical simulation of flooding scenarios were processed. For each of the simulated scenarios were in selected time steps prepared following outputs:

- Maps of water surface elevation, water depth and flow velocities in form of ESRI grids with 2 x 2 m resolution
- Extend of flooding in form of ESRI polygon SHP file
- Vector arrows of flow velocities in form of ESRI point SHP file
- Video animations of flooding evolution in time and space
- Text descriptions of flood protection failures and flooding processes

The above mentioned model results and GIS layers will be elaborated into an interactive crisis management viewer, from which user will be able to open various maps, video animations, text descriptions and relevant technical drawings.

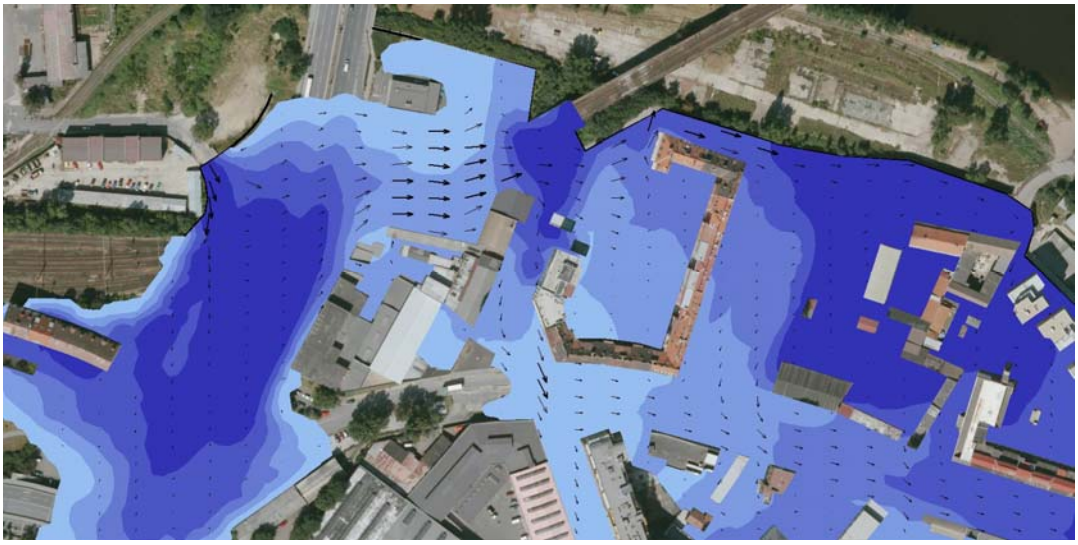

Fig. 3 Simulated depths of flooding and flow directions in one of the model areas

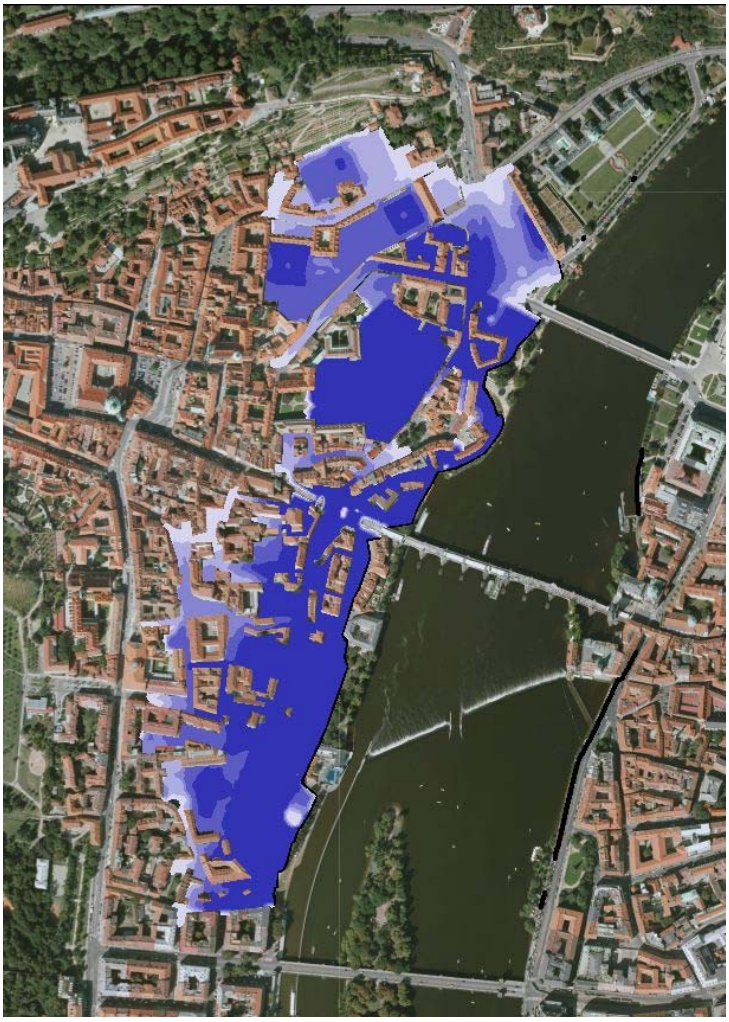

Fig. 4 Simulated flooding of part of historical centre

5 CONCLUSION

Flood protection measures and structures should not be regarded as 100 % reliable. They can fail in consequence of various reasons. It can be overload of structure design parameters, technical failure or other destruction. Potential damages in such situations can be significantly reduced by adequate preparedness and preventive measures. For such purpose can be adequately used results of numerical simulations of crisis scenarios.

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KOLÁROVO WATER STRUCTURE – PREPARATION OF THE RESEARCH AND DESIGN PROPOSALS

Ľ. Možiešik[1], P. Šulek[2] and M. Orfánus[3]

Abstract

In 2012 was within the Research and Development Program of the Ministry of Transport, Construction and Regional Development of the Slovak Republic realized the study of research, design and implementation of the Water Structure Kolárovo on the river Váh. The objectives of the preparatory study was to particularly identify suitable locality of the water structure, determine the composition of the hydraulic objects, determine hydraulic and hydrological parameters of the structure, determine the parameters of the operation performance and determine the workload and methodology of further detailed research for optimal design parameters and performance standards.
This article briefly summarizes the results of the preparatory study on the research and design work.

Keywords

Kolárovo Water structure, preparatory study, river Váh

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1 KOLÁROVO WATER STRUCTURE AS A CONDITION OF NAVIGABILITY OF THE LOWER VÁH RIVER

The original concept of navigability of lower Váh River envisaged the backwatering of the lower Váh river by the Nagymaros Water Structure on the Danube river, which should ensure a backwater at the level of the reservoir 107.83 m a.s.l. during the average discharges. This backwatering should intervence the Selice Water Structure on the river Váh at river kilometer 43.900, while the Selice Water Structure and the Králová Water Structure (rkm 63.150) were designed to meet the conditions of navigation depths at least of 3.5 meters for minimal operational level. Situation of lower Váh River is shown in Fig. 1.
The concept of navigability of the lower Váh River with Nagymaros Water Structure in operation was not carried out by known reasons. As a consequence are the following facts:

1. There are currently at the lower Váh River unrealistic conditions for navigation of large vessels and in some sections also by small vessels so far (sections between Selice Water Structure and river kilometer 32).
2. Unless the Danube River backwater in the section downstream from Gabčíkovo Water Structure will not be achieved, the navigational depths of 3.5 m cannot be ensured on the Slovakia-Hungarian section of the Danube River as well as on whole lower Váh River.
3. In terms without backwater is possible to provide navigational depth of 2.5 m at low control and navigation water level of the Danube River in Komárno by excavating the river bed in section Komárno - Kolárovo (rkm about 25 of Váh River).
4. In the section of the Váh River between the Selice Water Structure and Kolárovo (rkm 43,900 - about 25) it is possible to provide conditions for navigation, i.e. necessary parameters of the fairway, otherwise than by backwatering the river (Fig. 2).

Follows from the above that the navigability of th Váh River in section Komárno - Selice Water Structure is necessary to design and implement a hydraulic structure (navigation grade) in the area of Kolárovo city.

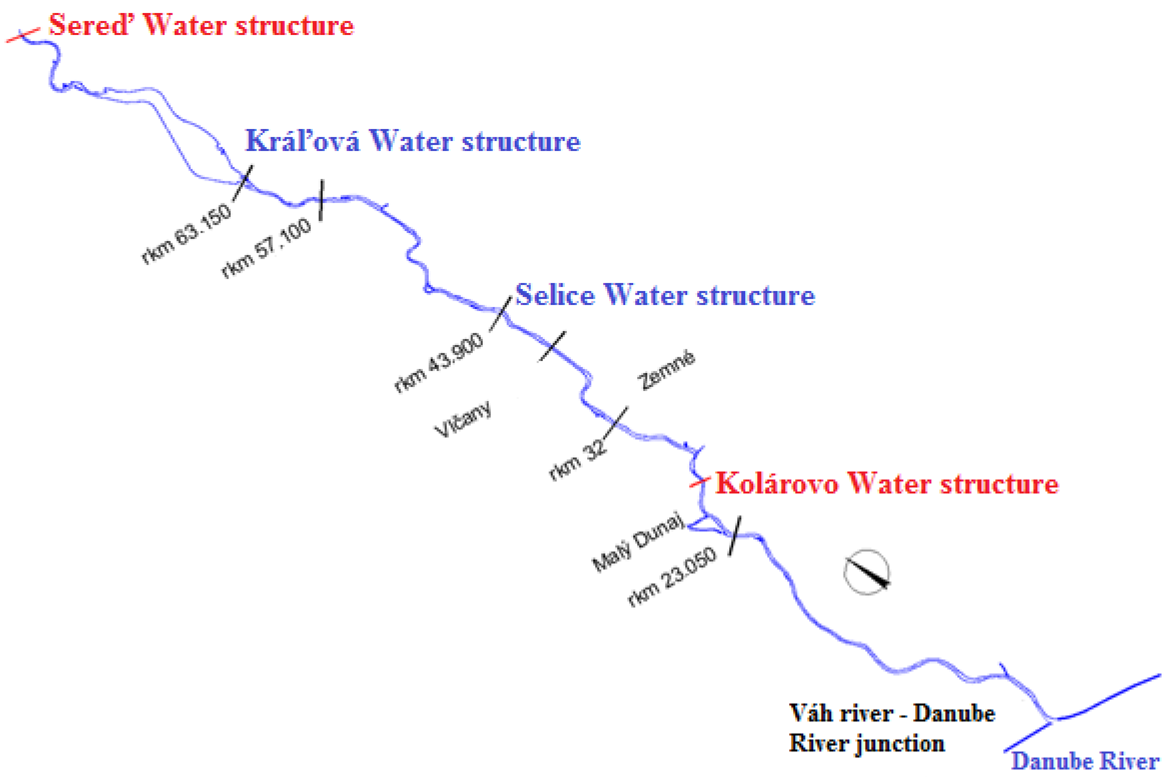

Fig. 1: The lower Váh River locality

Purposes of the Kolárovo Water Structure in the field of navigation are as follows:

1. Allow the opening a continuous waterway navigation on the Váh River unlimited by low discharges
2. Ensure minimal operating water level of about 107.80 to 111.00 m a.s.l. (aim of the research), in which it is possible to provide parameters of fairway in section Selice Water Structure - Kolárovo Water Structure, especially nautical depth of 3.5 m.
3. Ensure the backwater in the rkm of Nitra River relocation, thus allowing navigability of Nitra River upstream to the Nové Zámky city

The Kolárovo Water Structure cannot be in terms of ensuring the fairway parameters of the Váh waterway full refund of the Nagymaros Water Structure; it does not affect the water level regime in section Kolárovo - Nitra. It can be designed so that it provides better utilization of the balance rkm section 43.900 (Selice Water Structure) – The Kolárovo Water Structure in terms of the parameters of the fairway and in terms of energy use and allow partial navigability of Nitra.

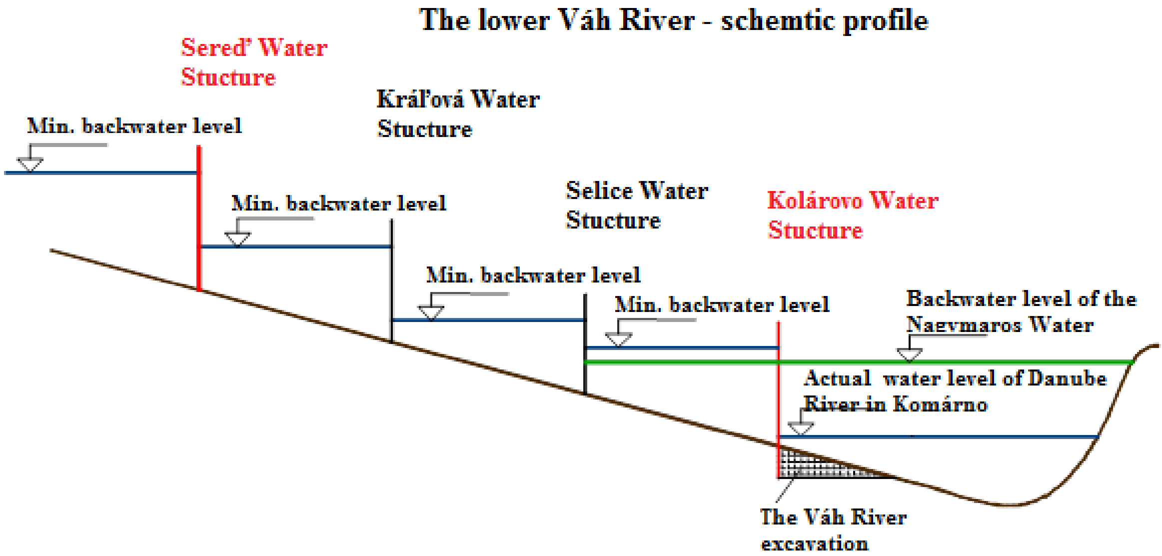

Fig. 2: The lower Váh River - schematic profile

2 THE NEEDS OF FLOOD PROTECTION AND ENVIRONMENTAL PROTECTION

The Kolárovo Water Structure should be designed to satisfy the conditions relating to the ability to convert dimensioning discharge without the water left the inundation area. The measurements and calculations of the Water Research Institute (WRI) [2] in 2002 shows that the combination of high discharges in the Danube River and Vah River in some sections of the flood protecting dykes have currently insufficient height. Levels at these discharges are so high that the best solution appears to be overflowing water structure, i.e. water structure, which is of extreme flows for the most part submerged. Even combined discharges of $Q_{average}$ in the Danube River and Vah River discharge Q_{2R} will reach the water level at rkm 25 Kolárovo of 112 m a.s.l. [2]. The situation with the transfer of high discharges can significantly improve the excavated river bed of the Váh River in section of Kolárovo Water Structure - Nitra, which is necessary, to reach navigable depths of 2.5 m, in conditions without backwater. This excavation will have a multiplier effect - of navigation, flood and help to increase the capacity of Kolárovo Water Structure weir by increasing the gradient of lowered position at the downstream.

For the project of the Váh River Waterway has been processed a study of the environmental impact, so called EIA, in the late 90 years. Regardless of whether it will be processed by an updated version, it is clear that damming the section above Kolárovo, for environmental conditions it can only help.

The current status of the discharges under Kráľová Water Structure in rkm 63.15 in regular daily operation (outside flood extremes) limited by use of hydropower regulation of water structure. This operation is characterized by considerable fluctuations in discharges in the range of Q_{min} = Q_{BIO} (minimum biological discharge passed through Kráľová Water Structure) ÷ Q_{max} = Q_{peak} (max. capacity of the turbines in Kráľová Water Structure) and consequently the water levels as well. When the discharges $Q_{BIO} = 7\ m^3.s^{-1}$ passes the water structures Kráľová and Selice singly, the water level downstream from Selice Water Structure is extremely low. The section of rkm 43.900 – 32 of the Váh River is in time of non-energetic operation practically without water which is expressly the state of environment threatening (Fig. 3) [6].

Fig. 3: Photo of Váh River nerar the Zemné in time of non-energetic operation [6]

The water level fluctuation is in this section during energy peak extreme and unnatural to the environment. The water level regime at Zemník during different modes of operation of the Kráľová Water Structure is shown in Fig. 4 [6]. Fig. 4 shows that within a few hours the water level reaches the fluctuation range of 1.5 m and the velocity increases considerably.
The backwatering of this section should cause the positive effect only. Especially for habitat directly linked with the hydrological and hydraulic regime of the Váh River.
Together with positive effects it is necessary to verify the backwater effect on ground-water regime, especially in the areas of villages Komoča, Zemné, Vlčany, Neded.

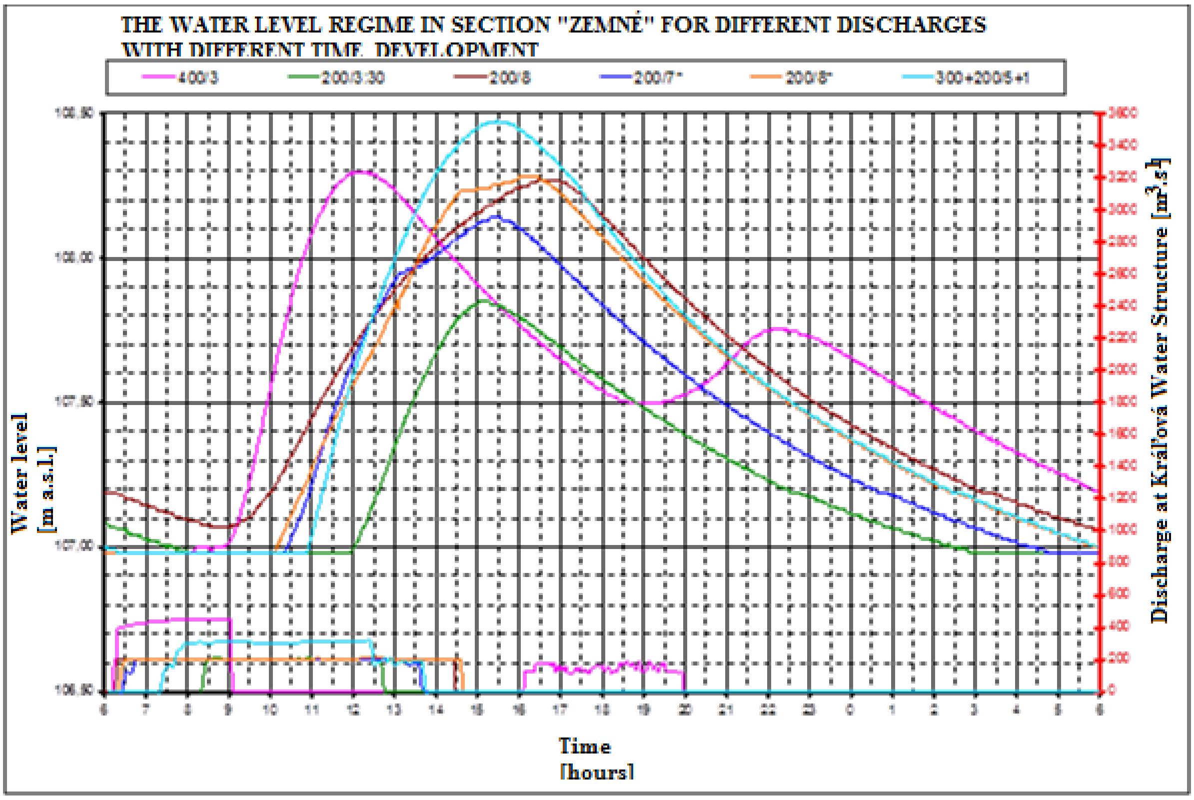

Fig. 4: The water level fluctuation in section of Zemník during the energetic peak operation of the Kráľová Water Structure [6]

3 KOLÁROVO WATER STRUCTURE LOCALITY

In preparation study were proposed three variants of Kolárovo Water Structure locality (Fig. 5.)

Every variant got its own positives as well as negatives. It is important the rigorous advisement, especially, of navigation point of view (comparison of excavation works with effect on the coherent objects – for example the bridge in Kolárovo) and environmental point of view (ground-water regime).

4 THE PURPOSE OF KOLÁROVO WATER STRUCTURE AND OBJECT COMPOSITION

The resulting proposal of Kolárovo Water Structure decides, among other factors, the definition of all purposes of water structure and its parts.

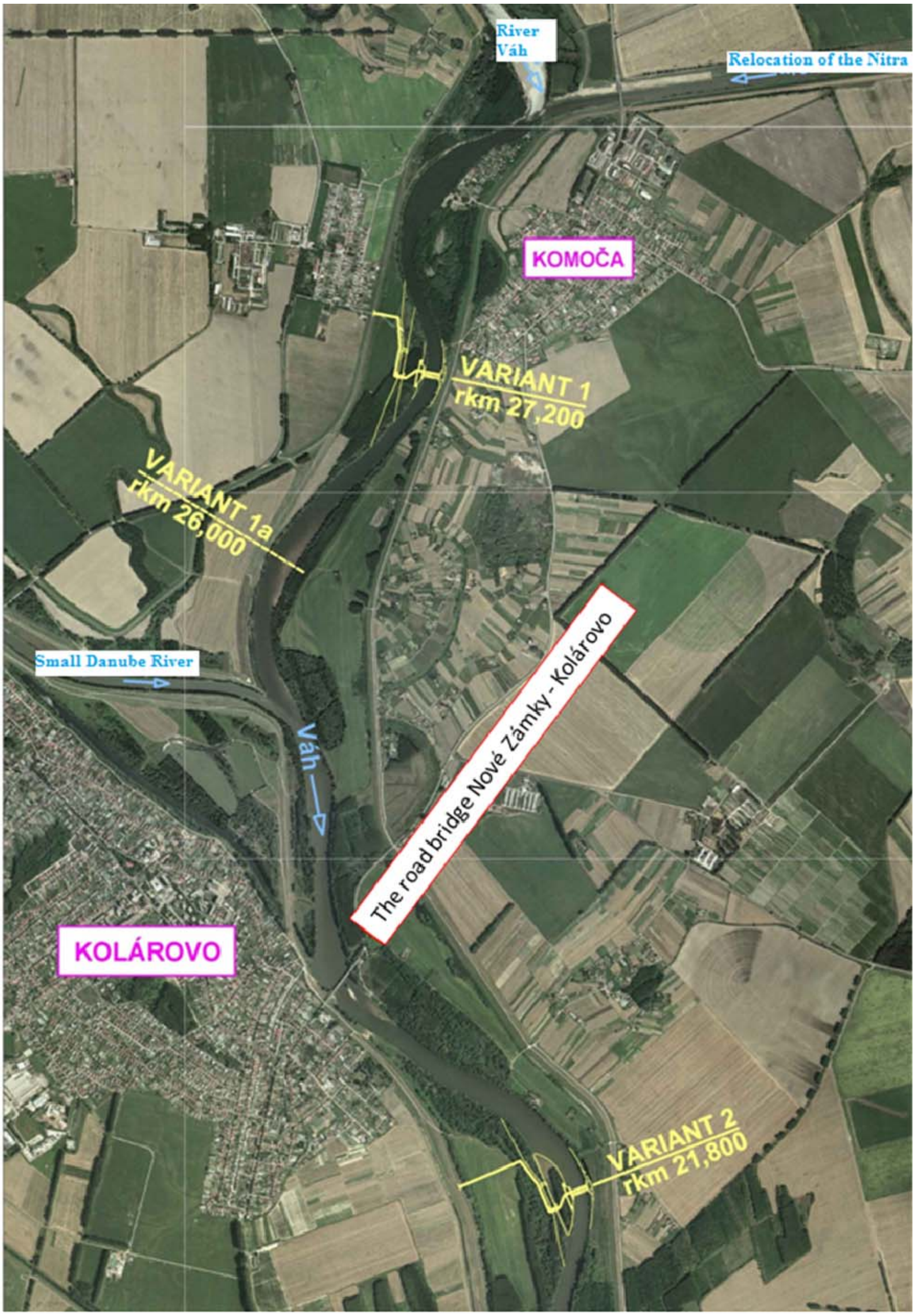

Fig. 5: The Kolárovo Water Structure variants [1]

The Kolárovo Water Structure has above all

1. ensure the performance of the water-way in the Váh River in rkm section 43.950 - Kolárovo Water Structure and in section of the Nové Zámky - Nitra for full range of discharges
2. improve environmental conditions for the habitat of Váh channel especially at low discharges in section Selice Water Structure (rkm 43.950) - Kolárovo Water Structure

These effects are possible to achieve by the hydraulic structure only, e.g. weir
Secondary effect should be

3. the use of water energy by utilization of discharges in hydro-power plant. According to upstream water level position and graphs of possible reachable heads of water it is necessary to manage if the hydro-power plant will utilize only the Q_{BIO} + tributary rivers or discharge up to maximum capacity of Kráľová Water Structure hydro-power plant, e.g.420 m^3s^{-1} increased of the Nitra River discharges.

The part of the object composition is so called "caused objects". The Kolárovo Water Structure in order to fulfill its functions with the best operational parameters, must be included the ship lock and bio-corridor respectively the fish passes. Based on the analysis it is necessary to decide whether the object part of the Kolárovo Water Structure to overcome the head of the water by small vessels (estimated by the density of the movement of small vessels and navigational safety assessment ratios) and what type of object it will be (small ship lock, ship-way). Due to geological structure of the riverbed of the lower Váh River section in the foreseeable implementation (fine sand) is necessary to assess the need for building the sediment outlet as a part of water structure.
In some reasonable conditions (discharge, water head, habitat) it is possible to combine the bio-corridor and ship-way into one object.
It follows that the included objects should be weir, large ship lock and bio-corridor. Objects that can be part of structure are hydro-power plant (variants - big capacity of discharge 420 m^3s^{-1} + tributaries), a small hydro-power plant with capacity of Q_{BIO} + tributaries), a small ship lock respectively water-way (water-way with possibility to combine with the bio-corridor), sediment outlet.

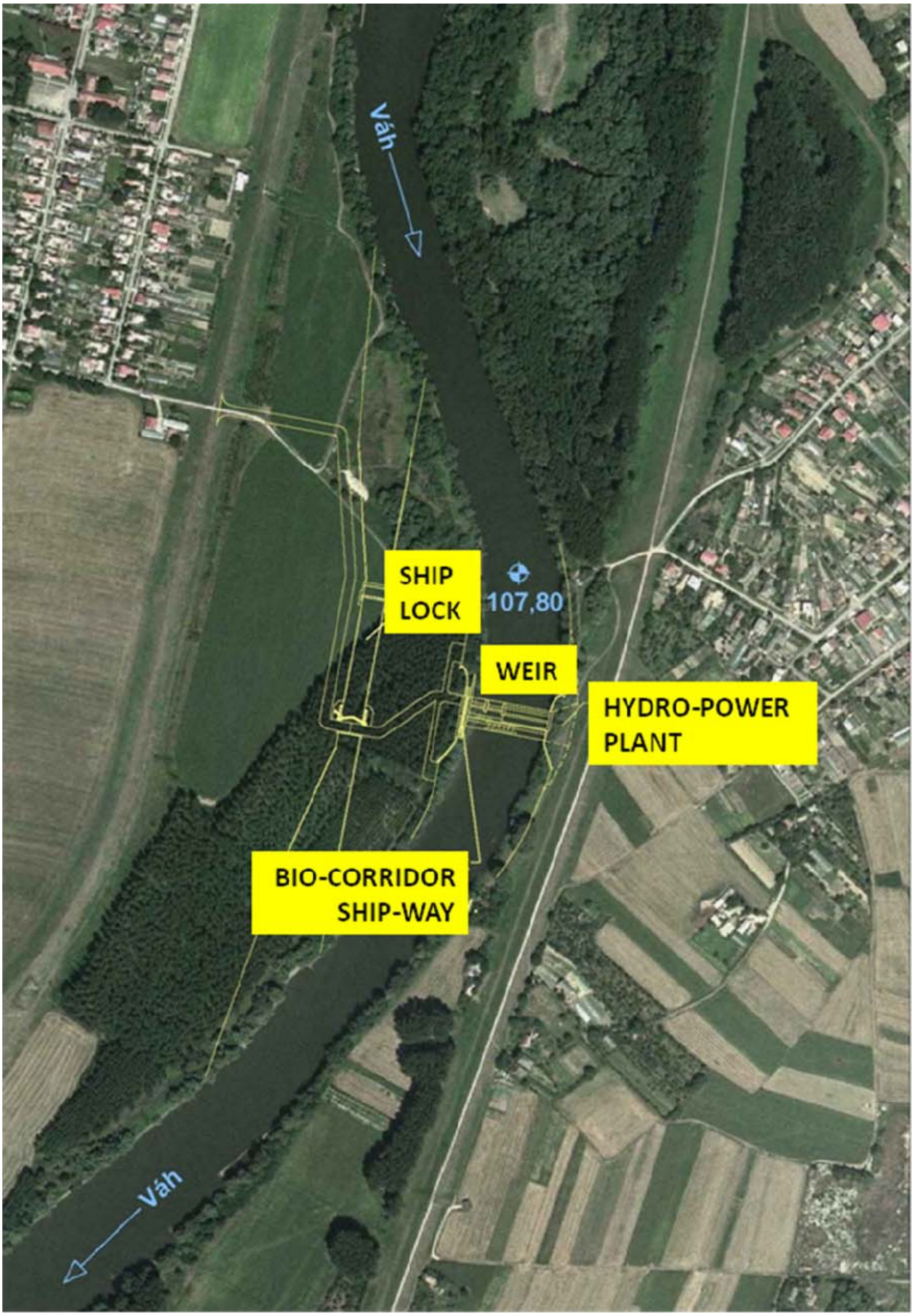

Fig. 6: Variant 1 situation [1]

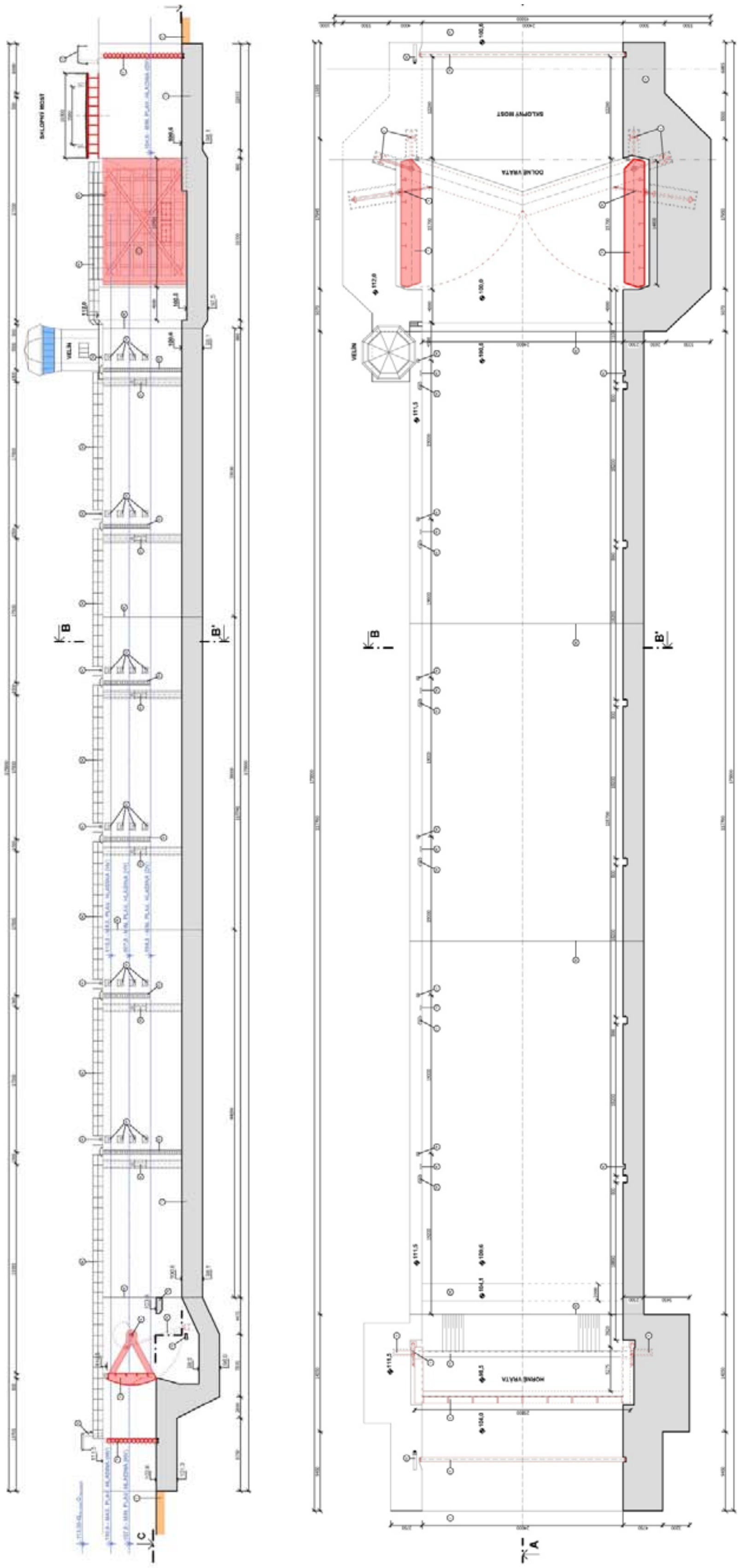

Fig. 7: Kolárovo Water structure - The plan and section of ship lock (variant 1) [1]

Fig. 8 Kolárovo Water structure - The plan of weir (variant 1) [1]

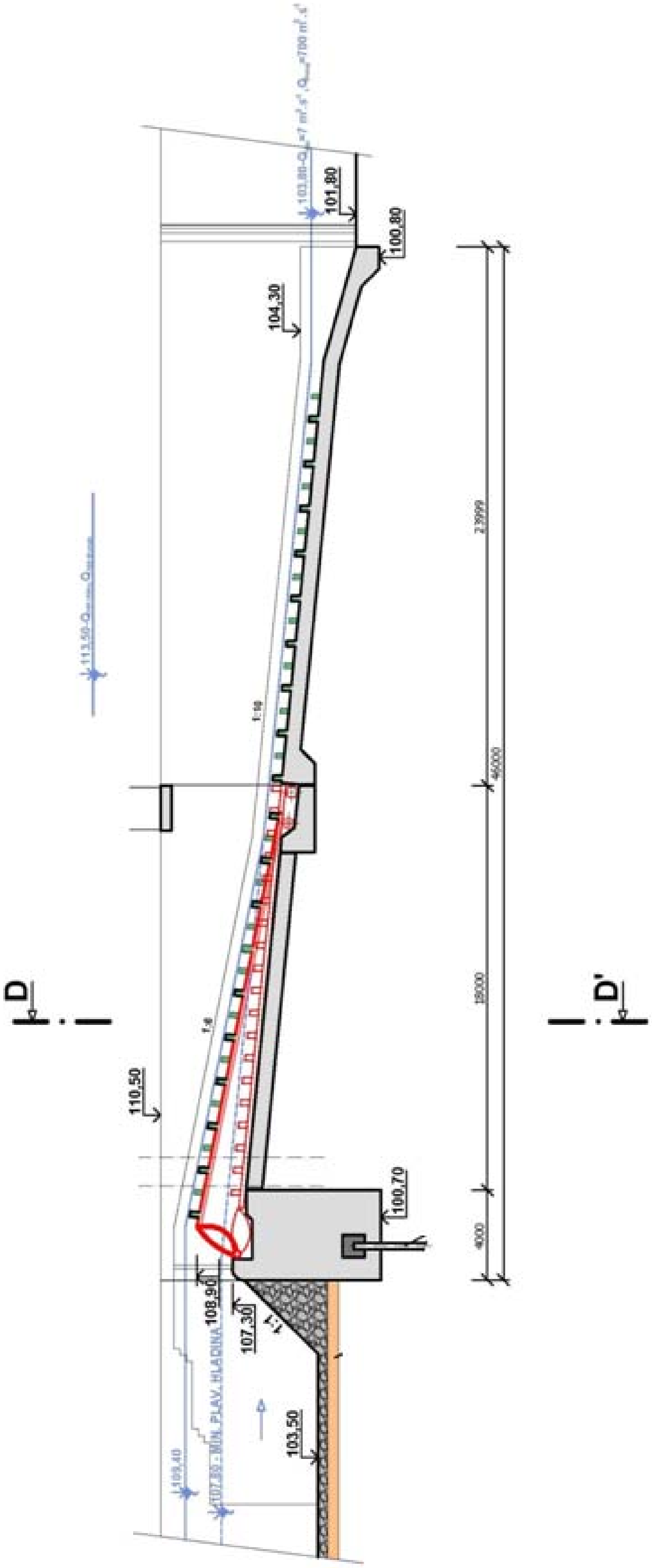

Fig. 9: Kolárovo Water structure - The section of the ship-way for small boats (variant 1) [1]

5 THE WATER LEVEL REGIME

Based on the performed simulations of water level regime for different combinations of discharge on the Váh River and The Danube River was designed navigation mode according to the scheme in Fig. 10. On water level regime of water structure is its considerable variability in the short time of several hours. This is the result of discharge fluctuations caused by regulative operation of Králová hydro-power plant. Whereas the Kráľová is operating in peak mode of operation (daily peaks around 2 with a length of tens of minutes to a few hours with a variable discharge), at the Kolárovo Water Structure will occur to ongoing changes in water levels and water heads. This instability will generate permanent changes of navigation conditions.

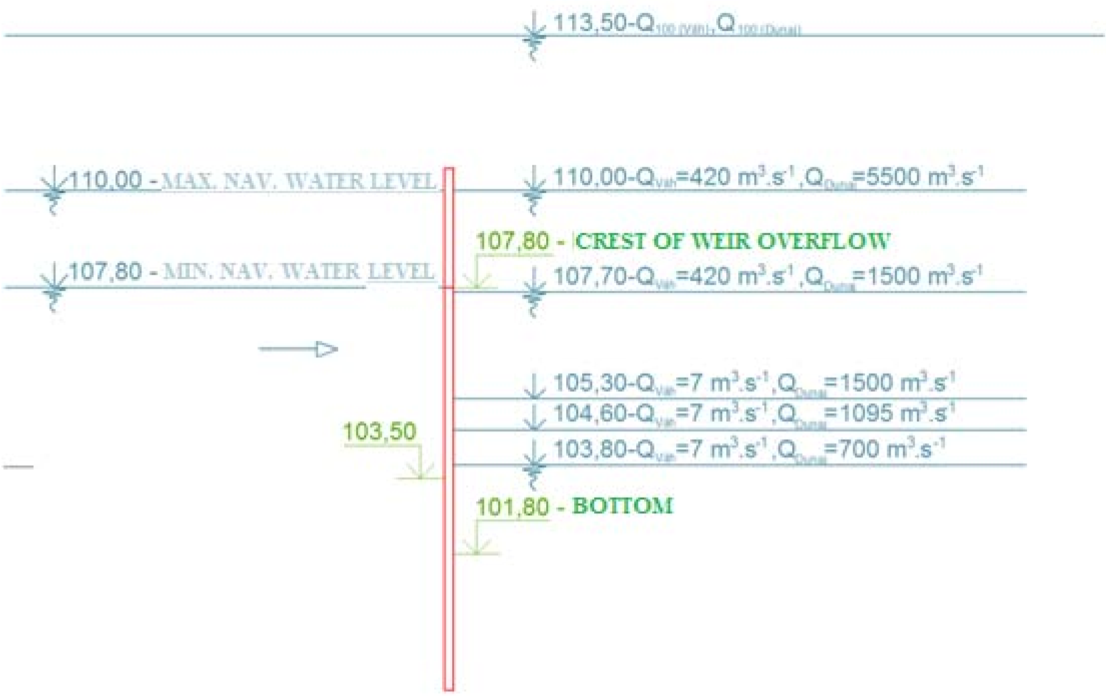

Fig. 10: The Water Level regime at Kolárovo Water Structure [1]

6 THE KOLÁROVO WATER STRUCTURE REQUIREMENTS

Kolárovo Water Structure must accomplish following requirements by its parameters, operational characteristics:

1. ensure specified parameters of fully navigable water way for whole range of discharges and water levels in section Selice Water Structure – Komárno at the min. level at which should ensure the Nagymaros Water Structure,
2. ensure sufficient transport capacity to the adjacent waterway and navigation objects,
3. ensure adequate navigation security for large vessels on the adjacent waterway even when using the navigation objects,
4. ensure adequate navigation security for small boats on the adjacent waterway even when using the navigation objects,
5. ensures to overcome the structure for all migratory animals,
6. not worsened conditions for the transition of many waters,

7. allows the ice and other possible obstructions to pass the water structure,
8. allows the energy utilization of hydro-potential in section Selice – Komárno
9. allows the navigation of the Nitra River up to Nové Zámky

7 CONCLUSION, LIST OF PARTIAL RESEARCHES

For the requirements to be met for Kolárovo Water Structure, the research is needed to optimize the parameter in following researches:

1. The hydraulic regime of surface water, extent and adjustable operating levels at the dam with respect to nautical conditions.
2. Hydraulic groundwater regime depending on location of water structure at the operational modes.
3. The disposition of water structure with respect to nautical terms of large vessels, size, shape, discharge capacity of ship lock and adjoint objects.
4. Optimization of sediment and ice pass though the profile of water structure with respect to maintain the parameters of the water way.
5. Bio-corridor – optimization in migration point of view
6. Ship lock – optimization of filling and emptying with respect to capacity and navigation safety
7. Object for passing a low water head by sport and small boats with respect to capacity and navigation safety
8. Energy utilization at Kolárovo Water Structure
9. The effect of unsteady flow causd by hydro-power plants in the section Sereď – Komárno to ensure the parameters of the water way and safety conditions

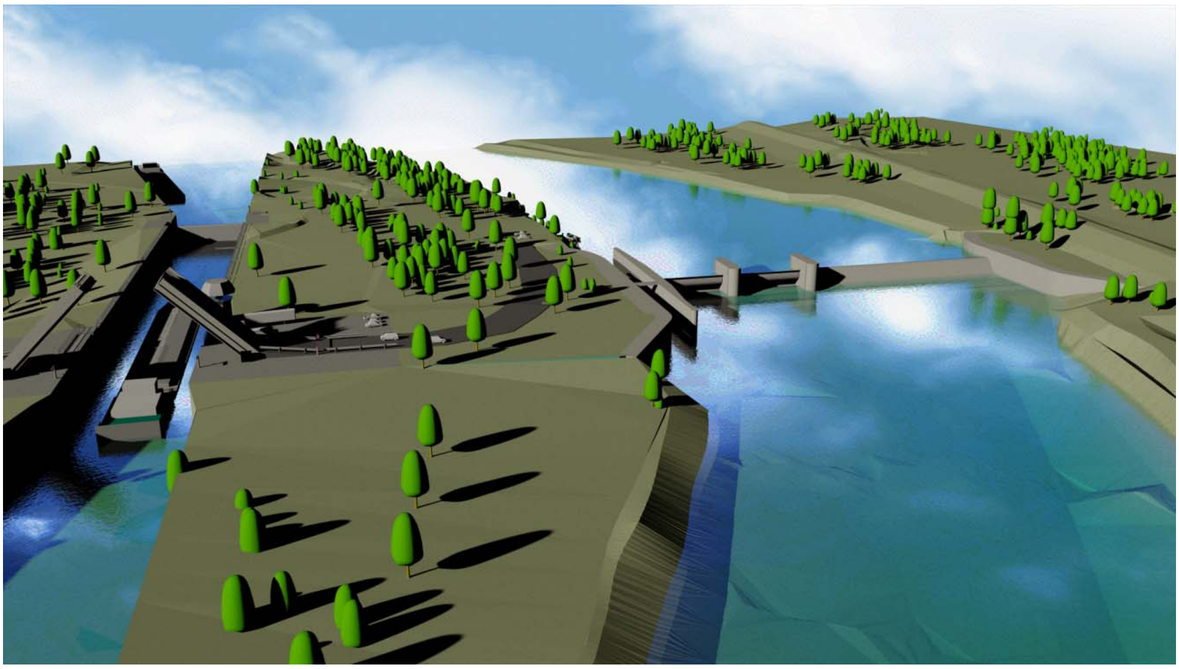

Fig. 11: Kolárovo Water Structure visualisation (variant 1) [1]

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FISHWAYS DESIGN

Eva Ocvirk[1], Mirjana Prša[2], Matea Rebrina[2], Ivana Srednoselec[2]

Abstract: *Currently renewable energy resources are of main interest and among them small hydro power plants are the most important hydrologically speaking. In design process the negative impacts of hydropower dams and stations on ecosystems, particularly on fish migration must be accounted for and adequate protection measures must be achieved. The solutions for restoring fish migration are almost invariably specific to each site because of a unique combination of local features. The identified solutions should take into account all relevant factors including the nature of the obstructions, target fish species, hydrological and hydraulic conditions. This paper deals with the types of fishways suitable for the use in the small hydro power plants on the continental Croatian territory, such as: pool fishways, vertical slot fishways, obstacle fishways and rock-ramps (natural) fishways. An analysis of the required shape, size and length of the fishways conditioned by fish species in continental Croatia was conducted depending on the head that has to be overcome. The presented results can be used for preliminary selection of possible solutions in the early stages of the design.*

Keywords: *Fishways, renewable energy resources, small hydro power plant*

1 INTRODUCTION

At this time in the world, whose number of residents are considerably increasing, and the amount of non-renewable sources of energy decreasing, there is a big interest in renewable sources of energy. Among that kind of energy sources, small hydro power plants (SHPP) are standing out. On the other hand, society becomes more aware of the problem regarding sustainability of the existing ecosystem. SHPP don't have any larger impact on nature, except on fish migration. The problem of fish migration is solved by the application of fishways - constructions which enable upstream and downstream migration of fish species and other organisms inhabiting watercourses. In this paper, basic information on designing and calculation of three types of fishways which were estimated to fit best Croatian freshwater

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fish species, is given. The criteria used to select types of fishways to be studied was that the barriers in fishway are placed so that the fish species don't have to jump across it, but can swim in a curving path between them or through them, using some kind of openings in the barriers. There have been chosen two types of technical fishways and a natural type of fishway, which was used for comparison.

Among technical types of fishways, fishways with alternately placed barriers and pool passes were chosen. As the result of analysis of dependence of fishways geometrical characteristics about hydraulic flow conditions, graphs have been made. Using these graphs and basic project input data fish way can be preliminary modelled. Project input data considered in this paper were suited to, beside freshwater fish species of Croatia, expected heads for SHPP which are possible to be built on Croatia water courses [1]. The goal of this paper is to define graphs which can be used as a basis in early phases of fishways design. In this paper, a comparison of results, regarding design, for studied types of fishways is also given. Advantages and disadvantages of studied types of fishways were also emphasized.

2 PROCEDURE AND RESULTS OF CALCULATION

The goal of the analysis is to show the dependence of fishway design parameters on different input data and to give the basis for designing a fishway. Analysis of fish ways was conducted for various geometries of fishways (for different slopes and width of fishways), and various flows and velocities.

A calculation was made primarily for SHPP for which is foreseen the head between 2 to 6 meters and flow in fishways from 100 to 400 l/s. Width of canal is chosen by the recommendation for each fishway individually. For Croatian freshwater fish species recommended width of the canal is between 2 and 4 meters for fishways with alternately placed barriers and 1 meter for pool passes. Flow velocities suitable for freshwater fish species of Croatia are between 0, 5 m/s and 1, 0 m/s.

2.1. FISHWAYS WITH ATERNATELY PLACED BARRIERS

This type of fishways functions as a canal with increased roughness, which provides lower velocities for given flow and slope of the canal. The main design parameters of this type of fishway are a height of barriers and their distance [2].

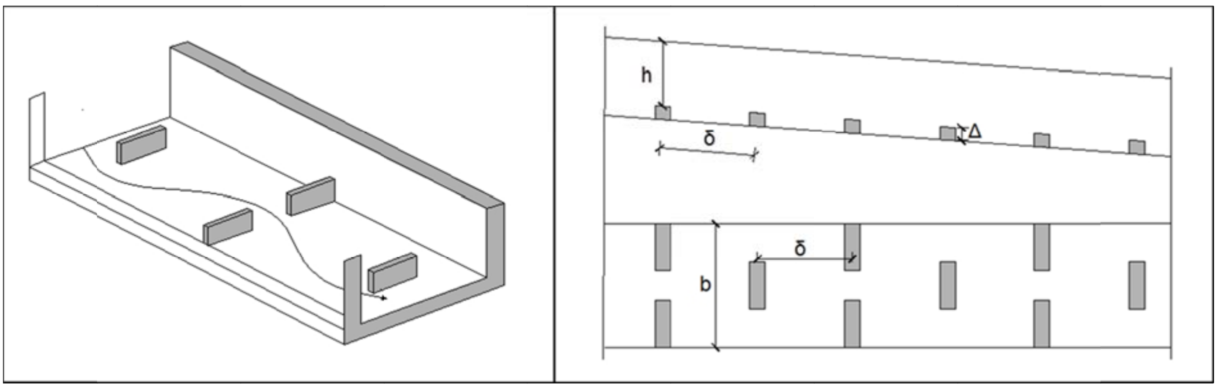
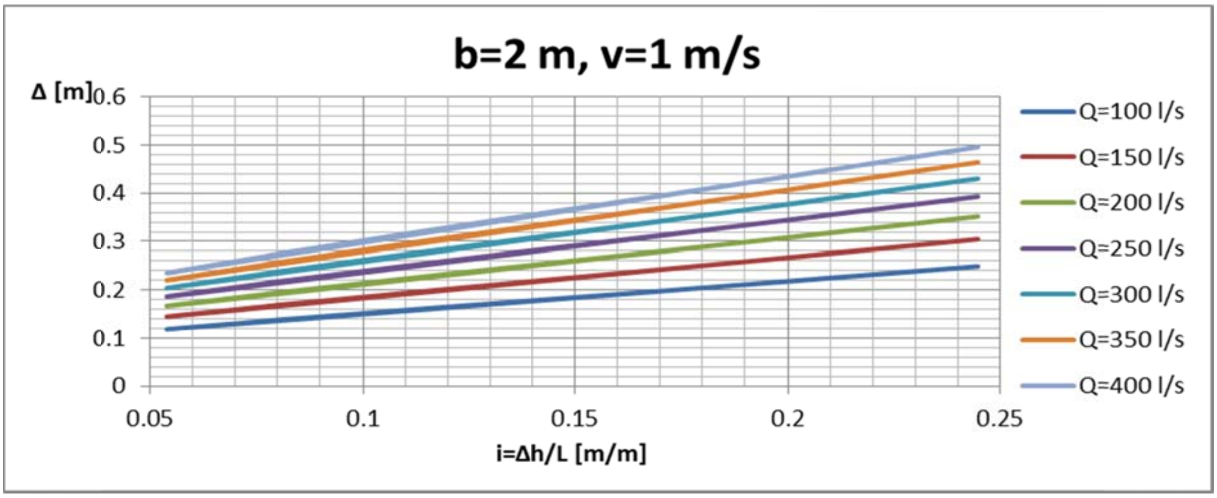


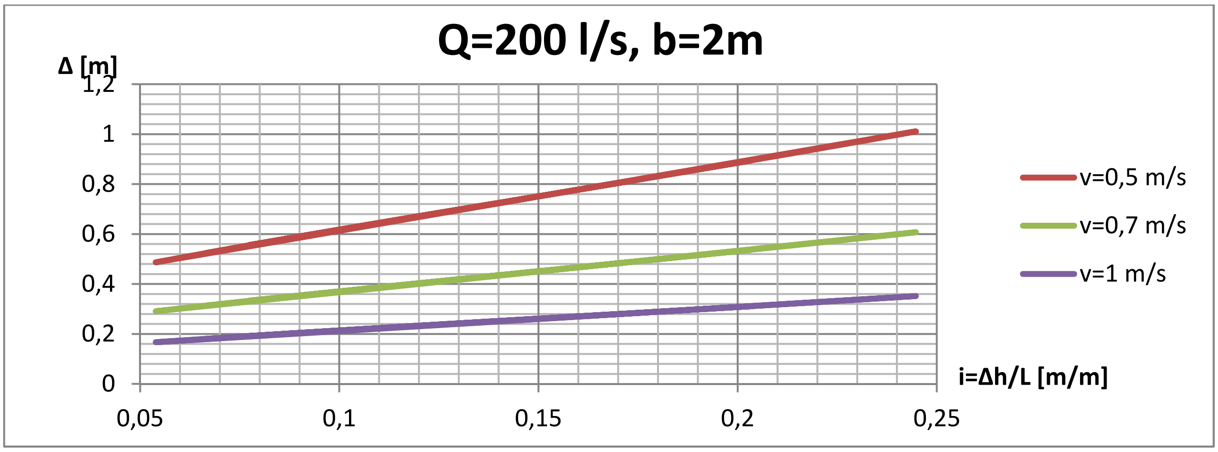
Fig.1 Fishway with alternately placed barriers

Calculation is made according to calculation of canal with increased roughness. Following graphs are given as the result of this calculation.
For the maximum allowed velocity of 1 m/s and minimum width of canal of 2 m, the required height of the barrier can be read from Graph 1, depending on canal slope and flow in canal. Distance between barriers is calculated as $\delta = 7\Delta$.
In that way this graph enables preliminary design of fishway with increased roughness for limiting input parameters v= 1m/s and b=2 m. This input parameters were chosen because they result in minimum excavation works and minimum used land for this type of fishway.
From the Graph 1 the dependence of barriers height and distance on flow is also visible.
With the increase of flow from minimum observed 100 l/s to 400 l/s height of required barriers increases linearly depending on slope. For small slopes of 5 % the change is from 0,12 m to 0,24 m, and for large slopes of 25 % increase of required height is from 0,24 to 0,50 m. Proportional to barrier height increase changes also the distance.



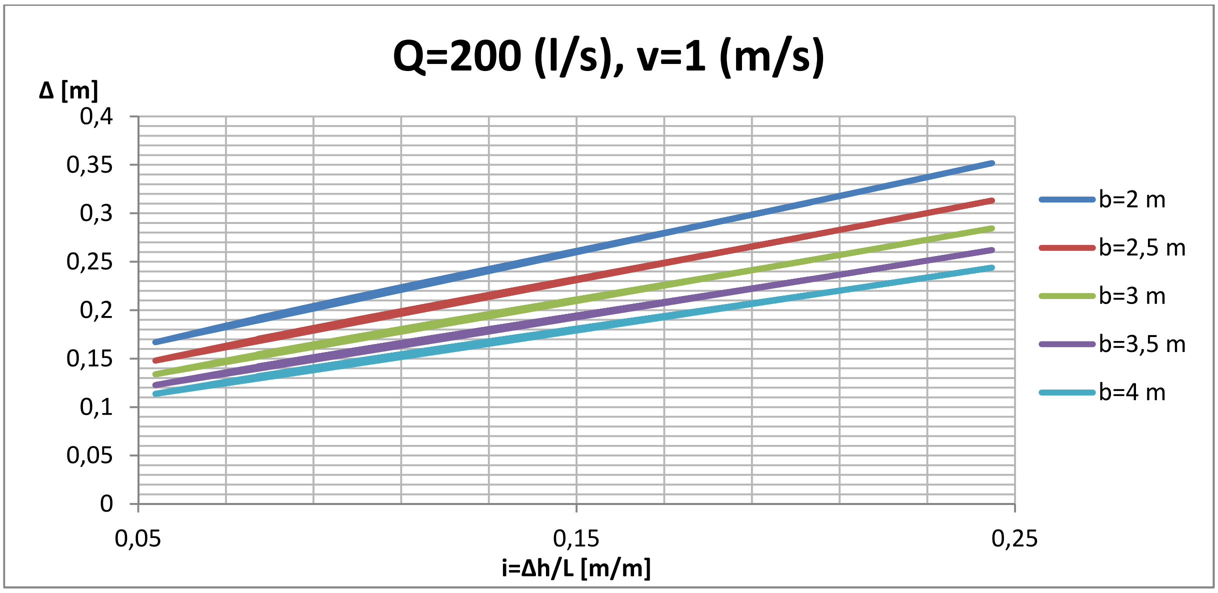
Graph 1. Dependance of barriers height on slope, for various flows

On required height of barriers, and therefore their distance affects also largely canal width and required velocity in canal. Influence of lowering the required velocity on barrier height is shown in Graph 2. Display of influence is given for the flow Q= 200 l/s and canal width of 2m.
There can be noticed that required barriers height increases with decreasing required velocity. The increase is again less visible for smaller slopes and more noticeable for bigger slopes. For slope of 5 % to achieve flow conditions v=0,5m/s compares to v=1m/s height of the barrier must increase from 0,16 to 0,48 and for slope of 25% from 0,36 to 1,00 m.



Graph 2. Dependance of barrierheight on a slope and velocity for Q=200l/s i b=2m

The impact of canal widening on the height and the distance barrier in a fishway is presented in Graph 3. It can be seen that the increase in canal width reduces required barrier height. Representation of influence is given for the flow Q = 200 l/s and velocity v = 1 m/s.
Although the changes are noticeable, they are not significant. By changing the width of the canal with 2m to 4m required barrier height decreases only about 10 centimeters.



Graph 3. Dependance of barrier height on a slope, for various velocity and flow Q=200 l/s and canal width b=2 m

The results of the variability analysis are logical because barrier causes energy dissipation, so for achieving of lower flow rate is required higher energy dissipation (higher barrier). For increased canal width and maintain a pre-defined speed (v = 1 m/s) requires less energy dissipation through the barrier (lower barrier).

2.2 POOL PASSES

When calculating this type of fish pass the goal is to design flow, water surface, a certain canal width and flow velocity in the canal, depending on the needs of fish species to determine the required number of pools, their length and the total length of fish pass. In sections with barriers is achieved permitted flow velocity, while in the space of fish pass without barriers is created calmer zone, which enables fish to rest if it is needed [3].

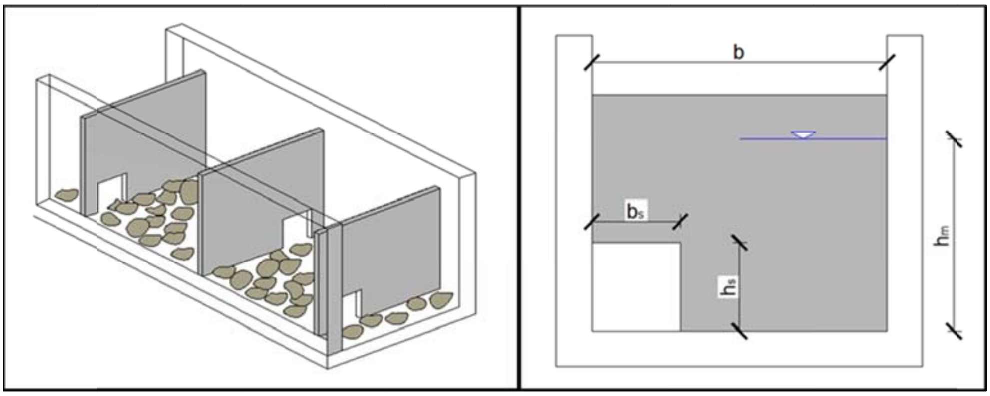
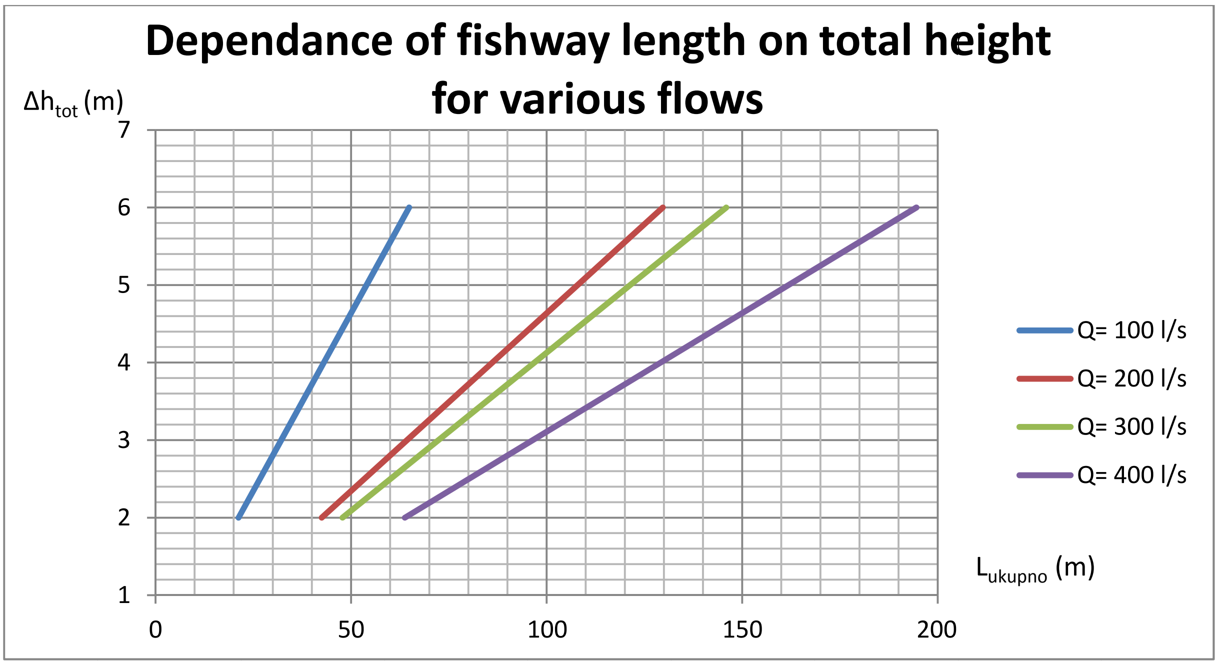


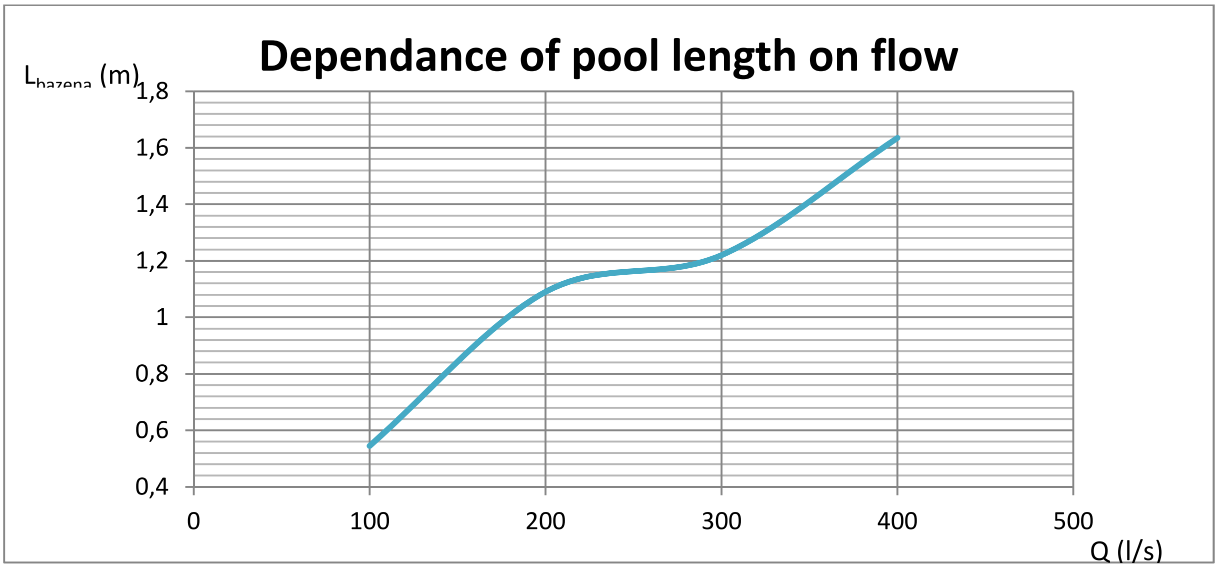
Fig.2 Pool pass

The analyses are resulted in graphical representation dependence of fish pass length on total height, for certain flows. Representation of dependence is given in Graph 4. From this graph, it can be directly read out the fish pass length required for a given flow and total height. Besides, one can observe the dependence of fish pass length to increase flow. The required length of the fish pass, logically increases with the increased flow, and increasing the required length is more pronounced for larger total height. Increasing the flow from 100 l/s to 400 l/s length of fish pass increases to 40 m to total height of 2 m, and 130 m to total height of 6 m.



Graph 4. Dependance of fishway length on total height for various flows

Also obtained is a graphical representation of a pool length dependence of the flow and it is shown in Graph 5. Using this graph for any flow it can be read required length of each pool. Increasing the flow increases the required length of the pool.



Graph 5. Dependance of pool length on flow

By combining Graph 4 and Graph 5 it is possible to solve the problem of designing pool passes. From Graph 4 for design flow and total height it can be read required fish pass length. Then, from Graph 5 for design flow required it can be read length of each pool. Number of pools are obtained by dividing the total length of the fish pass and the length of each pool. These parameters define the ground plan of pool passes.

2.3. NATURAL TYPE FISHWAYS

When calculating this type of fishways input data are cross section, flow and dimension stone blocks, as a result of the calculation gets the number of stone blocks, length and speed of flow. Depending on the arrangement of stone blocks vary the speed of flow along the fishways. In narrow sections (profiles with stone blocks) flow rate values are satisfactory, that is less than the maximum permitted. Speed values in the profiles without stone blocks were reduced, inrelation to the value of the speed of flow in the narrowed sections, and it is vital for the rest of fish species during migration [4].
The calculation was also carried out by HEC-RAS to compare obtained results of previous calculation with results of mathematical model [5]. The results of these two calculations were mostly overlapping.

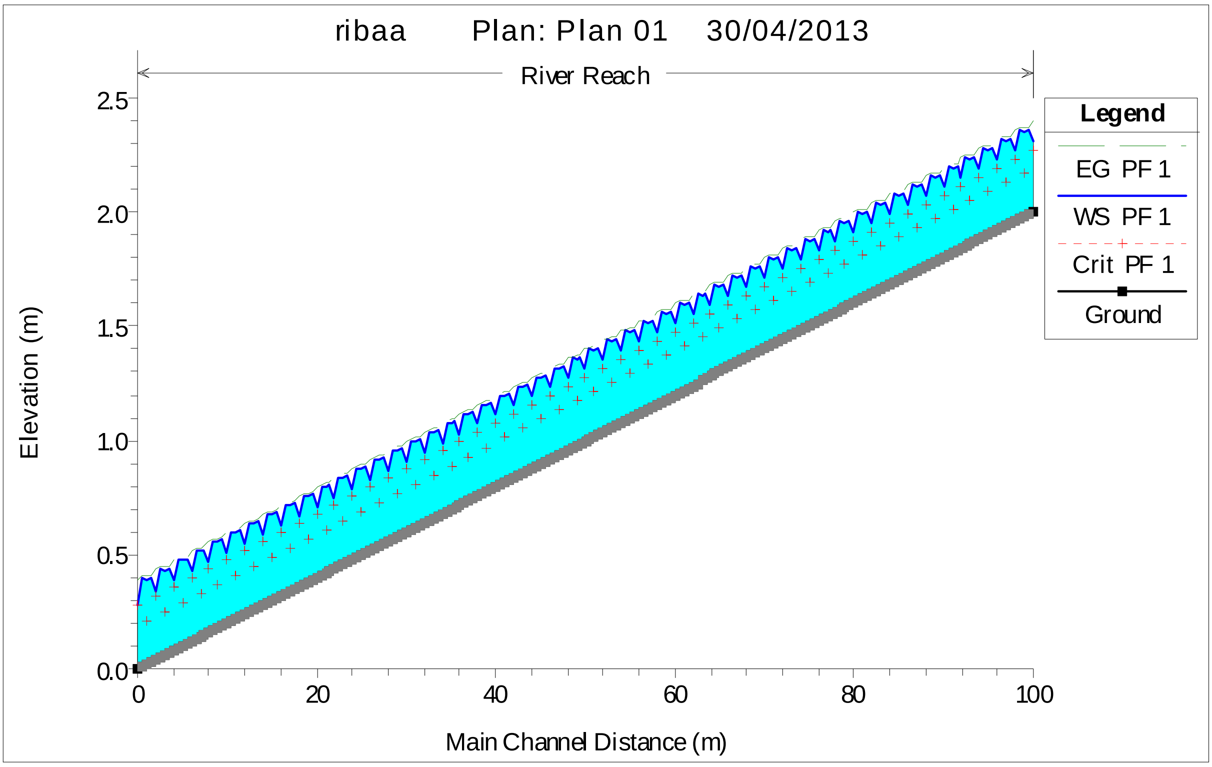
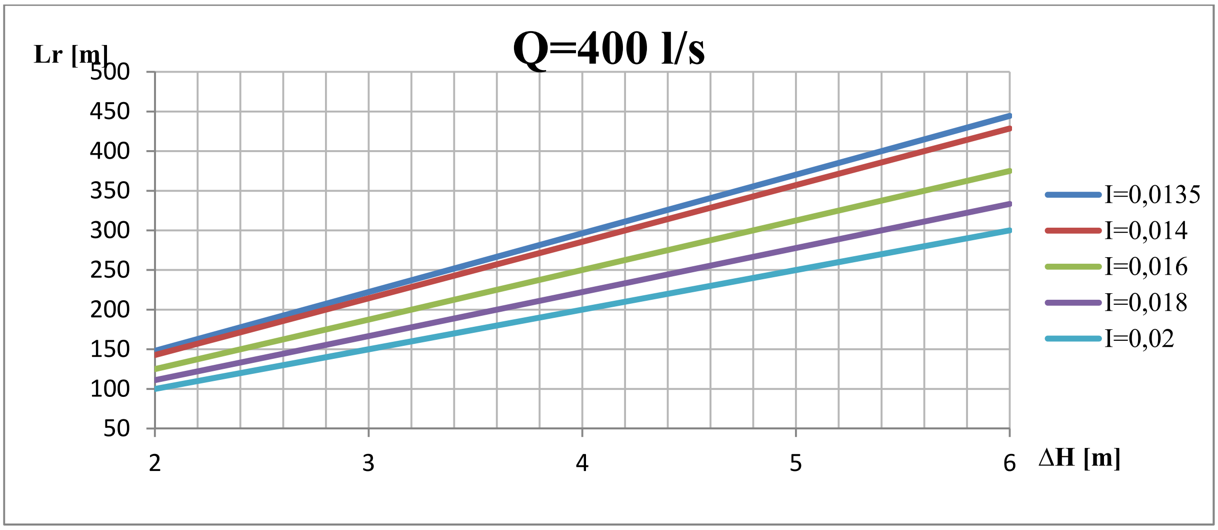


Fig.2 Longitudinal section of natural fishway obtained as result of simulation

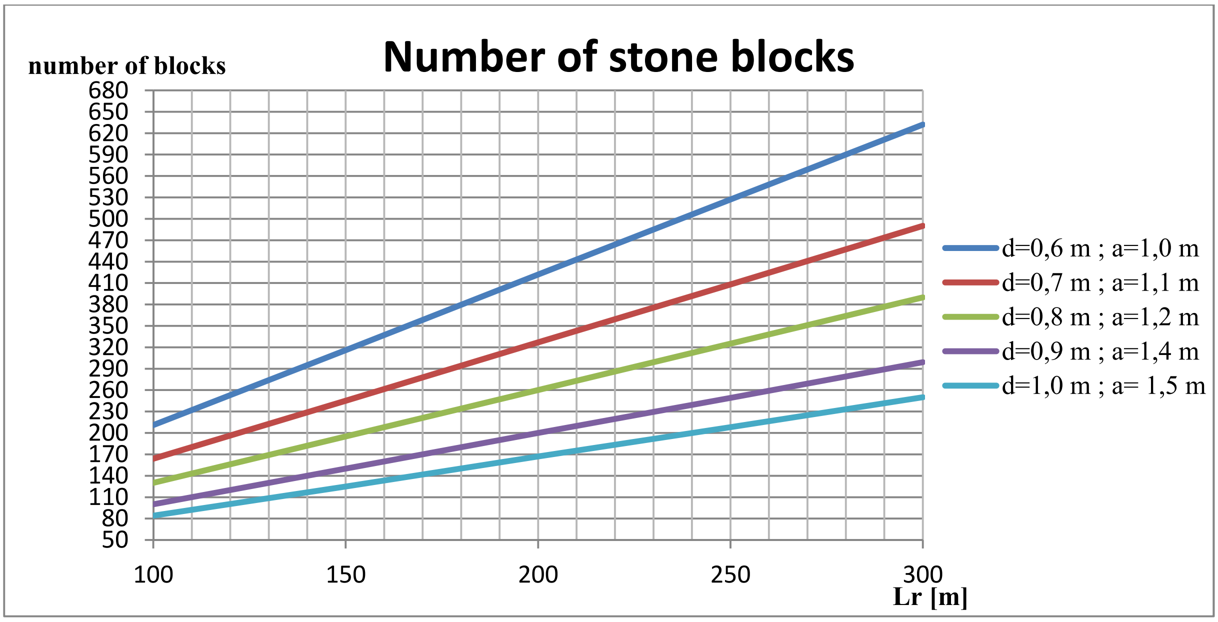
Due to the large number of interdependent, input parameters, it is difficult to show the interdependence of the parameters, the basic concept of sensitivity analysis was carried out using the calculation schemes and on the basis that they were obtained following graphs (Graph 6 and Graph 7).

From Graph 6 is visible dependence required length fishways on the slope. If the slope is less steep, the length is greater. The maximum length obtained for the slope of 1.35%. For this slope length vary from 148.15 m to 444.44 m, depending on the head.



Graph 6. Length depending on the slope and head

By changing the length of the course and the required number of blocks of stone is changing too. Graph 7 shows the dependence of the required number of blocks for the length of fishways ranging from 100 to 300 m, with a slope of 2%. The Graph 7 shows that for the same length of fishways, and smaller diameter stone block and lower their mutual distance, the greater the number of stone blocks. Analysis of changes in the number of stone blocks of the same size and spacing depending on the length of fishways shows that for the longer way, the greater number of stone blocks of the same type is needed.



Graph 7. A number of stone blocks, dependingon the length of fishways and size of stone blocks

3 DISCUSSION OF RESULTS

All three types of fishways that have been calculated, suit Croatian freshwater fish species and their carving swimming. However, each type has its advantages and disadvantages. Conducted hydraulic calculations and comparing the results by type of fishways observed, the following conclusions were made:

- Type with alternately placed barriers are most efficient because it can bridge the required water surface at the greatest slope, i.e. with a minimum canal length. The calculation of this type is easy and fast, as well as its design. Mostly small canal length allows proper positioning of fishway entrance. To provide the proper position of entrance it should be as close to weir, so it is suitable to have small canal length.

- Pool passes type is most convenient designed for migrating fish species. Fish species is easier to cope with parts small height difference between the pools, but in one part of the total height difference. In addition, advantage is that the fish get a place to rest, because the discharge velocity through the opening adapts to value that fish can endure while pool velocity is much smaller, so pool is perfect vacation spot. Besides, the calculation of this fishway is easy and dependence of design parameters is such that is possible its designing based on shown charts. The required length of this type is larger than with a type with alternately placed barriers, but significantly smaller than with natural type. Disadvantage of greater length can be solved by designing pool fishway so that it switch direction for 180°. Also, this type has smallest required width compared with other types.

- The natural type requires the greatest length in order to achieve required flow conditions. Therefore, natural fishways have highest expenses in the performance. The need for greater length results in problems with designing fishway entrance. What is certainly an advantage of this type is its natural appearance. Natural appearance implies that this type is a natural extension of old river bad, or it is built from natural materials of an old river bed and stone blocks. Besides it provides natural habitat to fish, this type also provides aesthetically most suitable fishway. Due to the stated advantages natural type is a contemporary way to regulation of waterflow.

In this study it was not performed an economic analysis presented solutions, which in practical application can play a significant role.

4 CONCLUSION

Fishways are important structures that allow the simultaneous use of waterways for energy production and conservation of the diversity of fish species in watercourses.
Designing fishways is a responsible task because efficacy of fishway depends on it.
As presented in this paper, the design is more structured and easier for technical types of fishways, a less elaborate and complex for natural types of fishways. Efficacy of the designed and built fishway should be controlled in the use. Efficacy of fishway is measured by recording number of organisms that pass through fishway. Thus adopted findings are used in the design of new fishways thus improve the given area ands species. Thus Croatia is facing a long period of designing and adopting fishways for its area and fish species. This period can be overcoming only with an integrated approach of designing and by efficacy measuring of fishways.
Successfully design will be achieved by exploiting rivers for energy purposes, without disrupting wild life in the water, which is the goal of any country that strives to protect its natural resources, but also the economic progress and development.

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MATHEMATICAL MODELLING OF EXTREME CHANGES IN FLOW MODE TO THE WATERWORK GABČÍKOVO

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Abstract

Extreme change of flow through the Hydroelectric Powerplant (HPP) Gabčíkovo has far-reaching consequences on the water level mode of the Gabčíkovo Waterwork. Abrupt changes in the water level as a result of a regulated and unregulated power flow through the HPP Gabčíkovo lead to hazardous situations in the past. Mathematical modelling provides a suitable tool to solution the critical situations using simulating and allows to modify manipulation order of the Waterworks with intention of the improve safety operational of the HPP Gabčíkovo. Calibration and verification of mathematical models has to be based on historical data, especially during the uncontrolled shutdown of the HPP Gabčíkovo.

Keywords

Hydroelectric Powerplant Gabčíkovo, mathematical modelling, emergency outage, flow mode, shock waves, navigation.

1. INTRODUCTION

Hydroelectric powerplant Gabčíkovo on the Danube is our largest channel powerplant. The installed capacity of 720 MW has been determined with respect to the originally proposed regulation – peak operation. This operation corresponds the total installed capacity of turbines (4 000 m3s-1). Incomplete Waterwork System of Gabčíkovo – Nagymaros according to the original contract project (mainly unrealized expansion reservoir which should have created by

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profile Nagymaros) has not allowed to do the original peak operation of the HPP Gabčíkovo. Now it is run-of-river powerplant.
The Waterwork Gabčíkovo is the multipurpose hydroelectric project. It has to give priority to the protection of the adjacent areas against floods, to ensure prescribed water consumption and to ensure international navigation. And then follow up using of hydro power in the hierarchy of importance function.
In ordinary operation of the HPP Gabčíkovo we have not to eliminate unforeseeable situations(for example emergency outage of the HPP Gabčíkovo, [2]) which can significantly endanger the safety of the navigation. From the previous operation of the HPP Gabčíkovo we have resulted the need to unequivocal quantify the impacts of uncontrolled outage of the HPP Gabčíkovo on the navigation and propose real measures to limit consequence of such failure.
The fig. 1 shows flow mode in the HPP Gabčíkovo during the emergency outage on 14.6.1995. As a result of sharp decline of the flow mode in the HPP Gabčíkovo about 3000 m3s-1 there was arised the shock wave with 0,6 m height in the derivation bypass. This wave was moving through the derivation bypass into the Hrušov reservoir. The shock wave caused separation the ferry to transport people and vehicles in the wharf. The wharf is located approximately 17 km from profile Gabčíkovo upstream.

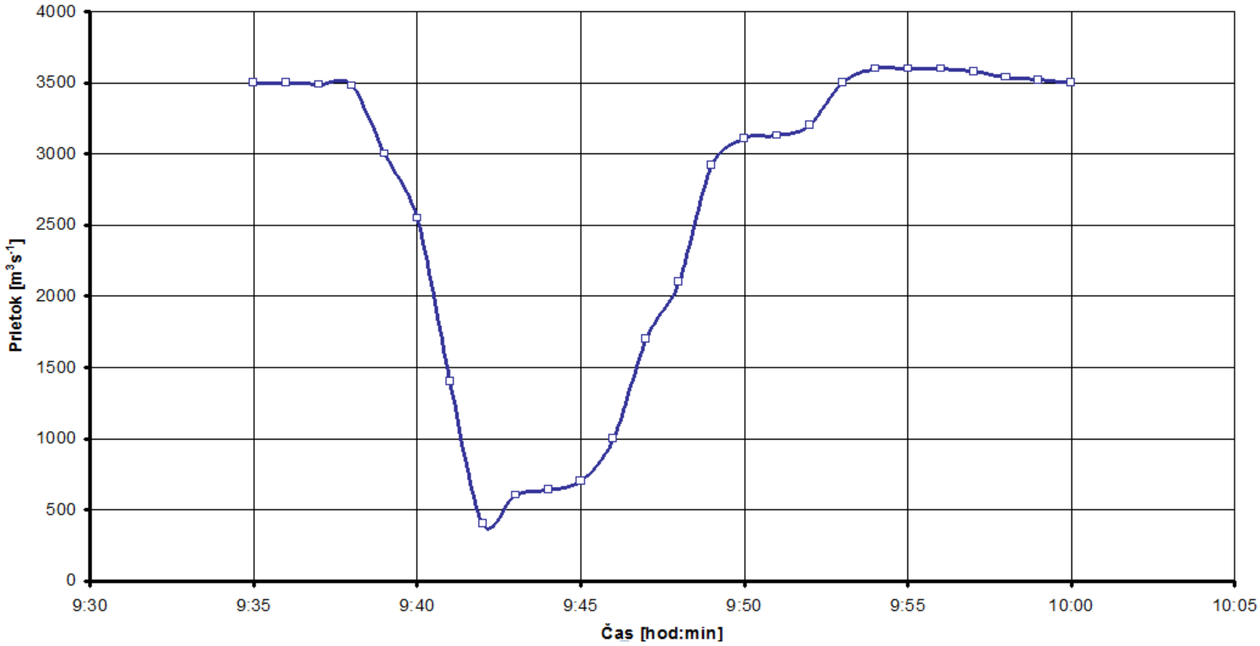

Fig. 1 Flow mode through the HPP Gabčíkovo – outage 14.6.1995

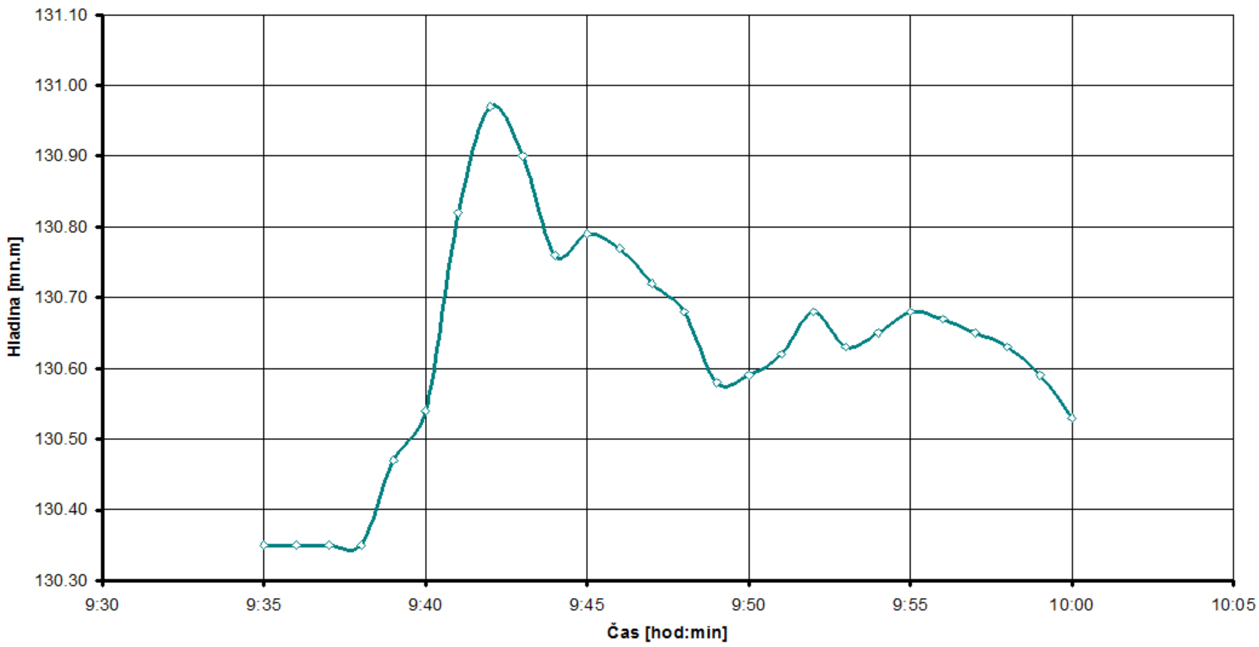

Fig. 2 Timing of the upper water level of the HPP Gabčíkovo during outage 14.6.1995

2 HYDRODYNAMIC MODELS

When flowing the water we can meet with two basic types of unsteady flow:

- Shock waves - they arise by sudden flow changes in boundary profiles. Shock waves represent the emergency operation.
- Smooth waves - they arise by successive flow changes in boundary profiles. They characterize smooth peak operation of the powerplant.

Subtracting the values of fig. 1 we find that the length of the flow reduction was about 5 minutes, after 3 minutes there occurred to the gradually restore of the flow and to complete restoration of flow after a further 10 minutes. It follows that for modeling the type of failures we can use equations describing fluid wave. This argument was necessary to verify directly by controlled outage of flow through the HPP Gabčíkovo with other additional measurements of water level development in selected profiles above and below the HPP Gabčíkovo. The measurement results were used for calibration and verification of the mathematical models. Up to now only one controlled outage was implemented on 26.6.1997 (fig. 3, [5]). The organizations which were participated with preparation of the very important experiment from the point of the further development of mathematical modeling:

- Danube Basin, state enterprise, section Gabcíkovo
- Hydroelectric power plants Trenčín, SpA, section Gabcíkovo
- Water management construction, state enterprise, Bratislava
- Department of Hydraulic Engineering, Faculty of Civil Engineering STU in Bratislava
- Hydrometrics, s.r.o., Bratislava

The results of detailed measurements of water level mode above and below HPP Gabčíkovo are used practicaly today.

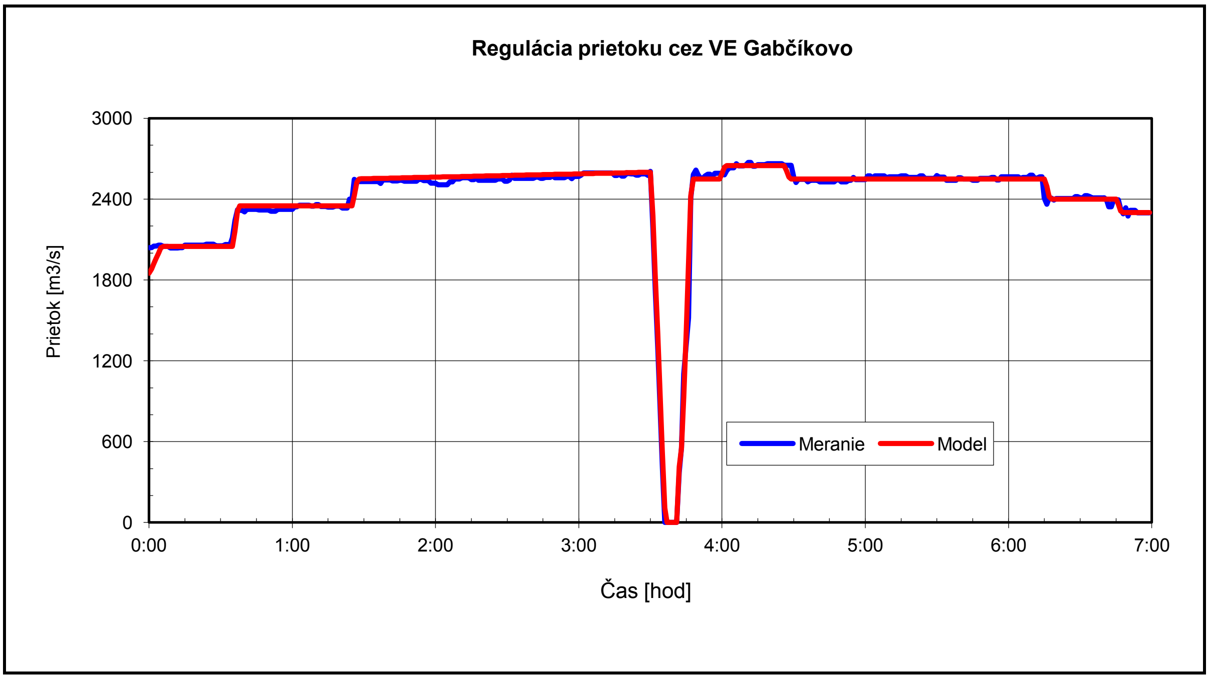

Fig. 3 The controlled outage of the HPP Gabčíkovo – 26.6.1997

In the fig. 4 there is an equivalent scheme of the Waterwork Gabčíkovo, which was used for one-dimensional (1D) complex mathematical modelling of the operation of the work. Calibration of the mathematical models (setting of the roughness of the bottom in individual sections) was performed for steady flows before its controlled outage.

Unsteady flow is characterized by a time change of flow and water levels of individual profiles in open channels. When slow-changing these variables (smooth waves) we can express the physical substance by a fundamental system of Saint-Venant partial differential equations in shaped:

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

$$\frac{\partial(\beta QV)}{\partial x} + \frac{\partial Q}{\partial t} + gA\frac{\partial h}{\partial x} = gA(i_0 - i_e) + q_\ell v_\ell \qquad (2)$$

where :

Q discharge [m3s-1]

A cross-section area [m2]

ql lateral intake or outlet [m2s-1]

x position of the section measured from the upstream end [m]

t time [s]

V mean velocity at the section [ms-1]

h elevation of water surface [m]

g acceleration due to gravity [ms-2]

b momentum coefficient

i0 slope of the channel

ie slope of the energy grade line

vl component of the velocity of the intake or outlet in the x direction [ms-1]

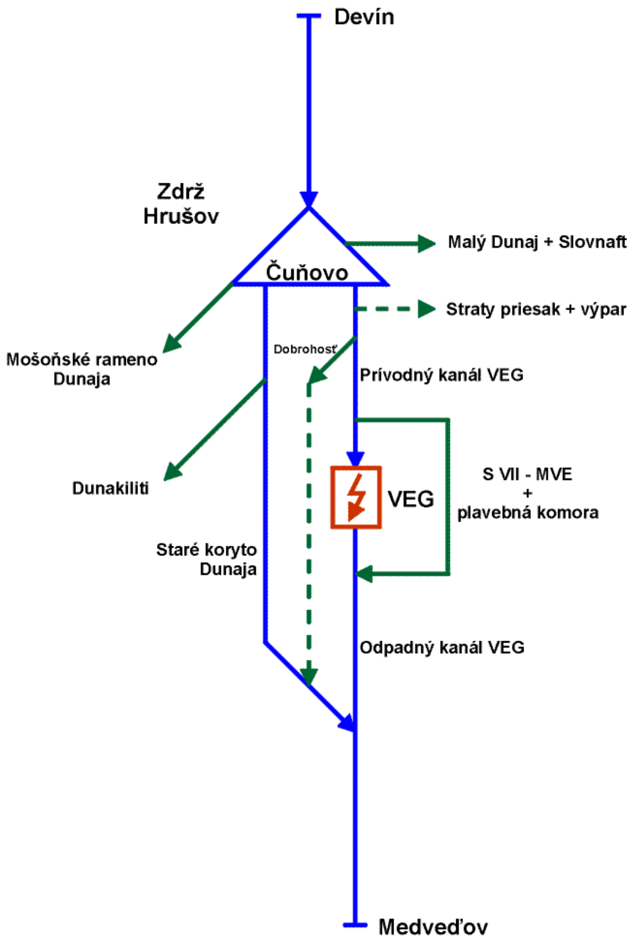

Fig. 4 The equivalent scheme of the Waterwork Gabčíkovo

The 1D model verification results (compare the temporal evolution of the water levels in selected profiles, [3]) in the derivation bypass are shown in the fig. 5 (upper water level of the HPP Gabčíkovo) and in the fig. 6 (profile of the ferry). We can observe very good agreement between measured water levels and the values obtained by the 1D model in the derivational bypass of the HPP Gabčíkovo.

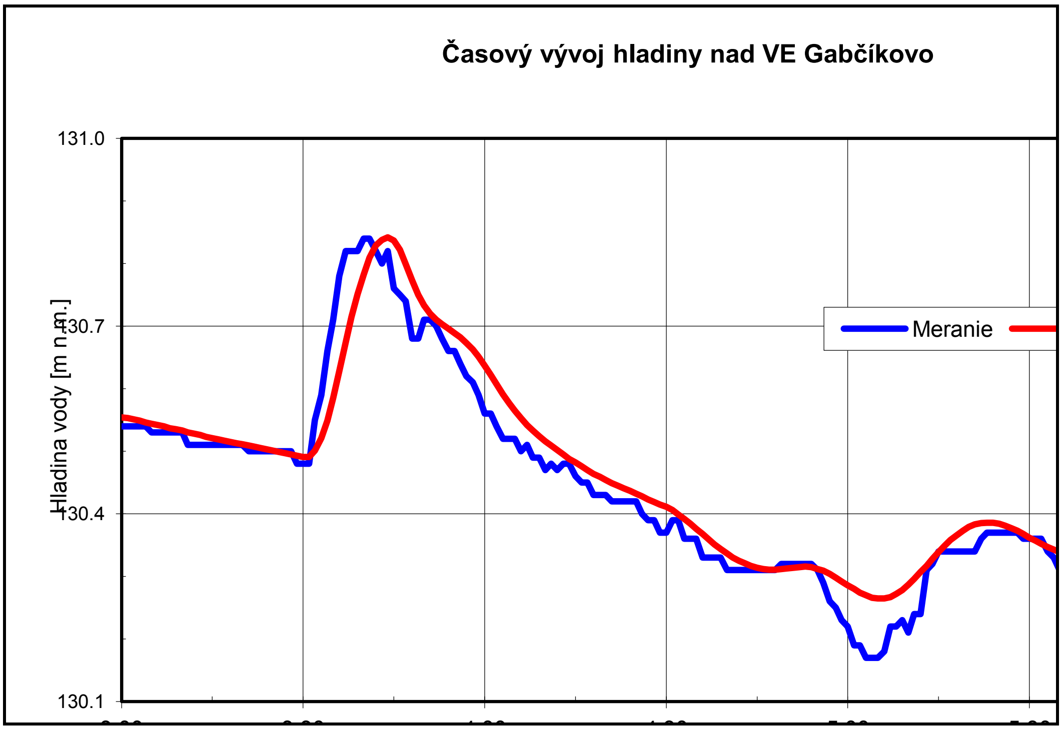

Fig. 5 Timing of the water level of the HPP Gabčíkovo – 26.6.1997

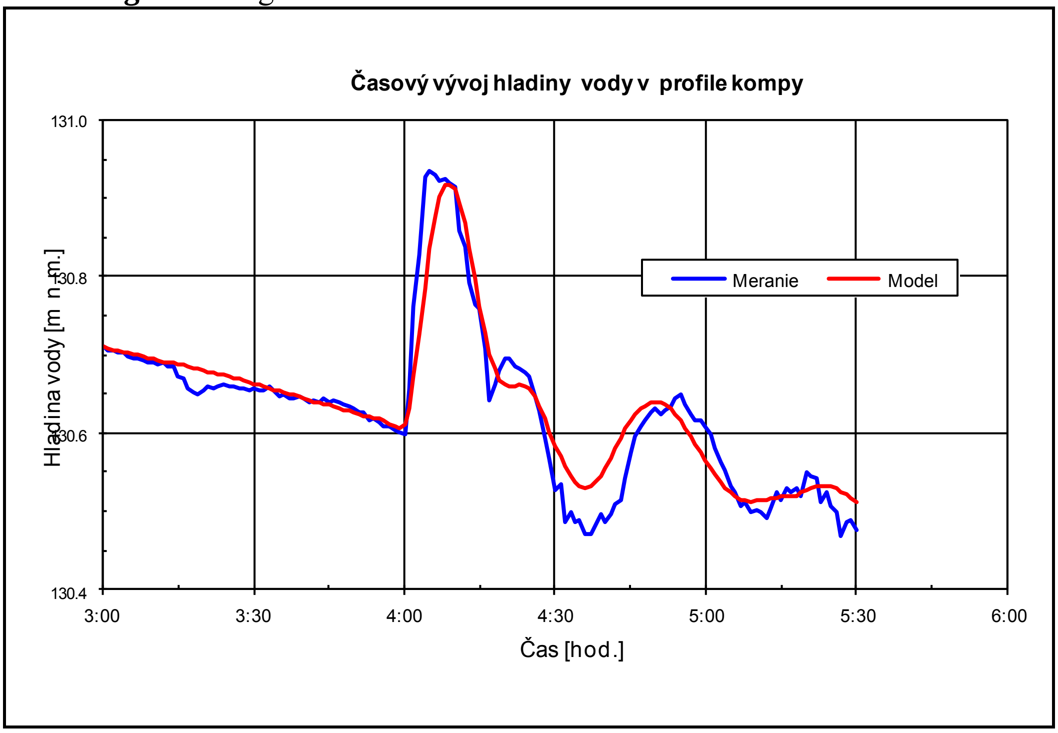

Fig. 6 Timing of the water level of the Ferry profile – 26.6.1997

The different situation is in the derivation bypass below the HPP Gabčíkovo (Fig. 7), where we can not apply the 1D models of water flow – the deviation of the obtained data on the basis of mathematical modelling of the measured values is no longer acceptable.. This causes significant interaction of upstream section to the lower gate of the locks (Fig. 8). In this case it is necessary to use the funds of the 2D modelling.

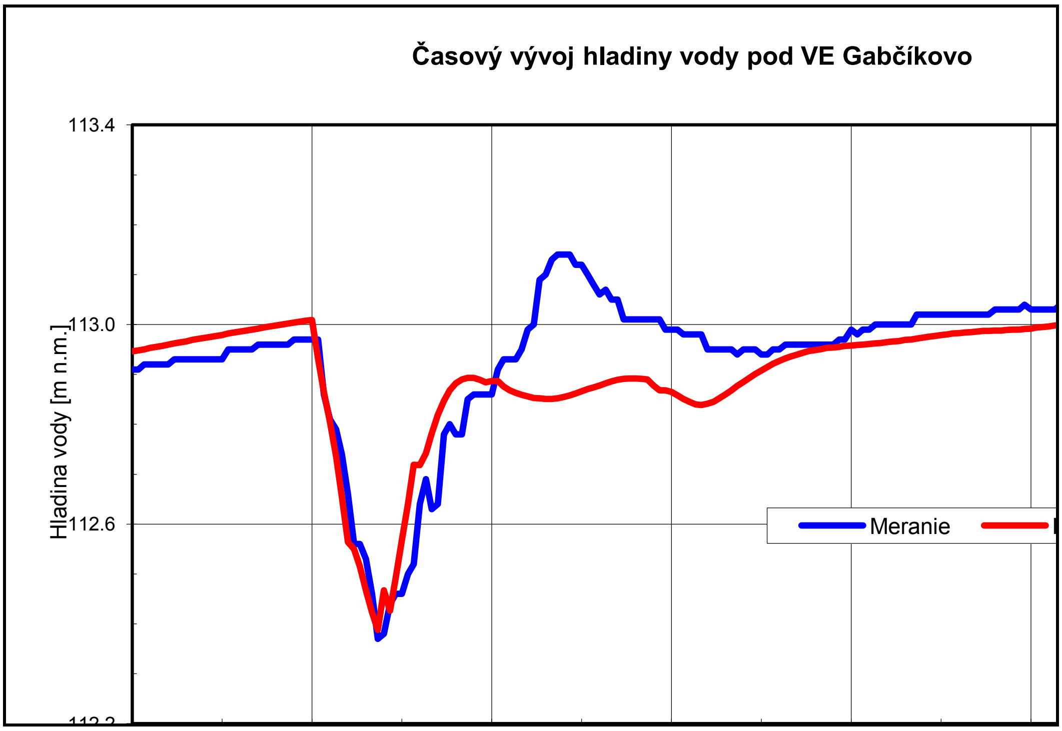

Fig. 7 Timing of the water level below the HPP Gabčíkovo – 26.6.1997

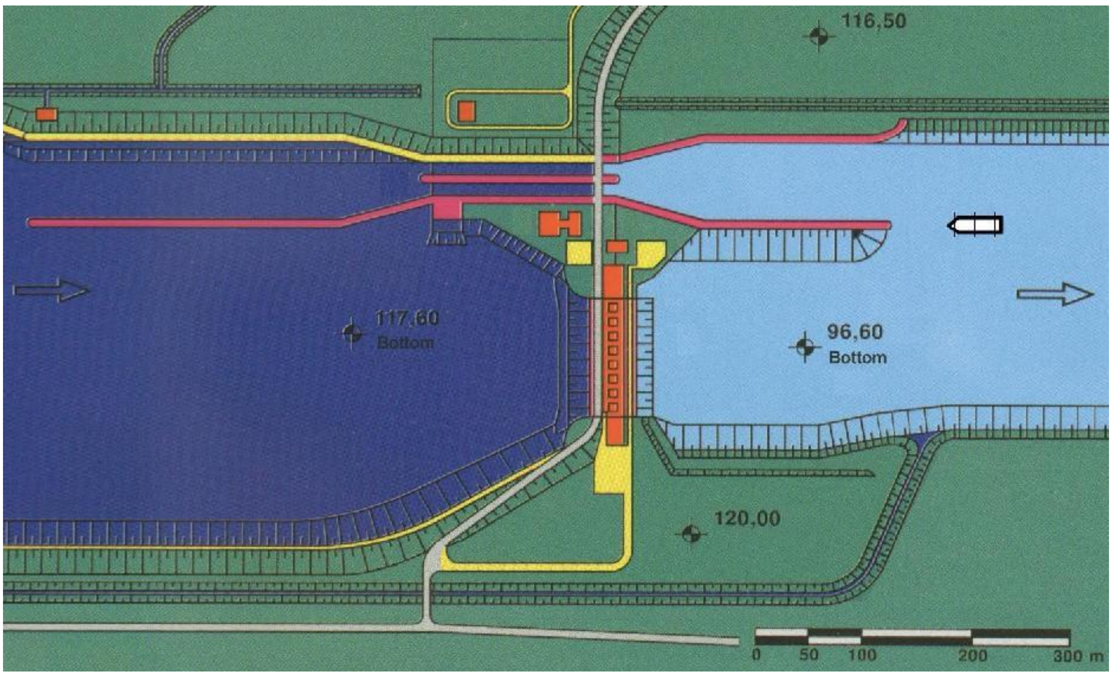

Fig. 8 The layout of the experiment for 2D measurement – cross component of the flow – 26.6.1997

For 2D mathematical modelling of water flow below HPP Gabčíkovo was used finite difference method for nonlinear shallow water equations:

$$\frac{\partial u}{\partial t}+u\frac{\partial u}{\partial x}+v\frac{\partial u}{\partial y}+g\frac{\partial \zeta}{\partial x}+\frac{gu\sqrt{u^2+v^2}}{C^2H}-\nu\left(\frac{\partial^2 u}{\partial x^2}+\frac{\partial^2 u}{\partial y^2}\right)=0 \qquad (3)$$

$$\frac{\partial v}{\partial t}+v\frac{\partial v}{\partial y}+u\frac{\partial v}{\partial x}+g\frac{\partial \zeta}{\partial y}+\frac{gv\sqrt{u^2+v^2}}{C^2H}-\nu\left(\frac{\partial^2 v}{\partial x^2}+\frac{\partial^2 v}{\partial y^2}\right)=0 \qquad (4)$$

$$\frac{\partial \zeta}{\partial t}+\frac{\partial}{\partial x}(Hu)+\frac{\partial}{\partial y}(Hv)=0 \qquad (5)$$

Where: u component of the velocity in the x direction
v component of the velocity in the y direction
□ elevation of water above datum
h depth of water below datum
H depth of water
g acceleration due to gravity
C Chezy velocity coefficient
ν dynamic viscosity

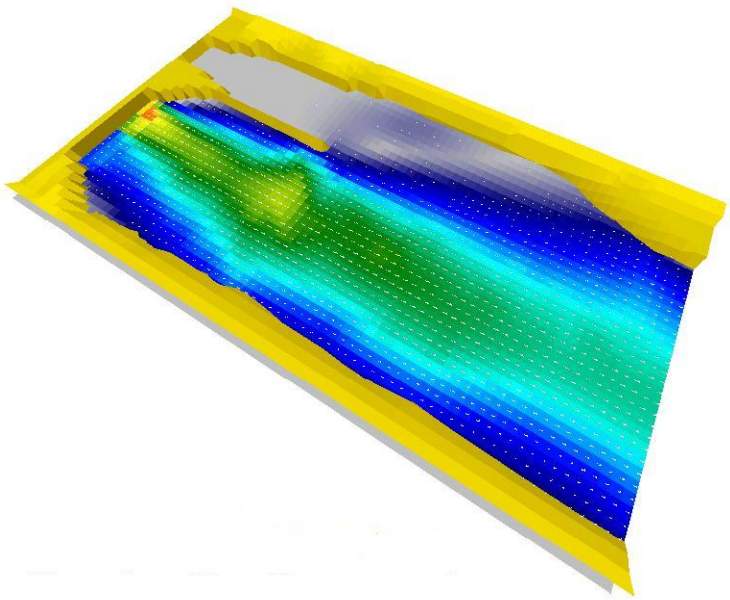

Fig. 9 The velocity field below the HPP Gabčíkovo before the controlled outage – 26.6.1997

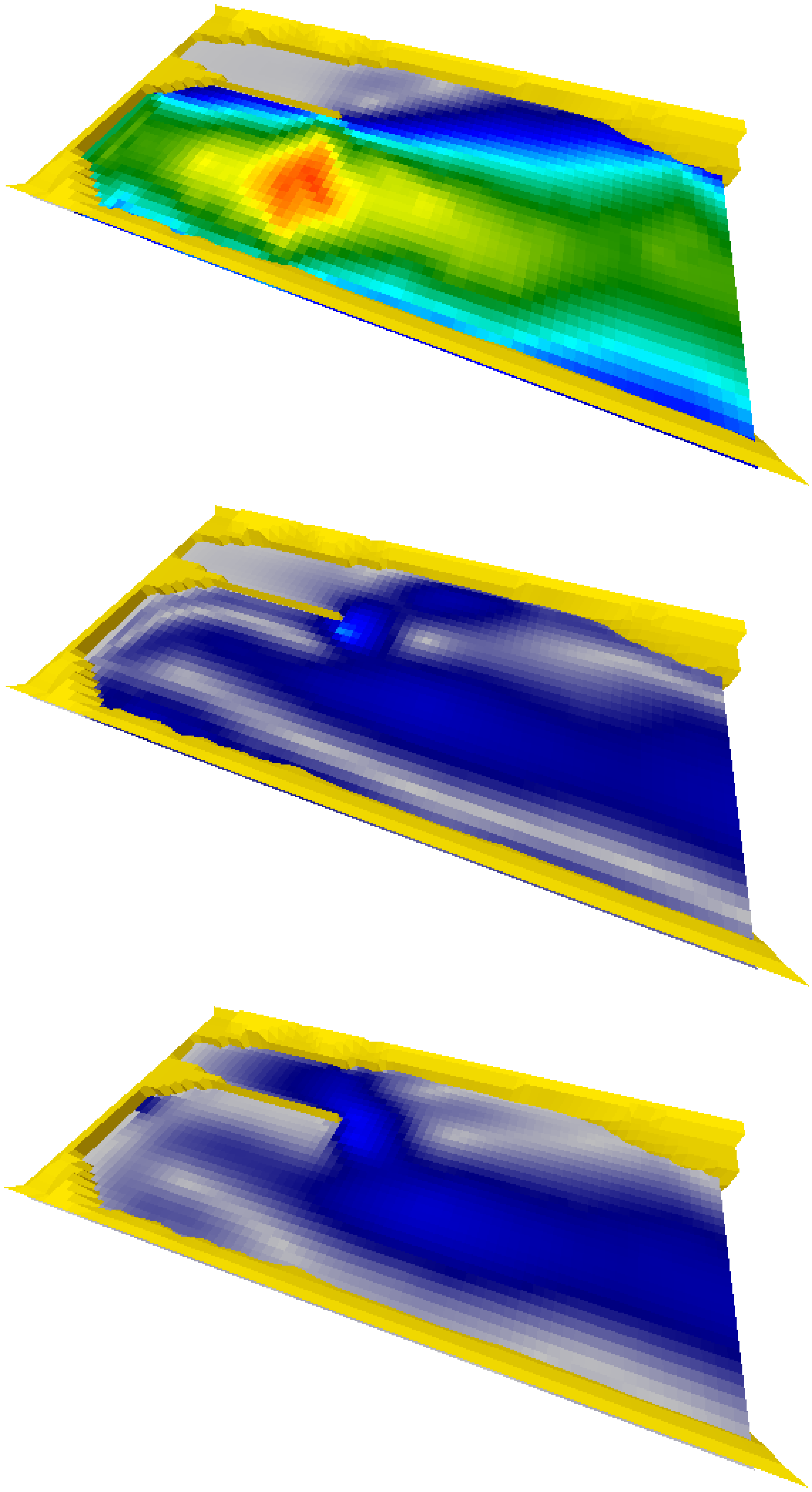

Fig. 10 The development of the velocity field below the HPP Gabčíkovo during the controlled outage – 26.6.1997

In the fig. 9 and 10 there are results of 2D mathematical modelling in the section below the HPP Gabčíkovo during controlled outage on 26.6.1997. In the fig. 10 there can see very well the interaction of the parts of derivation bypass below the powerplant during the first 10 minutes of controlled outage of powerplant. The Verification of 2D flow is very problematical in the terrain during measure and especilly in fast-going action.
The fig. 11 shows the illustration of particular verification of 2D model. There are compared the measurement of cross components of velocity from profile pontoon. The pontoon was at anchor below the guide wall which is located between the derivational bypass below the powerplant and lower mooring area (fig. 8). Measurement was influenced by strong wind and move of pontoon during the attempt. Despite the fact we can state good agreement in the shape of water level and identical maximum velocities in derivation channel.

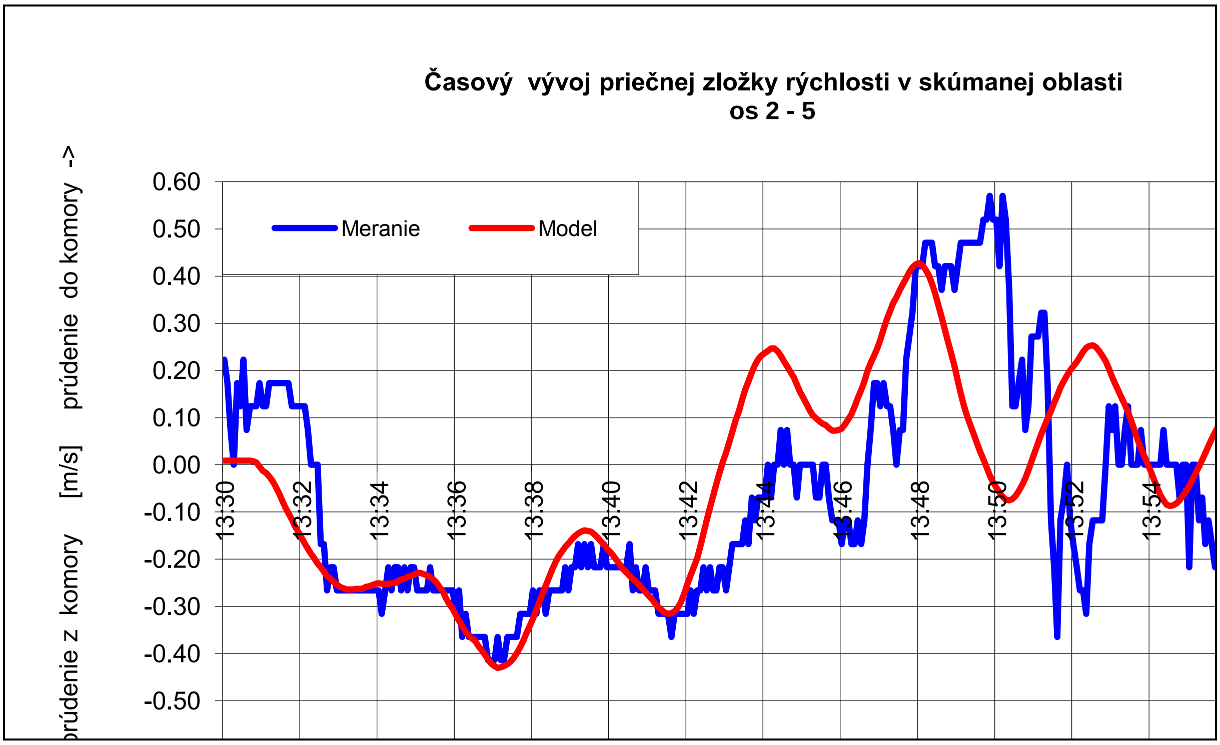

Fig. 11 The verification of cross elements of velocities, measured from profile of pontoon below the HPP Gabčíkovo – 26.6.1997

During verifications there was monitored the water level mode in the each important profiles of the Waterwork Gabčíkovo. Very important information from the viewpoint operation of powerplant there represent water level mode in the upper and lower reservoir of the powerplant Gabčíkovo and in the upper reservoir there arise positive wave with movement into the reservoir Hrušov during emergency outage of powerplant. Close Vojka village the water level mode is one of important indicator for navigation on the Danube. There was achieved very good agreement between the results of the mathematical model and control measurements. In the lower reservoir there arise negative wave which move to the Medveďov village. Modelling of the negative wave is important for decrease of water level of the Danube in ford sections (for example profile of the Sap village). Fall of water level under the navigation level can cause the impact of vessel on the channel bottom. It can do a lot damage. To improve the ability of the model of the Sap area and solution of further tasks (eg

artificially induced increased discharges in the old Danube riverbed in terms of Gabčíkovo Waterstructure manipulation) the sub-models were integrated together with the old Danube riverbed into the HDM2000 network model. For these needs was the group of Gabčíkovo Waterstructure schematized according to fig. 4.
The network model allows the calculation of complex hydrodynamic phenomena in the standard discharge manipulation through the Gabčíkovo Waterstructure. More detailed modelling of interactions of individulal areas of the downstream reservoir can be expected to increase the ability of 1D model under the Gabčíkovo Hydropower plant for cases of extreme discharge changes.

3 USE OF THE MEANS OF MATHEMATICAL MODELING

We can successfully used calibrated and verified mathematical model for research of the possibile eliminations of impact of the emergency outage of the HPP Gabčíkovo. In 2012 the diploma thesis of Andrea Palkovičová was devoted to the issue ([4]) in the thesis she investigated the consequences of emergency outage of the HPP Gabčíkovo and subsequently she examined the possible drafts for their elimination through leaking of flow through the locks or through the profile Čunovo.
Simulations of the flow outage were divided to the five categories:

- uncontrolled outage of the HPP Gabčíkovo without elimination (fig.12 a 13, tab. 1)
- controlled outage of the HPP Gabčíkovo without elimination - gradually decreasing the flow without energy production
- uncontrolled outage of the HPP Gabčíkovo with supplying of water through the profile Čunovo
- uncontrolled outage of the HPP Gabčíkovo with supplying of water through the one lock
- uncontrolled outage of the HPP Gabčíkovo with supplying of water through the both lock

Impacts were examined for discharges from Q = 750 m3.s-1 to Q = 3750 m3.s-1 in selection profiles :
K1 - rkm 8,171 in derivation bypass just below the powerplant,
K2 - rkm 1806,40 close the Medveďov village,
K3 - rkm 1796,02 on the limited ford,
K4 - rkm 1767,94 close the Komárno town.

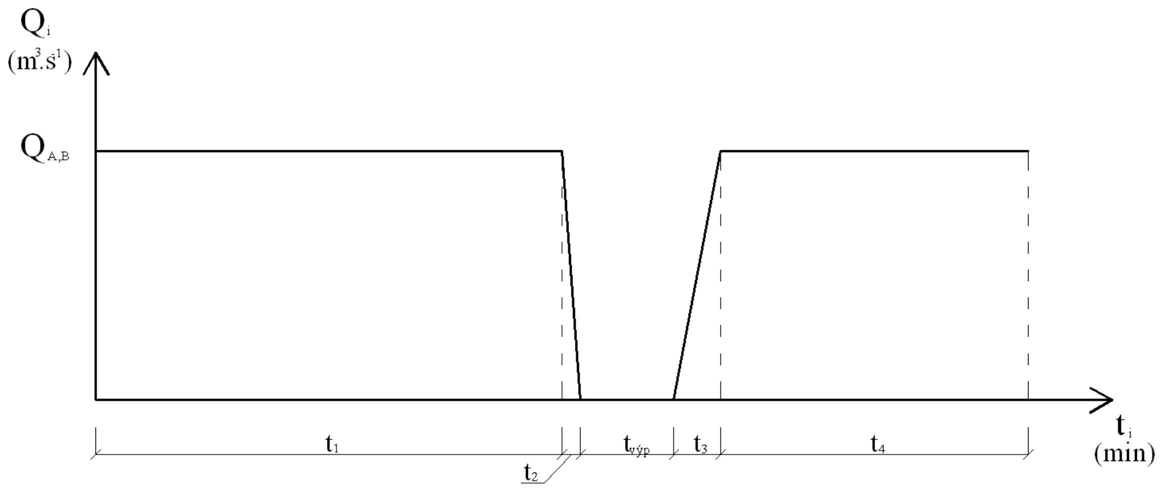

Fig. 12 The scheme of uncontrolled outage of the HPP Gabčíkovo

Tab. 1 The illustarion of tabular treatment of simulation – uncontrolled outage of powerplant without elimination

Var. č.	$Q_{A,B}$	t_1	t_2	$t_{výp}$	t_3	t_4	K_1		K_2	K_3	K_4
	($m^3.s^{-1}$)	(min.)					(m)	(m Bpv)			
1	750	1440	2	60	10	1368	0,6	108,53	108,14	106,63	104,55
2	750	1440	2	180	10	1248	1,42	107,71	107,41	106,06	104,22
3	750	1440	2	300	10	1128	-	-	-	-	-
4	3750	1440	2	60	10	1368	1,19	112,65	112,21	110,52	108,10
5	3750	1440	2	180	10	1248	1,19	111,43	111,01	109,74	107,66
6	3750	1440	2	300	10	1128	1,19	110,34	109,99	108,89	107,01

In the variant number three of the simulation there was a failure the software for extremly decrease of discharge during five hours emergency outage of the powerplant. In the variant number one on the limited ford (K3) there decreased water level about 1,07 m below the minimal level and in the variant number two there decreased about 1,64 m. Just below the powerplant (K1) there decreased water level about 1,42 m during one hour, therefore the vessels in this section are threatened. In the variants number four, five and six water level decreased about 1,19 m during 21 seconds. This decrease is quite steep and it threatens bound vessels.

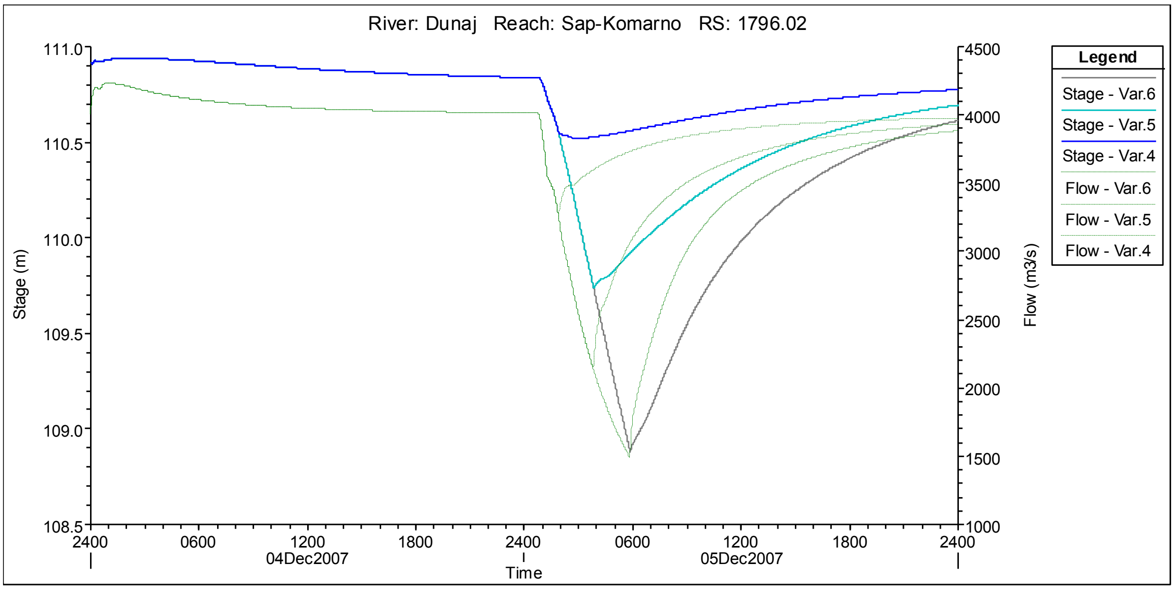

Fig. 13 The illustration of graphics treatment of modeling results

Other groups of simulations were similar prepared. A large quantity of information obtained from mathematical modelling are appropriate basis for negotiation with aim to solve the described issue.

4 CONCLUSION

Mathematical modelling of extreme changes in flow provides appropriate tools for solutions critical situations such as emergency outage and allows to modify manipulation order for the Waterworks with the main goal to achieve increase safety level to the Waterwork Gabčíkovo. Today development of methods of mathematical modelling allows application of 3D modeling of water flow close the outlet of the HPP Gabčíkovo. Future of modelling of the extreme flow changes we can see in a suitable combination of 3D, 2D and 1D mathematical models.

Acknowledgements

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FIELD RESEARCH FOR THE PURPOSES OF THE FLOOD RECONSTRUCTION IN THE PÍLA VILLAGE

T. Pindjaková[1] and S. Kelčík[2]

Abstract

Year 2011 is characterized as dry with uneven distribution of rainfalls in each month from the hydrological point of view. But there were also some important hydrological events in form of the flash flood. One of such events was the flash flood on the river Gidra in the village Píla, which is situated below the Low Carpathians. To avoid future devastating consequences of such floods is necessary to do a correct evaluation of the flood and design solutions, which will be able to catch a flood and protect people and their properties. One of the points, which precede the designing of the flood protections in specific area, is field research. Information about the potential flood area is learned from the field research. The field research was made in the village Píla in the river Gidra basin for making the reconstruction of flood in 2011. Reconstruction of the flood will be provided for making of flood protection in the area of the Low Carpathians and for more efficient and more specific forecast of flood events in the region Červený Kameň.

Keywords

Flood, field research, numerical model, the river Gidra, village Píla

1 INTRODUCTION

The main aim of the field research was to collect information in micro region Červený Kameň (Fig. 1), which included 9 villages situated under the Low Carpathians (Častá, Píla, Vištuk, Štafanová, Báhoň, Budmerice, Jablonec a Doľany). Field research was made for the purposes of the water flow in open channels research that is getting emphasized in view of the possible landscape risks during extreme hydrological regimes, which are followed by floods.

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On 7 Jun 2011 big storm with heavy rainfalls swept over the Low Carpathians and it caused flash floods on adjacent streams with devastating consequences. Fortunately there were no fatalities despite huge damages including some stripped road bridges [1].
The measurements were focused on the river Gidra that flows through the village Píla (Fig. 2.), where the flash flood was occurred in 2011. The field research was done in the framework of the program that support young researchers of Slovak University of Technology in Bratislava while doing the project "Field research and data collection for the determination of interaction between river and groundwater in micro region Červený Kameň". Leica Viva GNSS equipment, digital levelling instrument Leica Sprinter and Flowmate device were used in this field research.

2 CHARACTERISTIC OF PÍLA VILLAGE

Píla village (Fig. 2.) belongs to the district Pezinok, which is located northeast of the capital city of the Slovak Republic. The village stretches about 2 km through the valley. This valley begins in the part Píla - Lindava (crossroad) that is located on the halfway between the villages Častá a Dubová. Píla is situated in a protected landscape area of the Law Carpathians (CHKO Law Carpathians) on the south-eastern edge of The Law Carpathians. Altitude of the village is 245 meters above the sea level. River Gidra flows through this village [2].

3 RIVER GIDRA

River Gidra belongs to river basin of the Danube River and starts in the Law Carpathians under the Baďurka mountain (547,4 masl) in altitude of about 470 masl. Close to Sládkovičovo Gidra mouths to the Dudváh. Mostly it flows south-easter direction. At first it flows around the recreation settlement Biela Skala, where Gidra connects small reservoir and then it flows through the Kobyla valley. Behind the settlement from the left side small stream Pajdla flows in the Gidra and at the end of the valley (next to the Horná Píla grove) from the right side Kamenný stream flows in it. Gidra flows through the area where several recreation cottages are situated, then through Píla village and it enters the area of the Danube Plain. On the right bank Gidra bypasses original forest community of the Danube Plain (PR Lindava) and on the left bank wetland communities (PR Alúvium Gidry). It connects water reservoir Budmerice (bigger one) and smaller reservoir Hajíček. Then Gidra flows through the urban area of the village Budmerice, where Štefanovsky stream flows in it and it continuous around Jablonec village. After Jablonec it begins to meander and it divides the channel (158,5 masl). It passes settlement Jarná and flows into the territory of the area of Cífer village. It flows through the small lowland with small pond and it creates sharp bent to the north and flows around archaeological site Pác. Gidra continuous around Slovenská Nová Ves, where it creates other bent, and then it flows around edge of the lowland forest and south edge of the village Voderady. The river Ronava (122,1 masl) flows into Gidra next to the Pavlice village. Bed of the river extends and flows next to the village Abrahám [4].
Slovak Hydrometeorogical institute (SHMI) provides gauging station Píla, in the village with the same name [5].

4 MORFOLOGY CHARACTERISTC OF THE RIVER GIDRA

The river Gidra has various characters. Above the village Píla (at the upper stream of the river) it is unbroken torrent, but in the village we can see local river bed design. Down under the village Častá has river Gidra concrete channel and in the middle and lower part of the river regulated and unregulated parts are being alternated [6].

Leica Viva GNSS devices and digital levelling device Leica Sprinter were used for the accurate determination of morphological characteristics of the river Gidra. With these instruments we identified positions of cross sections points of the Gidra River, then bridges and also characteristic points of terrain (such as road, slopes and surrounding terrain). The field measurements helped for the best integrating of the river to the digital terrain model (DTM). Measured data were processed in the program Hec-Ras 4.1.0 that is used for the 1D modelling of flow in open channels [7]. To create an accurate numerical model it was necessary to find all information about the water buildings that are located on the river Gidra, and also bridges, which may affect the water regime during extreme hydrological situations. The output from the Hec-Ras software is in Figure 3. and it shows the 3D multiple cross section of the river Gidra in the village Píla.

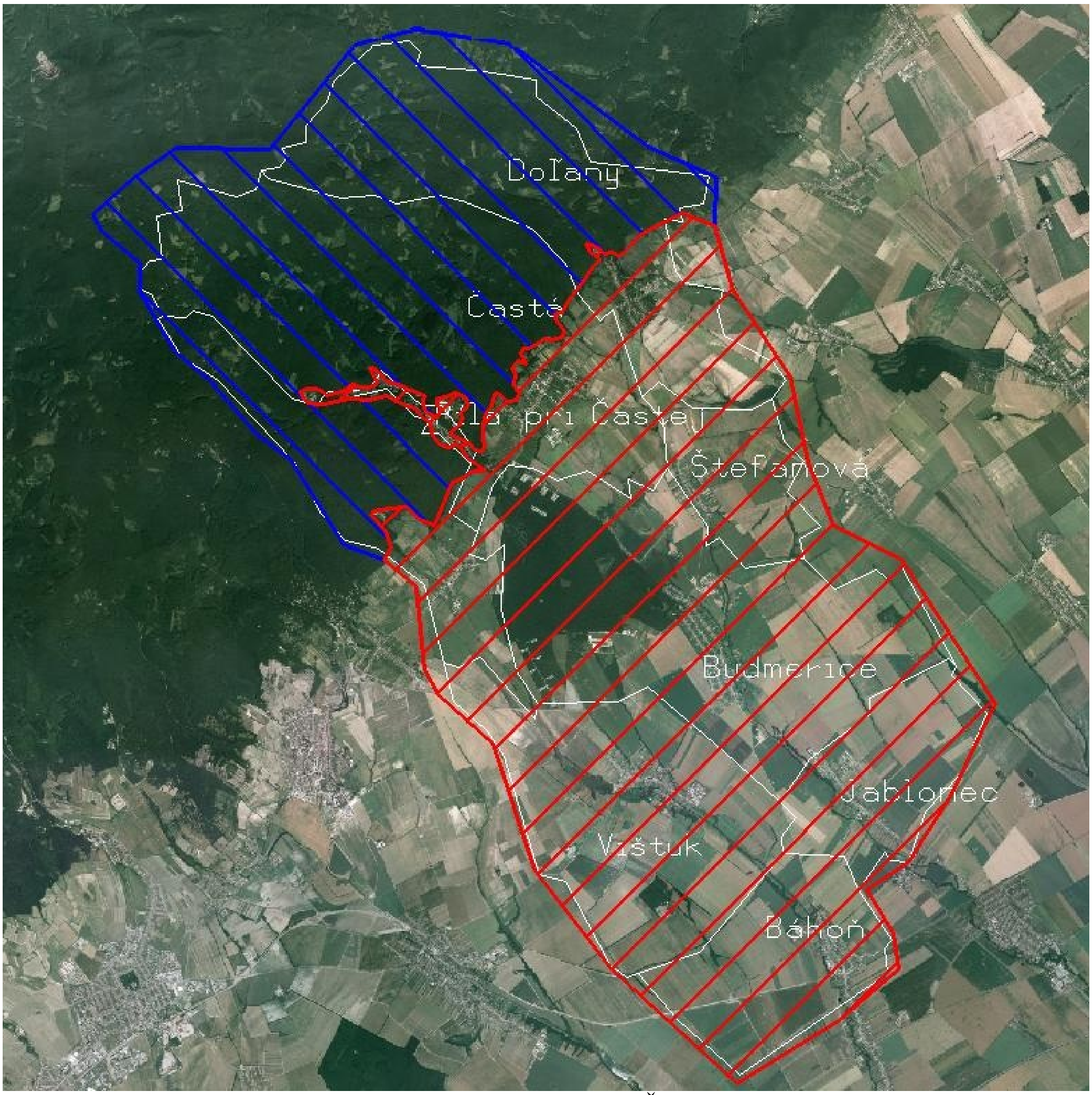


Fig. 1. Area on interest (micro region Červený Kameň) [3]

Fig. 2. Píla village [3]

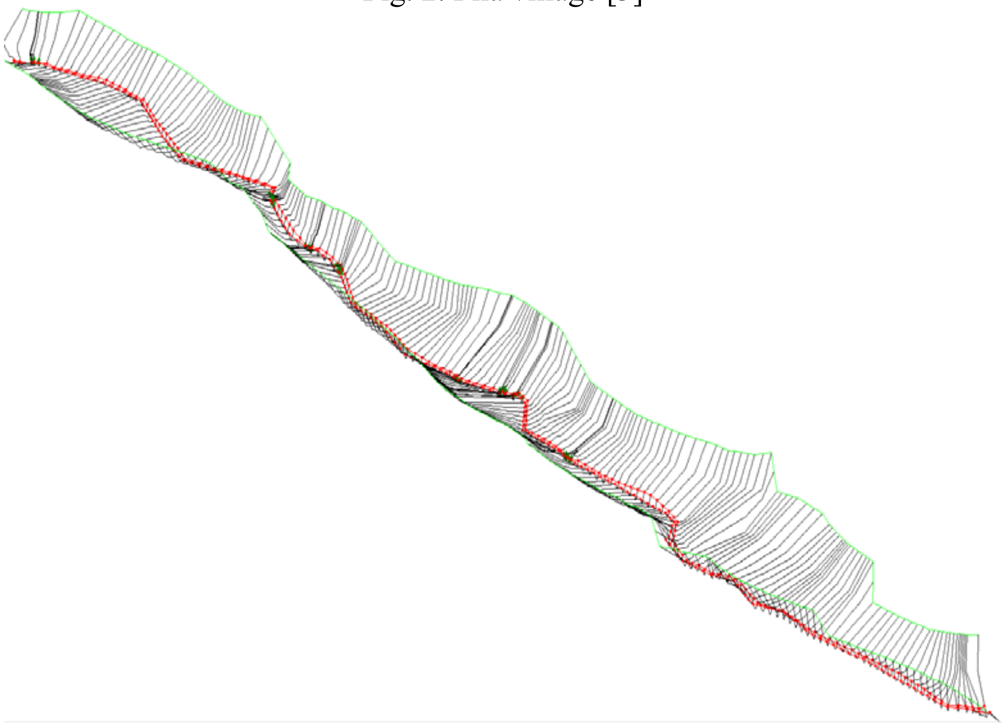


Fig. 1. 3D multiple cross section (Hec-Ras)

There are 9 bridges which go over the river Gidra. The numerical model will be done for the extreme hydrological period (i.e. flash flood from June 2011) so it was necessary to carefully measure all the bridges, walkways and barriers, which go over the river Gidra. While field measuring was being made (April and May 2013, it means 2 years after the flash flood) all bridges were after reconstructions (some of them were after partial and other after total reconstruction). So we had to modify the bridges as they used to be before flood. As an example Figures 4. and 5. shows the first bridge that is identical to the bridge which existed there before flood. In the partially reconstruction only the height of it was increased exactly of 50 cm, but the same structural elements were used.

First part of the field research was focused on the getting of morphological data of river Gidra, because these data are necessary as an input data to the Hec-Ras software and for creating 1D model, which will be the best corresponding to the reality.

Fig. 4. Reconstructed bridge in the village Píla (May 2013)

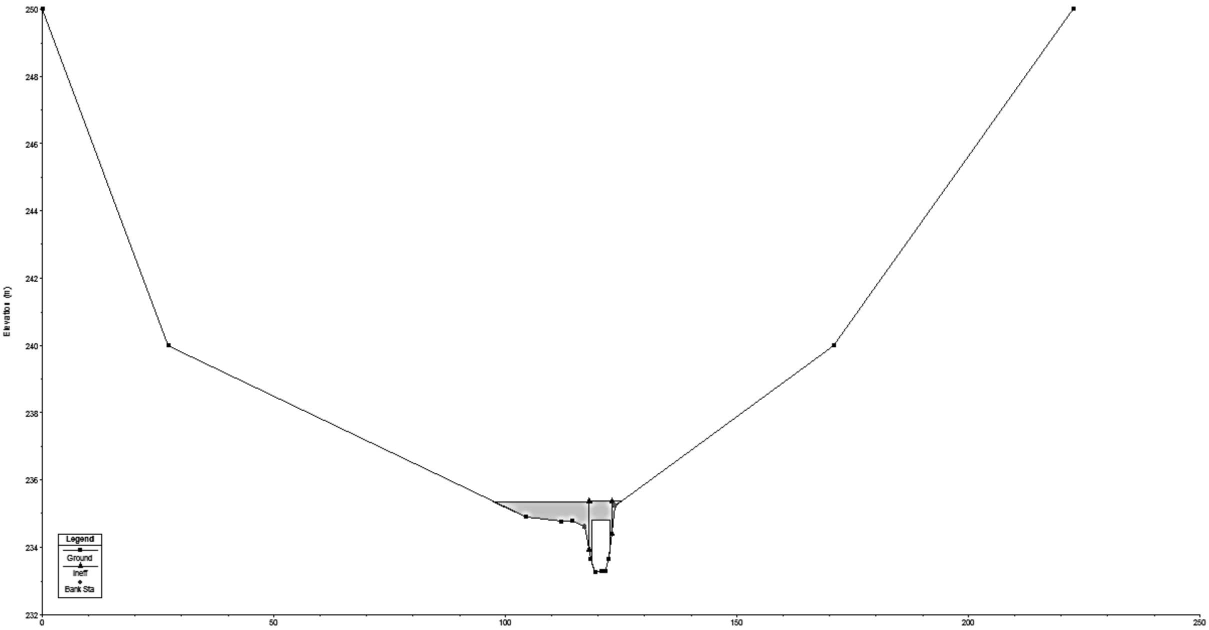


Fig. 5. Cross section of reconstructed bridge (Hec-Ras)

5 HYDROLOGICAL DATA

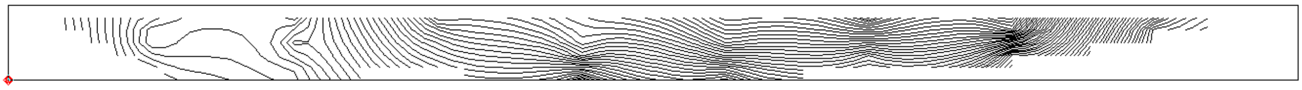
After the evaluation of all morphological data and data of surrounding terrain that were available we proceed to the second part of the evaluation of the field research. This was focused on hydrological data. In the village Píla there is only one gauging station. There are continuous measurements of the water levels recorded. For making numerical model of flood

reconstruction we have data about these water levels in the river Gidra from Slovak Hydrometeorological institute [5]. Situation of gauging station cross section with marked river basin of Gidra is shown in Figure 6. Short characteristic of the river Gidra basin is shown in Table 1. [8].
While measuring cross sections we measured also water levels in these cross sections. Cross section of the river Gidra was measured in the location of the gauging station Píla (Figure 7. And 8.).

Fig. 6. River basin of the river Gidra with marked gauging station [8]

Flow of water levels is basic input information for making numerical model of flood, but also with other data which are measured right in the field and with values of discharges measured during that water regime. Therefore we further focused (in framework of the field research) on the river velocity measurements that we used for evaluation of the river discharge ($m^3.s^{-1}$). Velocity was measured with the Flowmate device and as a software for discharge evaluation was used Surfer 8. Surfer is a contouring and 3D surface mapping program that runs under Microsoft Windows. It quickly and easily converts data into outstanding contour, surface, wireframe, vector, image, shaded relief, and post maps. Virtually all aspects of maps can be customized to produce exactly the presentation we want [9]. The average year discharge of the river Gidra is 0,85 $m^3.s^{-1}$. In the gauging station profile Gidra – Píla was culmination discharge calculated to about 63,6 $m^3.s^{-1}$ and then modified with other input data and field research to final 44,5 $m^3.s^{-1}$, by workers of SHMI [8]. Our calculations of the river Gidra discharge corresponded to the hydrological situation which was during the field

measurements (24.04.2013). It was calculated to 0,73 $m^3.s^{-1}$. Velocity isolines are shown in Graph 1. Graphical output that shows dependence x, y, z (x = width of the profile, y = velocity of river and z = depth of the river) is in the Figure 7.
Obtained hydrological data will further used as a boundary and calibration parameters for the simulation of unsteady flow in the software Hec-Ras.



Graph 1. Velocity isolinies (Surfer 8)

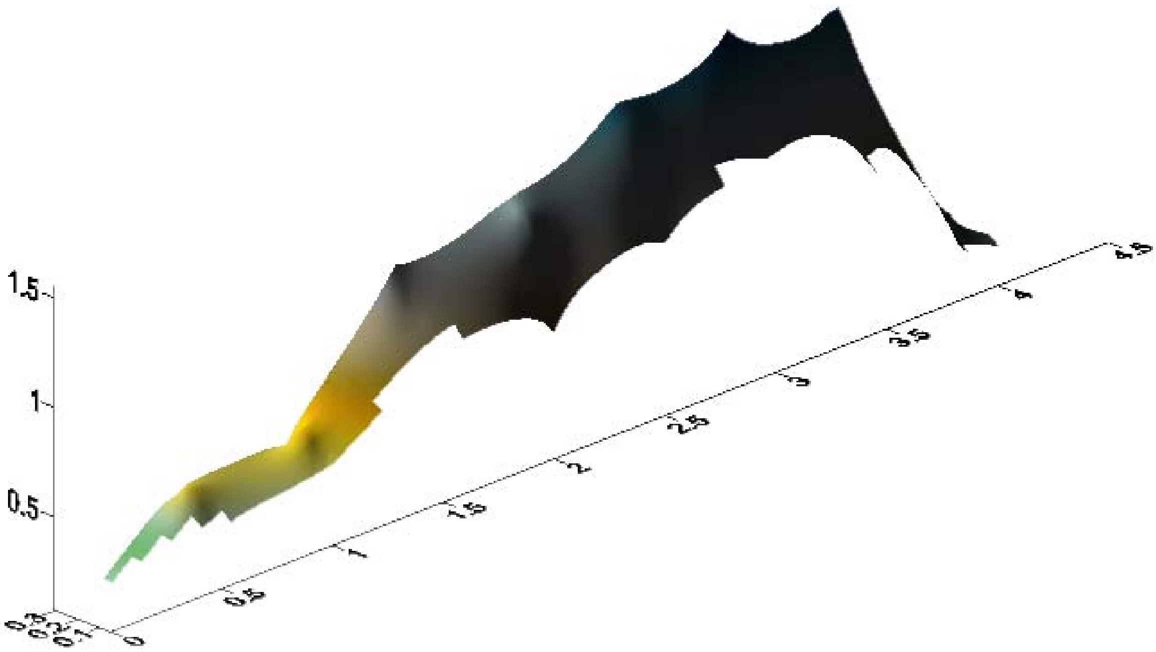


Fig. 7. Graphical output (Surfer 8)

During the flood situation the gauging station wasn´t out of service fortunately. But it was necessary to edit information on flood conditions, because the river Gidra flew of the river bed and gauging station couldn´t capture the situation in the way it was. We had to consider also information which was provided by mayor of the village Píla and random witnesses (locals). These data are often only one and in present time (when there is large availability of mobile phones and cameras) can be considered as a reliable. Until now we can see the flood marks on the houses in the village. All of those available marks were measured with the GNSS device and will be used as verification parameters in the numerical model of the flood reconstruction. An example of this mark is shown in the Figure 10. and in the Figure 11. is longitudinal section of the river Gidra in the village Píla shown also with the flood marks that will be used to verify model.

Tab. 1. Characteristics of the river Gidra basin [8]

River	Profile	Hydrological number	River basin area [km^2]	Forests [%]	Average slope of river basin [°]
Gidra	Píla	4-21-16-038	32,95	97,8	9,93

Fig. 8. The river Gidra in gauging station Píla

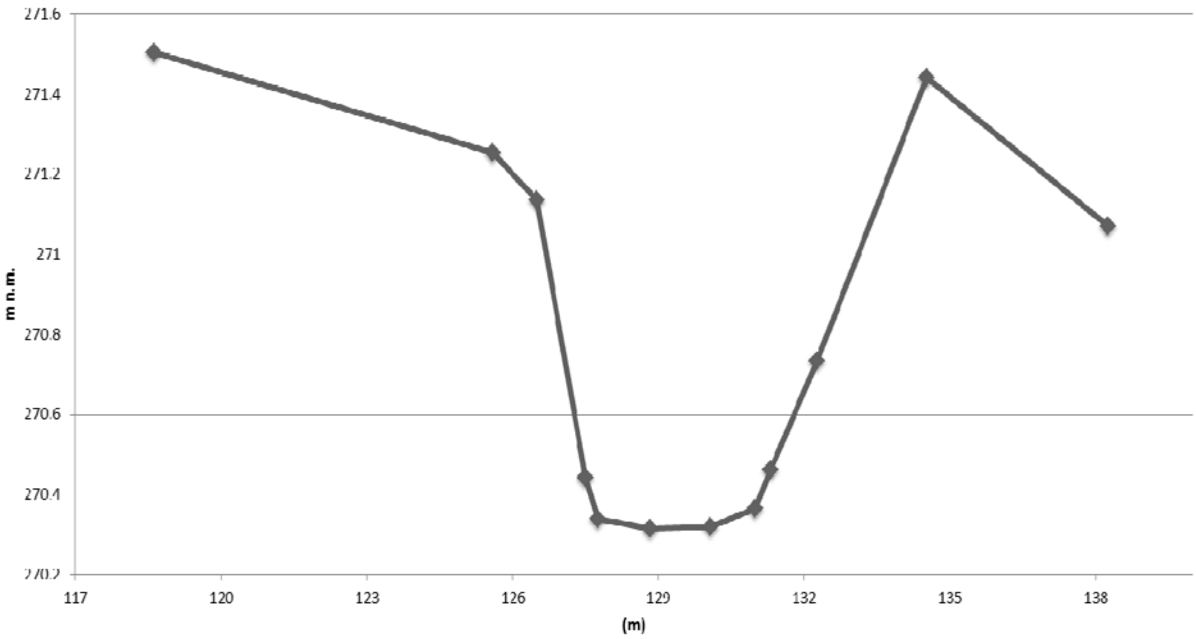


Fig. 9. Cross section of the river Gidra in the Píla gauging station

Fig. 10. Flood mark [10]

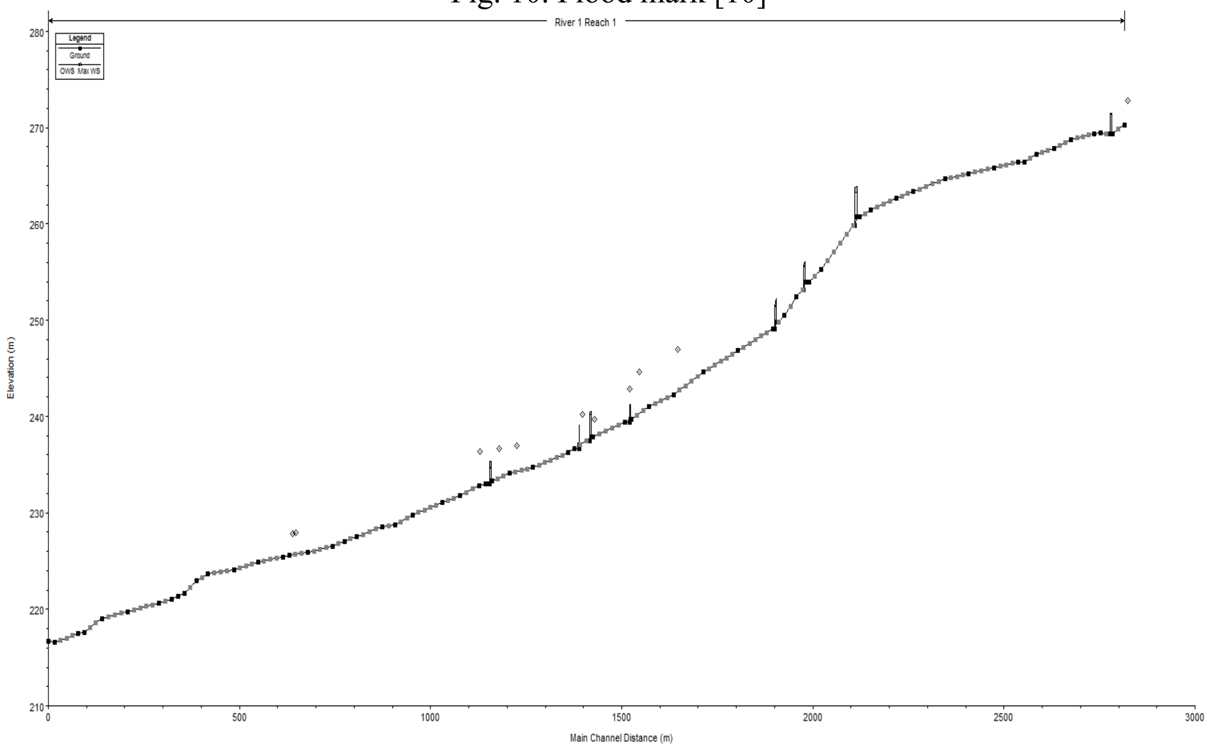


Fig. 11. Longitudinal section with flood marks (Hec-Ras)

6 CONCLUSION

The article briefly describes the evaluation of the field research, which was made for the purposes of creating a numerical model of water flow in open channels. All collected data will be used for making a numerical model of the flood reconstruction in the village Píla, where the flash flood was occurred (June 2011). During the field research were hydrological data about the river Gidra collected, then morphological data about this river and also data about the surrounded terrain. It was also necessary to measure all of the buildings and bridges that are situated on the river, because they may influence water regime while extreme hydrological situations. We got information from the locals, who were witnesses of the flood and mayor of the village Píla (RNDr. Ing. Mgr. Radovan Mičunka, PhD.) provided us important information about the bridges and situation in the village before the flood. Flood marks will be used for the model verification. These marks can be seen on the houses until now (Fig. 10.). The result is the 1D model (created in the program Hec-Ras) and is ready for simulation of the water flow in open channels.

ACKNOWLEDGEMENT

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STRUCTURES IMPROVING PASS-ABILITY OF SMALL HYDROPOWER PLANTS

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Abstract

A small hydropower plant (SHPP) project includes structures for overcoming the difference in water levels upstream and downstream the SHPP. These structures are fish passes, which enable the passing through the SHPP for fish and other aquatic organisms, and small sport navigation structure, which enable continuous shipping through the SHPP. The requirements for such structures often increase the total investment cost for SHPP, which leads to slowing or in some cases to a complete stop of the building processes of SHPP. This paper describes different types of these structures suitable for SHPP in Slovakia as well as a possibility of a combined structures joining together both of these objects, which may decrease the total investment costs and still provide the required pass ability of a SHPP.

Keywords

Small Hydro Power Plant, Fish-Pass, Canoe Shute

1 INTRODUCTION

In relation with the construction of small hydro power plants (SHPP) in Slovakia, comes to fore the construction of associated objects that are not necessary for their operation itself, but without including such a structure to the construction is the building of a new SHPP on our streams practically impossible. These objects are designed to ensure continuity of water streams or rivers, which was interrupted by the construction of small hydropower plants. Such discontinuity of rivers is concerning particularly living organisms, for which the river is their natural environment (fish, aquatic animals, bottom organisms, etc.). At the same time, small hydropower plants affect at recreationally attractive rivers the recreational and sport sailing (especially water tourism), where the SHPP creates a barrier for canoeing down the river.

Objects that allow overcoming the barriers on rivers created by main structures of SHPP and so keeping the continuity of rivers at SHPP can be divided in two groups – fish passes and sport navigation structures. For building new SHPP is the construction of these objects particularly problematic. The current valid legislation dealing with these issues, both in terms

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of water management or waterways, clearly defines for new investors and operators of existing SHPP to ensure the continuity of rivers for living organisms and based on the waterway class of the particular river it defines the level of navigation pass ability of the SHPP.

From the perspective of an investor of SHPP these objects are making a significant portion of total investment costs and they do not contribute to the generation of electricity and thus they are not necessary for the operation of SHPP, on the contrary they reduce the possibility of SHPP to use entire hydropower potential of the river (discharge, which cannot be energy used). As a result, they are substantial burdens for the investors, which are negatively reflected in the overall economy of SHPP; mainly caused by the investment costs for building of these objects, their operation and maintenance costs and reducing the possibility of using entire discharge in SHPP. The field of fish passes and sport navigation structures is quite well explored and supported, especially abroad. By accepting such an experience it is therefore possible to build a functional object, which will ensure the river continuity. In many cases, however, especially by state organizations (and civil society organizations as well) involved in the processes of the approval procedure, there are placed unreasonable, impossible and often conflicting requirements for such objects. These requirements mostly come from the ignorance of technical solutions but also principal prejudice against the SHPP construction. Such requirements on these associated objects often lead to the stopping of the permitting process.

In the following we will try to describe the basic characteristics of the various types of such structures and outline a possible compromise in their construction, which could meet the requirements and still would be economically viable.

2 FISH PASSES

Fish passes are used by migrating fish, which live their lives partially in the sea and partially in upper reaches of rivers and streams (salmon, eel, sturgeon). At the same time they are used by river fish (carp, pike, catfish, etc.) to return to their original habitats when they are torn down by floods or are moving to higher reaches with more suitable living conditions in terms of water purity, quantity and type of food, etc. Their structural solutions allow the fish to migrate around a barrier on the river by swimming or jumping through a series of relatively low steps to the other side of the barrier [1].

Fish passes allow aquatic animals to overcome the difference in water levels upstream and downstream the SHPP. Their principle is to create a short by-pass of the SHPP by extending the line between these water levels, thereby reducing the flow rate and the longitudinal slope between these water levels. In addition, flow velocities in this by-pass are reduced by placing of rows of obstacles of different materials forming successive chambers with low mutual differences in water levels. Specific parameters of fish passes directly depend on species of fish and bottom organisms living in a given river reach.

In general, the fish passes can be divided to nature-like fish passes, which consist largely of natural elements (coarse gravel, stone, wood) and technical fish passes, formed by a concrete channel with slots of different materials (concrete, wood, plastic).

The nature-like fish passes include boulder chutes (Fig. 1), boulder (migration) ramps (Fig.2) and by-pass channels (by-passes) (Fig. 3). Boulder chutes and ramps are built in the own riverbed, they are made by rows of boulders and shade areas which are usually permanently fixed in the concrete bed, which is also a stabilizing part of the whole structure. The difference between them is that the chutes are being built across the entire width of the river, while the ramps are a part of the dam and they take just a part of the river width. Bypass channels are also constructed of obstacles such as boulders and shade areas, but unlike the chutes and ramps they are built on the river bank sides making a separate channel bypassing the water structure in the riverbed.

Fig.1 Boulder chute [7]

Fig. 2 Boulder ramp [7]

Based on their nature, the nature-like fish passes fit well into the landscape and they create a close to natural habitat, which is easily passable for fish and aquatic organisms. Their disadvantage is their discharge requirements and the need of sufficient depths of water especially during dry seasons. These bypass channels generate considerable territorial requirements on surrounding lands.

Fig.3 By-pass channel [8]

If the spatial and technical design of a water structure allows it, it is advantageous to build nature-like fish passes. In many cases, however, mainly due to legal problems with surrounding lands and due to limited discharge conditions, when the discharge has to be precisely controlled (e.g. when increasing the amount of discharge for SHPP), the fish pass has to be included in the body of the water structure itself.

In such cases, technical fish passes are used. They are suitable for higher heads and in terms of their cross section requirements they require minimum space. The typical used types are Denil´s fish pass, pool fish passes, slot fish passes and bristled fish passes.

Denil´s fish pass or Alaskan steeppass (Fig. 4) consists of a linear channel, in which baffles are arranged at regular and relatively short intervals, angled against the direction of flow. The backflows formed between these baffles dissipate considerable amounts of energy and, because of their interaction, allow a relatively low flow velocity in the lower part of the baffle cutouts. This allows the Denil pass to have a steep slope, relative to other types of fish passes, and to overcome small to medium height differences over relatively short distances. The compact construction of the Denil pass and the possibility of prefabricating the pass in dry conditions and installing it once assembled makes this type of construction particularly suitable for retrofitting of existing dams, that do not have a fishway, and for use where there is not much space [6].

Pool fish passes (Fig. 5) are one of the oldest types of fish passes. Their structure is created by a concrete channel with wooden of concrete walls across the channel creating a system of equal consecutive pools. At the bottom and the upper edge of the walls, there are openings through which the water flows from one pool to another and the fish can move (they jump from pool to pool). Due to the higher possibility of clogging of the openings, the pool fish pass requires increased control and monitoring. A good design of the openings and pools is important in order to create sufficient flow in the fish pass which can be quite challenging. With a sensitive design the pool fish pass is suitable for small and medium-sized heads and is passable for all fish species.

Fig. 4 Denil fish pass [9]

Fig. 5 Pool fish pass [author´s archive]

Slot fish passes (Fig. 6) are verified to be functional and are accepted when nature-like fish passes cannot be built. They are created by a concrete channel with concrete or wooden walls, which contain one or two vertical slots through which the water flows. Fish overcome the water level difference by swimming and are not forced to jump. They are suitable for relatively low and high discharges, but they need sufficient depths in the channel. By adding the bottom substrate they can be passable for all kinds of fish as well as for benthic organisms.

Bristled fish passes (Fig.7) are relatively kinds of fish passes. They consist of a concrete channel in which rows of blocks of plastic bristles are placed. The flow character is similar to the flow in slot fish pass but the water is not flowing just between the blocks, but also among the bristles, giving rise to excellent hydraulic conditions for fish. After adding the bottom substrate, these fish passes are passable for all kinds of aquatic organisms. Their advantage is in the fact that thanks to the flexibility of the bristles and at sufficient width they are suitable for canoes as a canoe pass. For high heads they can be combined with other types of fish passes [1].

Fig. 6 Slot fish pass [10]

Fig. 7 Bristled fish pass [11]

3 OBJECTS OF SPORT NAVIGATION

Among the rivers most interesting for water tourism in Slovakia belong the Moravia, the Little Danube, Hron, Orava, Vah, Dunajec and Bodrog. Based on the navigation on rivers and their waterway class it is possible to define the type of vessels that occur on the particular waterway. The type of occurring vessels determines the types of structures for navigation and their design parameters. In the construction of small hydropower plants in Slovakia, recreational water tourism is considered, particularly with the use of small non-motorized vessels, such as canoes.

Objects for non-motorized small boats on rivers should create the conditions for landing, getting the boats in and out of the water or overcoming existing weir of SHPP. Technical subjects dealing with overcoming navigation barriers (weirs, SHPP) are determined by the character of the river and anticipated types of sports and recreational boats.

The simplest technical objects are:

- simple reinforced banks with moderate slopes,

- sloped ramps going under the water level and side slopes with stone tiles. This solution enables the launching or hauling of a small motor boat,

- concrete or stone stairs, built in a slope.

In terms of SHPP operation are these technical navigational solutions easy to construct, maintain and operate, and have no requirement for water flow. However, in these objects interrupt the continuity of the navigation and vessels must be manually carried over the barrier, which greatly reduces the attractiveness of water tourism and does not meet the continuity requirements. Therefore, these objects are for SHPP very unpopular.

The most used objects for maintaining continual shipping on SHPP are:

- Small locks,

- Boat lifts,

- canoe chutes (Fig.8).

Locks are designed for such barriers, where heavy traffic of large pleasure crafts that such as yachts, sailboats and motorboats can be expected. In our conditions the construction of such navigation objects at small hydropower plants due to the nature of rivers and the high investment costs is unrealistic.

Boat lift is a device that carries a vessel over the structures of SHPP on a boat carriage along the connecting road. It consists of a loading ramp, a loading the bridge at the upstream and downstream water side and a connecting road. Loading ramp is built on the upstream side, so that even at the lowest water levels the boat carriage is allowed to enter under a floating boat. The loading bridge is located on the bank side, it is used to bind the vessels when loading or unloading the boat carriage. The carriage is a robust, lightweight construction, equipped with a safety brake. On steep slopes and for large boat weights, a winch with recirculating rope is used for hauling the carriage. With the boat lift all types of small boats can be carried except for large yachts and motorboats.

A canoe chute allows overcoming a barrier in the river in a short time and without interruption of shipping. It is designed for small boats and vessels with a width of up to 2.1 m. Over a specific time more vessels with crew and cargo can pass through such a chute that though a ship lift or a lock chamber. For safety reasons a pedestrian walkway is considered along the chute. Children and less skilled individuals can overcome the object on foot.

A canoe chute consists of inlet, chute and outlet. The inlet can be equipped with gates which allow controlling the discharge in the chute. Without the gates the discharge cannot be controlled. The canoe chutes are placed on the bank side, and at SHPP they are placed on the opposite side to the SHPP. The chute is usually a concrete channel with an orthogonal cross section of constant width, with a longitudal slope of 1:20. For increasing of water depth and creating stabilizing flow conditions that keep the boats in the middle of the chute, V-shaped segments are placed at the bottom of the chute. The side walls are designed to be low [3].

Fig. 8 – Different canoe chutes [12], [13]

4 A COMBINED OBJECT OF A FISH PASS AND A CANOE SHUTE

The bristled fish pass is a rather new type of a fish pass, which can be used as well as a canoe chute. The character of the chute is similar to the slot fish pass, but the slotted wall are here replaced by blocks of bristles which increase the roughness, similar to the stones and vegetation in nature-like fish passes allowing. In addition, the bristle fish pass is suitable for small boats and canoes, since the bristles can direct the flow, while being flexible enough not to damage the boats resulting in their overturning (Fig. 9, 10). The functionality of the system in practice was demonstrated in a pilot project of Au-Schönenberg water structure (hydropower plant) on the Thur River [4]. Flexible and permeable bristle elements are substitute for the previously used concrete elements. The bristles elements consist of individual bundles firmly anchored in a concrete base plate on the chute’s bottom. The placement of these elements provides meandering flow and creates flow shadows, which provide shelter and relax to fish. The bottom tray is filled with coarse substrate (gravel), which makes the bristle fish pass suitable for bottom organisms.

The layout and installation of rough bristle elements depends on the local conditions of a particular water structure (SHPP) such as discharges in river, flow velocities, head or the expected number of fish. The bristle chute channel width and water depth in it do not affect the flow velocity, thus a constant flow velocity is maintained at a constant longitudinal slope of the chute. Laboratory tests with live fish also showed that bristle fish pass is passable for all species of fish in both directions. The advantages of a bristle fish pass are:

- energy conversion takes place in a short section with low turbulence, thanks to the bunches of bristles, which increase the roughness,

- velocity between the bristles depends only on the difference in water level between the rows of bristles and is less than 1.2 $m.s^{-1}$, at an logitudal slope of the chute bed of 8 to 10%

- places in the flow behind the bristles (with a maximum velocity of 0.3 $m.s^{-1}$) are used by fish as resting places (the refuge)

- hydraulic properties are largely independent to the shape and layout of the chute

- the effects of the flow on the coarse bottom substrate is negligible due to the low flow velocities between the brush elements, making it passable for bottom of organisms as well [5].

Fig. 9 Bristled fish pass combined with canoe chute [14]

The bristle fish pass build as a combination of two objects (a fish pass and a canoe chute) appears to be an acceptable solution of overcoming a barrier in the river created by an inline water structure such as a small hydropower plant. It provides sufficient pass ability for fish and other water organisms and still a secure possibility to continually overcome the barrier in small boats and canoes. The construction of this object is at affordable terms without significant impact on the water structure’s layout and acceptable amount of investment costs.

5 CONCLUSION

Objects of small hydropower plants that ensure the migration continuity and navigability at SHPP are necessary parts of such structures (depending on the specific nature of the SHPP site). Their construction and operation, however, make considerable financial burdens to the investment process and the actual operation of the SHPP (investment costs for standalone objects, permanent water consumption, which reduces the usability of the discharge in SHPP, thus it reduces the amount of electricity produced by the SHPP).

The possibility to reduce the investment and operating costs of small hydropower plants is to build structures combining the object of a fish pass and a canoe chute. Such an object will reduce the total cost of investments and the requirements on discharges for non-energetic purposes for SHPP. However, in our conditions the realization of such objects is difficult mostly due to the opposition of the State authorities, such as the State Nature Protection and Slovak Fishing Association, despite the fact that these objects are commonly used abroad.

Therefore an appropriate step to take would be to realize a complex long-term research on pilot projects of such objects, in order to objectively assess their functionality and in cooperation with concerned authorities and organizations to develop a methodology for their design.

Acknowledgements

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COMPLEMENTARY MODELLING IN HYDRAULIC ENGINEERING – A CASE STUDY

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Abstract

In hydraulic engineering, especially in river engineering, different types of hydraulic models are applied, such as numerical and distorted physical models. The model selection is dependent on the problem itself, its scale and data availability, and thus, it is essential for the further investigation program. Of course, this affects the efficiency of the work flow as well as the reliability of the results. While the efficiency deficits usually cause only higher effort, wrong results may cause serious subsequent damage (e.g. inadequate flood protection measures provoke damages). Thus, the accuracy and reliability of a model are the most important aspects for choosing an adequate hydraulic model.

The presented paper highlights a combination of different modelling techniques and illustrates them by two case studies in the field of basic research and of river engineering.

Keywords

Complementary model approach, hydraulic engineering, hydraulic model, numerical model, physical scale model, river bend flow, sediment transport

1 INTRODUCTION

For projects in hydraulic engineering either numerical and/or physical models are applied to predict flow and sediment transport processes. Until two decades ago, the capabilities of numerical flow models were limited due to the limited computational power and data handling. In that time physical scale models were the common instrument to simulate flow and sediment transport in rivers. Today the progress in computer technology allows new areas of application in hydraulic modelling. Due to cheaper numerical tools the labour intensive

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scale models are under pressure. However physical and numerical models still offer different pro and cons. Hence it is necessary to identify areas for preferential application of either physical or numerical models.

According to Novák [1], we can distinguish between different simulation methods. Simulation may be direct, semi-direct, or indirect. Examples for direct simulations are scale models, where the flow is simulated directly by a fluid. Semi-direct simulations like analogue models base on similarities of different physical processes (e.g. electric currents and water flow), or just use a different medium, as aerodynamic models. On the opposite, there are indirect simulations like mathematical models, numerical models or computational models. Figure 1 gives an overview on the different types of simulation techniques.

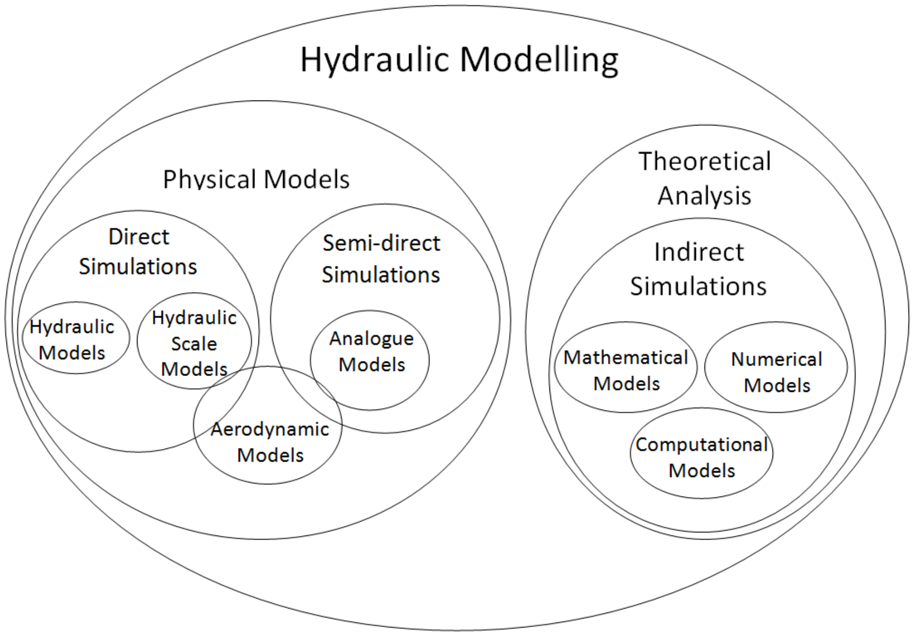

Fig. 1 Classification of models in hydraulic engineering

As it can be seen in the Figure 1, the hydraulic scale models and the computational models belong to absolutely different model groups and also the functionality is dissimilar. While in physical modelling similar physical processes are observed and interpreted, on contrary computational models base on a theoretical analysis of the assumed processes. Thus using physical models requires know how in similarity law, while using computer models implies knowledge of the occurring flow processes. Consequently, selection of the modelling method has to be based on the available knowledge and field measurement.

In addition to this differentiation between the possible methods of simulation, also combinations of different model techniques are used. This idea came up very soon, when the first numerical models were developed. So described Holz [2] in 1979 e.g. a computer language for hybrid river modelling. An up-to-date overview on composite modelling can be found in the work by Gerritsen [3]. The basic idea of composite modelling is to combine different types of models to overcome their specific handicaps and to use their advantages cooperatively together. In the Table 1, the limitations of physical and computer models based on Kobus [4] are listed. Maybe some of the limitations of the computer models seem to be obsolete, like the storage capacity or the computational speed. Yet, only the ceiling of what is technologically possible gets pushed farther and so there is still a limitation left.

Tab. 1 Principal limitations of the physical and computer models [4]

Physical Model	Computer Model
Principal limitations	
Model size (size of laboratory)	Storage capacity
Discharge (pumping capacity)	Computational speed
Energy head (pumping capacity)	Incomplete set of equations
Model laws	Turbulence hypothesis
Practical limitations	
Minimum model scale (surface tension, viscosity, roughness, measuring accuracy)	In simplified set of equation - Accuracy of assumed relationships - Availability of coefficients
Model size (upper limitation)	Space and time resolution (lower limitation)
Measuring methods and data collection	Numerical stability and convergence of the solution scheme
Availability of boundary- and initial conditions	Availability of boundary- and initial conditions

Comparison of the limitations of both simulation techniques indicates that the limitation of one model type can be compensated by the capability of the other one. Therefore, a smart complementary application of physical and computer models should be applied, to increase the benefits and to improve the result. Gerritsen [3] already presented different types of composite models, but these methods are mainly focused on interactions between seacoasts and structures. Therefore, an approach of composite models for research and engineering on rivers should be developed. Nevertheless, two examples of feasible combinations of models are presented and discussed below:

- Basic research – visualization of flow processes in a physical model by a numerical model
- River engineering – application of different model types alternatively for pre-testing of variants.

Both cases present a suggestion for the appropriate composite model use and highlight the opportunities for use of this technique in hydraulic engineering.

2 MATERIAL AND METHODS

Bellow, two examples of realized models will be described briefly. Each presented example includes a physical and a similar computer model. Namely, the similar computer model has the same scale as the physical model, whereby no scaling effect could interfere the transfer of results from one to the other model. Basing on this propagation of findings the different models can support each other.

During the physical simulations, velocity, water level and, if necessary, ground elevation were measured. Velocities and water levels can be measured only discretely in time and space. In steady-state condition this may be not such a big problem, because the measurements can be done sequentially. Yet, if the conditions on the model are changing permanently, the measurements provide only a snap-shot of a single point in one specific situation.

For the computer simulation the two-dimensional model "Hydro-GS_2D" was used. This model offers a stable calculation, a powerful user-interface and a reliable bed-load transport model. Therefore, it is often used by river engineers in their projects. By this reason it is a common reference for the computer models used in practice.

2.1. Basic research – visualisation the of flow processes

In this experiment a model to study the aggradation of a reservoir was used. Therefore a rectangular reservoir was constructed with a narrow inflow on one side, while the outflow occurs over the whole length of the other three sides of the reservoir (see Figure 2). By the inflow water and a specified amount of sediment were transported into the reservoir. Due to a radical reduce of the flow velocities in the reservoir the sediment was deposited around the inflow. The input parameters in this experiment are the discharge, the sediment-concentration and the sediment properties (e.g. size, density). The boundary conditions are the geometry of the inflow channel and the reservoir and its overflow edges. The overflow edges determine the water depth in the reservoir. The water depth in the inflow channel is a function of the water depth in the reservoir and of the inflow discharge.

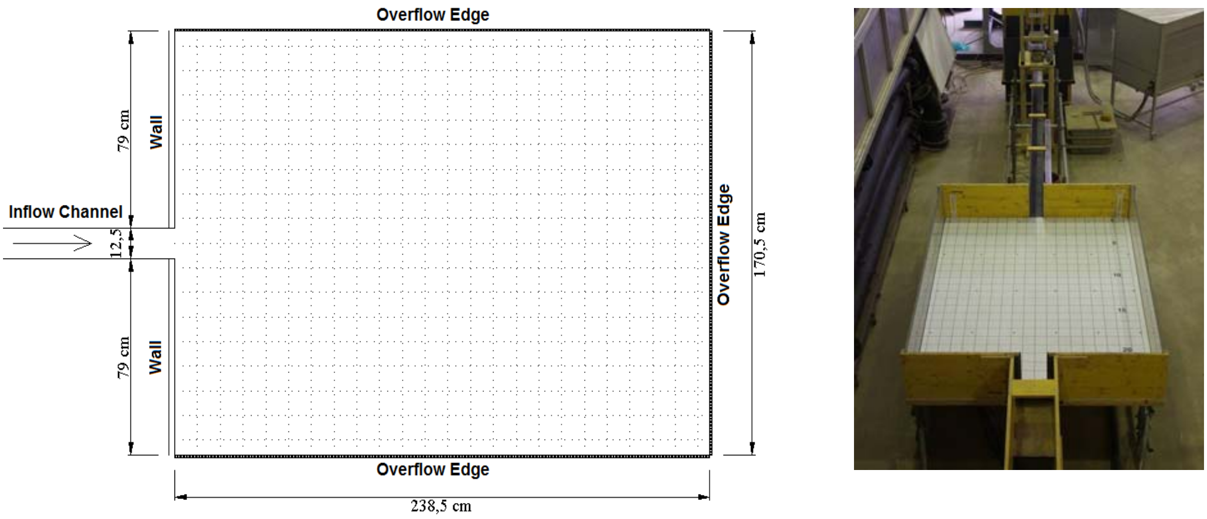

Fig. 2 Scheme and photo of the physical model on the delta development process

The essential measured variables are the flow velocities and the aggradations. The flow velocities are measured by an electromagnetic current meter. Therefore, only punctual measurements are available. Since the system is, due to the expanding aggradations, permanently changing, the flow velocities are continuously changing, too. Thus, the measured flow velocities represent only a snap-shot in a single point. The aggradations could be captured by the moiré methods. Thereby, is it possible to detect the elevation of the ground also through the water [5]. For more information on the moiré methods please refer to Cloud [6], Müller [7] and Post [8].

In this case, the intention is to use the computer model to supplement the measurements on the physical model. Therefore, in the first step the measurements from the physical model are used to calibrate the computer model. If the computer models reproduce the growth of the aggradation, as well as the measured flow velocities, it can be supposed that the calculated variables are similar to the physical model, too. Thus, it is feasible to display the flow processes over the whole testing area by extrapolation of the measured data.

2.2 River Engineering – alternately use of different models for variant studies

In contrary to the former case, this is an example for the engineering praxis. This means, that a practical solution has to be found for a given problem. This general philosophy is illustrated clearly by de Vries [9] in the Figure 3. The essential idea is that the procedure is principally the same for physical and for the numerical models.

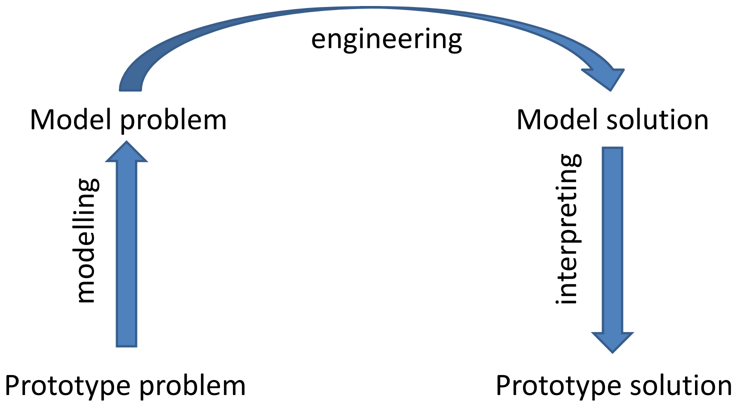

Fig. 3 “Use of models as a detour” by de Vries [9]

Hence also the method to find the optimum solution is similar. Yet, each simulation type has its pros and cons, as it has been presented in the Table 1. So, by use of two different models, each optimisation task can be done in the model where it is easier. Typical small-scale variants can be tested easier on physical models, while large-scale changes can be observed easier on computer models. Thus, if the results of both model types are combined, they become more reliable and the optimisation process of different variants is more efficient.
The aim of the described project example is to improve the flood protection situation in a river stretch. The physical model is presented in the Figure 4. Therefore, different measures seem to be adequate. On the one hand, significant changes of the river bed, and on the other hand, local protection measures.
The significant changes of the river bed include lowering of the bottom, widening the profile in critical regions, or bypass a portion of the discharge through a side arm. To test these variants, comprehensive changes on the model geometry are necessary. On a physical scale model, this is very labour- and cost intensive, because parts of the model have to be disassembled and reconstructed. On contrary, in a computer model, it is very easy to change the bed elevation of a whole region by a few mouse clicks. Furthermore it can be observed, that the results of the numerical models are in large scales more reliable than in small scales and details. Therefore, it is much more efficient to test different variants of comprehensive changes on the river bed by the computer models.
Local protection measures include e.g. structures like embankments or groins. This implies that only small changes in the geometry are conducted and their impact is observed. Basing on the fact that relatively small structures are used, also small changes at these structures have an essential influence on the flow and the sediment transport. At this point the physical model shows all its advantages. It is very easy and intuitive to put a small structure into the flow and

watch the influence in real-time. In this manner, it is also possible to correct and optimise the position of the structure. This capability can be provided only by a physical model, because a numerical model cannot afford real-time results and such an intuitive user-interface as a real fluid.

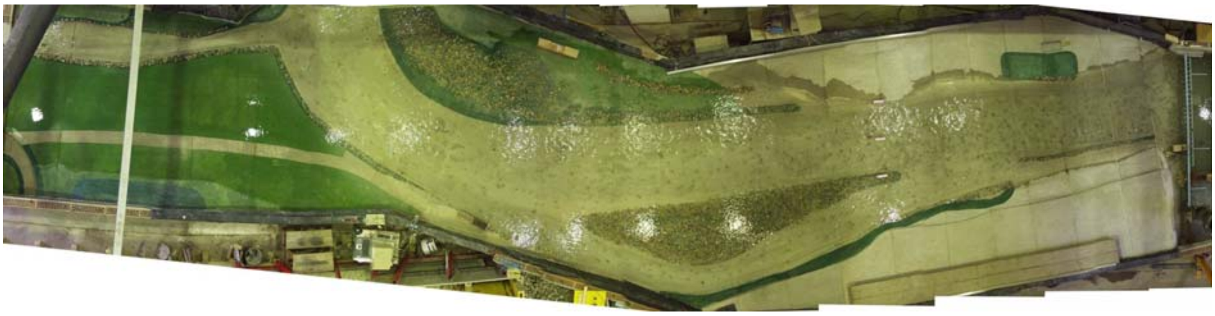

Fig. 4 Example of a river engineering project – physical scale model

For this project both, a physical and a computer model were used complementary. Hence a tight connection was established. In the first step, a computer model was used to find adequate limits for the testing area. After construction, calibration and validation of the physical scale model, the measuring results were used additionally to the field measurements to improve the quality of the computer model. Consequently, the computer model was used to test different variants of comprehensive changes on the river bed. Useful variants which reduce the flood water level were transferred and reviewed on the physical model. This procedure provides more reliability of the results and detailed measurements can be conducted. Furthermore local protection measures like embankments and groins were tested, their impact directly observed and interpreted, and their position and geometry optimised. By this work-flow, the most promising solution for the given problem can be found, and the involved physical processes can be understood.

3 RESULTS

3.1. Basic research – visualisation of flow processes

Basing on the input parameters and the boundary conditions, the essential output variables as flow velocity and aggradations are recorded. With this measured data a computer model can be calibrated and validated. Thus, it was possible to set up a computer model which reproduces the results of the physical model. Since the computer model was basing on simplified input parameters (e.g. single grain size), there are some discrepancies. In the Figure 5, the developing aggradations in the reservoir resulting on the physical and on the numerical model, are presented. The computer model produces a perfect symmetric geometry of the aggradation, which does not correlate with the physical model. In the physical model minimal uncertainties of the boundary conditions have an essential impact on the shape of the delta. Therefore, on the physical model, always a random shape is generated.

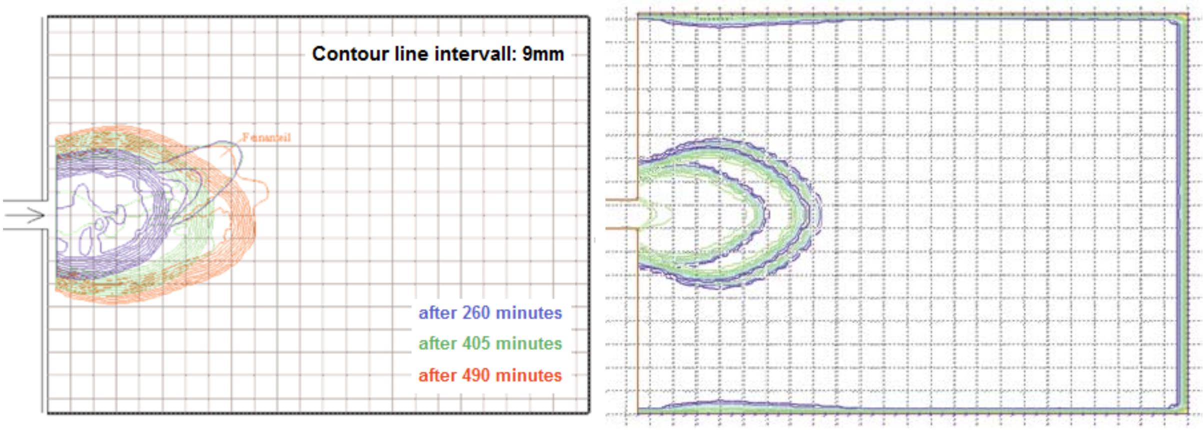

Fig. 5 Aggradation of the reservoir – on the left: physical model; on the right: computer model

Useful variables to describe the shape of a delta are the length and the width of the aggradation. Thus, these variables were used to verify the results of the computer model. In the Figure 6 it can be seen, that the discrepancy of the dimension between physical and computer model are most of the time less than 10%.

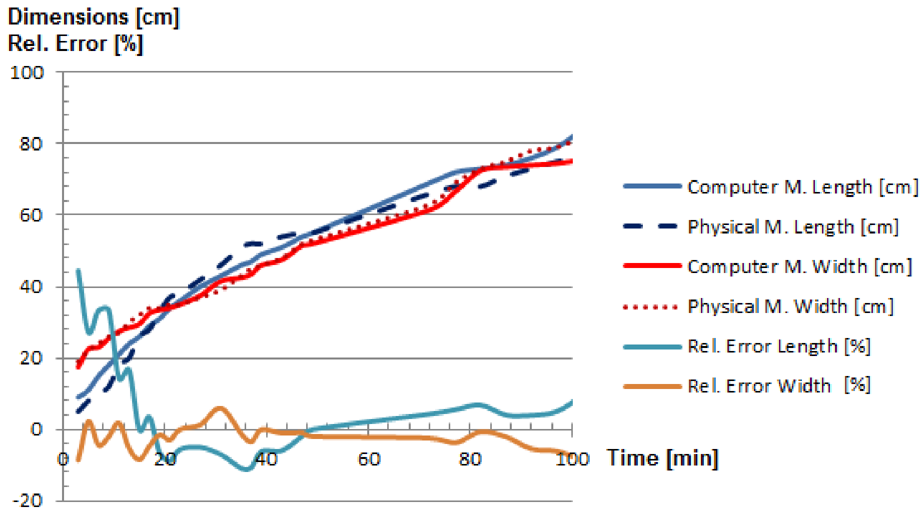

Fig. 6 Relative error in the computer model compared to the results on the physical model

Basing on this result we conclude that in the computer model similar flow processes as in the physical model occur. Therefore, the computer model can be used to illustrate the flow conditions in the reservoir, as it is showed in the Figure 7. Thus it is possible to study the assumed flow processes in the physical model without the need to measure them.

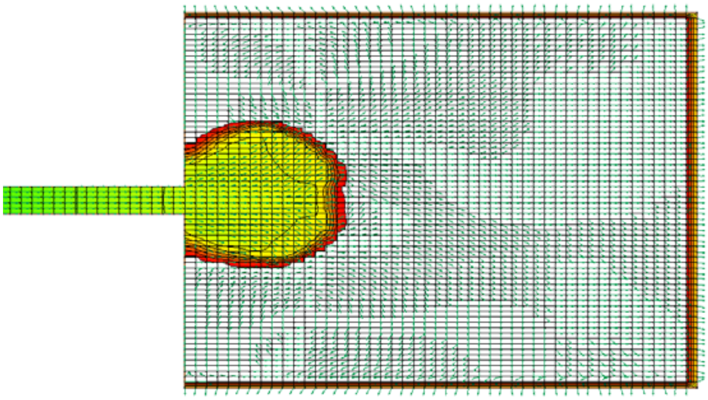

Fig. 7 Aggradations and flow conditions in the reservoir

3.2 River Engineering – alternately use of different models for variant studies

The final results of this project are different alternatives to improve the flood protection in the tested area. Bellow, the techniques to find these alternatives are presented. In the Figure 7 the solution variants were presented. C-1, C-3, and C-4 are comprehensive river-bed lowering areas. In the C-2 area the river bed has been widened. All this comprehensive changes in the geometry of the river bed would be costly in terms of labour, if implemented in the physical model. Thus, it is very useful to check such measures firstly in a computer model in order to validate whether they do have any useful effect. Furthermore, it is also possible to optimise the geometry in the computer model by testing several variants. Nevertheless, it would take one or two weeks to implement such substantial changes into the physical model, so it is much more efficient to test several variants of river bed geometries in the computer model and to find the optimal solution.

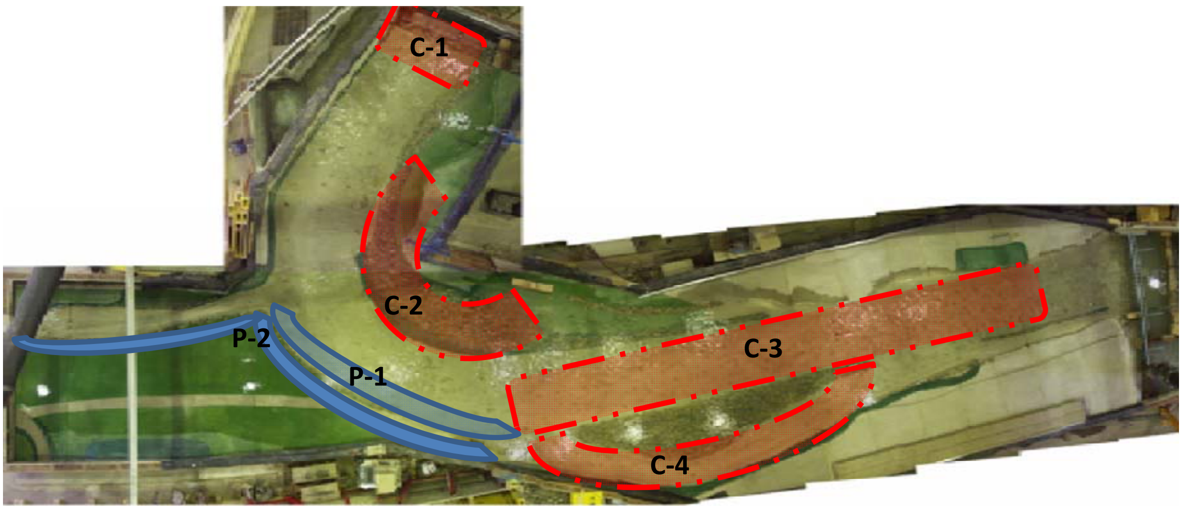

Fig. 7 Aggradations and flow conditions in the reservoir

Beside the changes in the geometry of the river bed, also smaller, less extensive measures have been tested in the project. So, groins have been tested in the P-1 area and embankment constructions in the area P-2 (see the Figure 7). By testing these small scale measures, the physical model can demonstrate its advantages. However, to implement groins or embankments in a computer model, it is necessary to change the mesh, what requires an additional, quite extensive effort. In a physical model, it is sufficient to put small obstacles into the water to simulate groins or embankments. By this method it is easy to rearrange the positions of the groins, or to change the shape of the embankment. Furthermore, if an obstacle gets into the water, the flow immediately responds to the new situation. So, it is possible to investigate the flow and also turbulences in real time, in contrary to the computer model, where it is necessary to wait for the results.

4 DISCUSSION AND CONCLUSION

Both, physical and numerical models have their pros and cons. Physical models offer the opportunity of direct interaction and real-time feedback. So, is it possible to check the influence of small obstacles to the flow directly. Another advantage of physical models is that they provide reliable results, because there is always the whole spectrum of flow conditions like turbulences and three dimensional flow processes implemented. The main disadvantage of physical model is the amount of work. But also limitations of the model, like restricted space and discharge in the laboratories, play a decisive role in the decision process for the selection of the model type. Last but not least, also the limitations of the data measurements should be kept in mind.

Numerical models offer advantages in less labour cost and feasibility of extensive changes in the geometry of the model. Furthermore, also the coverage of the results is comprehensive, because in each calculation step the results of all variables are available and can be stored and easily displayed. However, the challenge of numerical modelling is the prediction of the occurring flow processes. The application of a numerical model always includes a theoretical analysis where all the appearing flow conditions have to be identified in order to implement them in the computer model. Out of this follows the biggest problem of computer models, the reliability. Because, if not all essential processes are taken into account and calculated, the results are incomplete. And these errors can only be found by validation of the results by additional field data.

Complementary modelling offers in these conflict areas a useful alternative. Therefore, each single task in a particular case is conducted in in the more appropriate model. So e.g. small obstacles to guide the flow like groins are tested in the physical model, and different building variants with different river bed geometries are checked in the computer model. Furthermore, with complementary modelling one model can generate additional results by the implementation and use of insights of the other model. This means that e.g. a computer model calculates additional flow velocities basing on the measurements on the physical model, or that the flood water protection measure, found on the computer model, is rebuild and reviewed on the physical model in order to achieve more reliable, because double checked, results.

Besides the pro and cons of physical and computer models, complementary modelling offers a method to combine the advantages of all models and leads to a better work flow and to more reliable results. Thus it is recommended, especially if a physical model is constructed, to use complementary modelling were ever it is possible.

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MODELLING OF TRANSIENT FLOW IN STORM SEWERS

M. Szydłowski[1]

Abstract

The paper focuses on the assessment of second-order explicit numerical scheme for unsteady flows in sewers. In order to simulate the pressurized flow the 'Preissmann slot' concept is implemented. For simulation of the transcritical flow the original and improved McCormack scheme is used. The calculated results are compared with numerical solutions and laboratory measurements published in the technical literature. Moreover, the aim of this paper is to present the comparison between calculated and measured at the hydraulic laboratory of Gdańsk University of Technology (GUT) pressure values which were observed during experiment for water flow with pipe pressurization. The analysis proves that proposed numerical approach to flow simulation is a sufficiently accurate and reliable technique for prediction of basic storm sewers flow parameters.

Keywords

modelling, simulation, storm sewers, transient flow

1. INTRODUCTION

The process of water flow in the storm sewers systems is often affected with some specific hydraulic effects occurring in pipelines during high precipitation periods. If an extreme rainfall is observed the surface run-off from the urban basin can be relatively intensive exceeding the sewers pipes capacity. Such situation may cause the pipes start to be fully filled with water resulting with the transition from free surface to pressurized flow. After the storm event the vice versa effect can be observed. Such transitions are also possible in sewers when the discharge is controlled by some control devices, like gates for example or as a resu1t pipe blockade. Moreover, the rapidly varied flow with some hydraulic local effects like jump or bores can appear in water conduits during extreme storms.

The appropriate numerical methods have to be used to solve the storm sewers flow problem. In general, the water flow in storm sewers can be classified as a free surface open channel flow because the pipes transport water in partially full pipes and one dimensional models of

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flow are usually used to describe the flow. However, as it was mentioned before, the hydraulic transitions and transcritical flow in sewers pipes can be sometimes observed. Many mathematical models of flow adopting different techniques for transients implementation can be found in the technical literature. In general, the methods can be grouped into the three main approaches known as rigid column, full dynamic and Preissmann slot models [1]. The last one is very popular in storm sewers modelling but it can be used only when the flow is expected to be pressurized but without subatmospheric pressures. In the Preissmann concept [2] the hypothetical slot is placed at the top of the pipe. The slot makes the flow free surface even when the pressure line in a pipe cross section is located above the top of the pipe. This method is very attractive for the numerical calculation because there is no need to track the pressurization front. The Preissmann slot method can be found in many hydrodynamic models of storm sewers flow [3,4] and it was also implemented and described by Szydłowski and Machalińska-Murawska [5]. In that work the standard and improved McCormack schemes were used for simulation of the transcritical flow and compared with literature data. Now, the comparison between own numerical calculations and measurements of transient flow in storm sewers single pipe is presented in this paper.

2. MATHEMATICAL MODEL

Numerical simulation of flow in storm sewers involves solution of unsteady water flow equations. This set of equations can be derived from continuity and Navier–Stokes equations. Particularities of storm sewers flows allow for some simplifications for the continuity and momentum equations. The most important is a fact that the storm waters are conveyed in the system of channels and tunnels, what makes the flow character one-dimensional. The conduits are normally prismatic and the system is often composed of circular pipes. Moreover, the inflow into the system is possible only through the dropshafts and manholes, so between these elements there in no lateral inflow. Therefore, the free surface flow in the sewers can be described using the Saint–Venant equations. This mathematical model is a system of partial differential equations and the numerical method must be applied to solve the model equations. In this paper McCormack scheme of finite differences method (FDM) was chosen to approximate the model equations.

2.1. Model equations

Because during high surface water runoff from urban areas the various local phenomena like a hydraulic jumps can occur in sewers, the special form of flow model equations have to be used. To correctly reproduce the steep water surface fronts the conservative form of flow equations should be used in modelling [2]. For the case of prismatic channel such as a pipe with no lateral inflow or outflow the Saint–Venant system, written in the conservative form, can be presented as

$$\frac{\partial \mathbf{U}}{\partial t} + \frac{\partial \mathbf{F}}{\partial x} = \mathbf{S}, \tag{1}$$

where the vectors **U**, **F** and **S** are given as:

$$\mathbf{U} = \begin{pmatrix} A \\ Q \end{pmatrix}, \quad \mathbf{F} = \begin{pmatrix} Q \\ Q^2 / A + I \end{pmatrix}, \quad \mathbf{S} = \begin{pmatrix} 0 \\ gA\left(S_o - S_f\right) \end{pmatrix}, \tag{2}$$

where x represents distance along the sewers conduit, t represents time, A is cross-sectional wetted area, Q is flow discharge and g is gravitational acceleration and $I=pA/\rho$, where p is a pressure at the centroid of A, and ρ is a constant fluid density. For the hydrostatic pressure distribution the term I can be defined as $I=gAh_c$, where h_c is a distance between the free surface and centroid of flow cross-sectional area. The S_0 and S_f are the bottom and friction slopes, respectively. The friction slope can be defined by Manning formula

$$S_f = \frac{n^2 Q|Q|}{A^2 R_h^{4/3}}, \tag{3}$$

where n denotes the Manning friction coefficient, and $R_h=A/P$ is hydraulic radius and P is wetted perimeter. Under free-surface flow conditions in a circular pipe of inner diameter d, each geometrically related variable can be described by the wetted angle of the pipe θ (Fig. 1) [6]. The celerity c is also related to the wetted angle θ because of the form of A and the top width of free surface T.

$$c = \sqrt{\frac{gA}{T}} \tag{4}$$

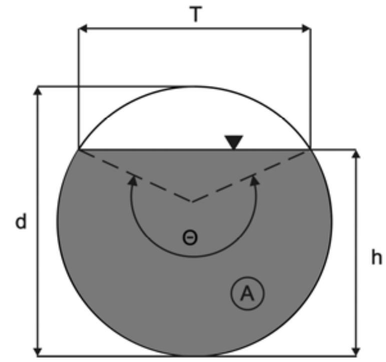

Fig. 1 Definition of variables in circular cross section

In order to simulate the water flow under pressurized conditions the 'Preissmann slot' concept is used in this work. The idea of hypothetical slot at the top of the pipe was presented by Cunge *et al.* [2] and it was implemented for numerical modelling of storm sewers by many researchers [3,4] and it was also investigated by Szydłowski and Machalińska-Murawska [5]. The slot assures that after filling the pipe the flow can be treated as the open channel flow in spite that piezometric pressure exceeds the pipe diameter. In such situation the speed of gravity wave in the open channel with slot can be expressed according to the equation (4) as

$$c = \sqrt{\frac{gA_p}{T_s}} \tag{5}$$

where A_p is the cross-sectional area of the pipe and T_s is the width of the slot. Moreover, the speed c for the pressurized flow must be the same as sound speed a in a closed pipe.
The 'Preissmann slot' idea makes the simulation of transient flow possible without need to separately track the pressurization front but such solution has some disadvantages. The most important is inability to simulate full pipe subatmospheric flow, like in some special segments

of sewers of siphon type. Other problems are inaccurate mass conservation due to additional slot in the pipe and simulation instability, when water surface exceeds the pipe diameter and enters to the slot.

2.2. Numerical solution

In this paper an improved McCormack scheme of finite differences method (FDM) was chosen to approximate the model equations. The improvement of the scheme is based on the theory of total variation diminishing (TVD) schemes that are capable of capturing sharp discontinuities without generating the spurious oscillations. The scheme is second-order accuracy both in time and space in non-critical sections, but it switches the accuracy to the first-order at extreme points where the oscillations occur. This technique was previously presented by Garcia-Navarro *et al.* [7] and it was originally proposed for solving one-dimensional open-channel flow equations.

The TVD improved McCormack scheme is an extension of the standard McCormack method [8] and it includes an additional computational term in updating step of original predictor corrector procedure, what can be finally presented as

$$\mathbf{U}_i^{n+1} = \frac{1}{2}\left(\mathbf{U}_i^{p} + \mathbf{U}_i^{c}\right) + \frac{1}{2}\frac{\Delta t}{\Delta x}\left(\mathbf{R}_{i+1/2}\mathbf{\Phi}_{i+1/2} - \mathbf{R}_{i-1/2}\mathbf{\Phi}_{i-1/2}\right) \tag{6}$$

where

$$\mathbf{U}_i^{p} = \mathbf{U}_i^{n} - \frac{\Delta t}{\Delta x}\left(\mathbf{F}_{i+1}^{n} - \mathbf{F}_i^{n}\right) + \Delta t \mathbf{S}_i^{n} \tag{7}$$

$$\mathbf{U}_i^{c} = \mathbf{U}_i^{n} - \frac{\Delta t}{\Delta x}\left(\mathbf{F}_i^{p} - \mathbf{F}_{i-1}^{p}\right) + \Delta t \mathbf{S}_i^{p} \tag{8}$$

where the superscripts p and c refer to the predictor and corrector step, respectively, and n is the time level. The second term in equation (6), which has to be estimated at an intermediate states between grid points i–1, i and i+1 equips the scheme with TVD properties adding a numerical dissipation to the original scheme. Due to this modification, the scheme retains second-order accuracy in space and time for continuous regions and it is able to limit the solution oscillations near the extremes by reducing the accuracy to first-order in these sections.

The TVD improvement requires the quasi linear form of the Saint–Venant equations (1,2) to calculate the additional term in formula (6). The original problem (1) can be transformed to the following form

$$\frac{\partial \mathbf{U}}{\partial t} + \mathbf{A}\frac{\partial \mathbf{U}}{\partial x} = \mathbf{S} \tag{9}$$

where **U** is the same as in equation (1) and the jacobian matrix **A** of **F** with respect to **U** can be written as

$$\mathbf{A} = \begin{bmatrix} 0 & 1 \\ c^2 - \frac{Q}{A^2}^{2} & 2\frac{Q}{A} \end{bmatrix} \tag{10}$$

where c is a celerity defined by Eq. (4). The jacobian matrix **A** is diagonalizable, so the following equation must be satisfied

$$\mathbf{A} = \mathbf{R}\,\boldsymbol{\Lambda}\,\mathbf{L} \tag{11}$$

where $\boldsymbol{\Lambda}$ is a diagonal matrix containing the eigenvalues of matrix **A,** whereas **R** and **L** contain associated right and left eigenvectors. The eigenvalues λ of matrix **A** can be evaluated by solution of the characteristic equation

$$\left|\mathbf{A} - \lambda\mathbf{I}\right| = 0 \tag{12}$$

where **I** is the identity matrix. Considering jacobian matrix (10), the roots of (12) equal

$$\lambda_1 = u - c\,, \quad \lambda_2 = u + c \tag{13a,b}$$

where $u=Q/A$. The matrix $\boldsymbol{\Lambda}$ and the corresponding right, used in updating step of TVD McCormack scheme (6), and left eigenvector matrices for matrix **A** are defined as

$$\boldsymbol{\Lambda} = \begin{bmatrix} \lambda_1 & 0 \\ 0 & \lambda_2 \end{bmatrix},\ \mathbf{R} = \begin{bmatrix} 1 & 1 \\ \lambda_1 & \lambda_2 \end{bmatrix},\ \mathbf{L} = \frac{1}{2c}\begin{bmatrix} -\lambda_2 & 1 \\ \lambda_1 & -1 \end{bmatrix} \tag{14a,b,c}$$

The two components of vector $\mathbf{\Phi}_{i+1}$ in Eq. (6), which are evaluated at the intermediate state between grid points i and i+1, are defined as

$$\Phi^k_{i+1/2} = \psi\left(\lambda^k_{i+1/2}\right)\left(1 - \frac{\Delta t}{\Delta x}\left|\lambda^k_{i+1/2}\right|\right)\left(1 - \varphi\left(r^k_{i+1/2}\right)\right)\alpha^k_{i+1/2} \quad (k = 1{,}2) \tag{15}$$

The function Ψ is an entropy correction to the eigenvalues preventing the appearance of unphysical flow discontinuities, those in which energy increases across the shock. In the simplest form it can be written as [7]

$$\Psi(\lambda) = \begin{cases} |\lambda| & \text{if } |\lambda| \geq \varepsilon \\ \varepsilon & \text{if } |\lambda| < \varepsilon \end{cases} \tag{16}$$

where ε is a small positive number, which value must be determined for each individual problem. Some propositions of formulas for the ε evaluation and other forms of the entropy correction have been proposed by Harten and Hyman [9].
The characteristic variable α in formula (15) is defined as

$$\alpha_{j+1/2} = \frac{1}{2c_{i+1/2}}\begin{bmatrix} -\lambda_2 & 1 \\ \lambda_1 & -1 \end{bmatrix}_{i+1/2}\begin{bmatrix} A_{i+1} - A_i \\ Q_{i+1} - Q_i \end{bmatrix}. \tag{17}$$

In order to calculate the mean values of flow parameters in (17), which must be determined at the intermediate point i+1/2, the averaging procedure proposed by Roe [10] can be applied. Following Garcia-Navarro *et al.* [7], the discrete approximations to the local water velocity and celerity can be presented as

$$u_{i+1/2} = \frac{Q_{i+1}/\sqrt{A_{i+1}} + Q_i/\sqrt{A_i}}{\sqrt{A_{i+1}} + \sqrt{A_i}}, \qquad c_{i+1/2} = \frac{c_i + c_{i+1}}{2}\,. \tag{18}$$

For obtaining non-oscillatory solutions in regions where some flow discontinuities like hydraulic jumps or bores exist, the limiter parameter has to be incorporated into solution procedure. In equation (15) the function φ is a limiter parameter and it is responsible for adding artificial dissipation to the numerical solution in a regions of steep gradients. The numerical dissipation makes the solution monotone at extreme points. In the continuous regions of smooth variation little or no dissipation is added. Many forms of the limiting function can be found in the literature. Their review in a relation to the water flow problem has been presented by Toro [11]. Following Tseng [12] the minmod limiter was used in this paper to simulate transient flow in the storm sewers. This function can be written as

$$\varphi\left(r_{i+1/2}^{k}\right)=\begin{cases}\min\left(\left|r_{i+1/2}^{k}\right|,1\right) & \text{if } r_{i+1/2}^{k}>0\\ 0 & \text{if } r_{i+1/2}^{k}\leq 0\end{cases} \tag{19}$$

where r is a ratio of characteristic variables estimated as follows

$$r_{i+1/2}^{k}=\frac{\alpha_{i+1/2-s}^{k}}{\alpha_{i+1/2}^{k}},\quad s=\operatorname{sign}\left(\alpha_{i+1/2}^{k}\right). \tag{20}$$

In order to perform numerical simulation of the Saint–Venant equations (1,2) it is necessary to specify additional conditions. According to the theory of solving partial differential equations they include the initial condition and boundary conditions [2]. The type and number of boundary conditions depend on characteristics theory and it is associated with variability of flow parameters at inflow and outflow cross–sections of the channel. The direction and regime of the water flow in the storm sewers are variable what determines the number and type of boundary conditions needed at inlet or outlet sections of the conduits. The detailed boundary conditions analysis is not substantial for this study, so it is omitted here but the proposition of the organization of possible boundary conditions for modelling of storm sewers flow can be found in the work of Capart *et al.* [3] for example.

3. LABORATORY AND NUMERICAL TESTS

In order to assess the quality of numerical solution, results of simulations can be compared to the calculations and measurements available in the literature. Such kind of the verification was presented by Szydłowski and Machalińska-Murawska [5]. It was concluded that an improved McCormack scheme together with idea of the Preissmann slot allows to model the transient and transcritical flow in pipes of storm sewers describing main flow features, such as positive and negative open channel pressurized flow interfaces quite good. However, due to no detailed information about experiments conditions and no exact laboratory data available for the authors of the paper, the model was not finally verified. In order to asses the numerical solution better than only on qualitative level, own measurements should be used for comparison. Now the laboratory experiments have been carried out at the hydraulic laboratory of Gdańsk University of Technology and can be used for comparison.

Here in this paper the numerical results obtained using McCormack scheme for two test problems are presented and analysed. Both examples consider the flow transients in circular pipes resulting from sudden manoeuvres of flow control devices during laboratory experiments. First test case can be found in the literature [3] and it was used for the preliminary test [5]. The second test presents a laboratory experiment carried out at hydraulic laboratory at GUT.

3.1. Test no. 1

The first test was described by Capart *et al.* [3]. It consisted in the pressurisation of steep slope circular pipe. The experimental set-up featured a 12.74 m long pipe with a 0.145 m inner diameter. The Manning roughness coefficient was equal to 0.009 s/m$^{1/3}$. The pipe consisted of three segments of different bottom slopes 0.01954 (0–3.48 m), 0.01704 (3.48–9.23 m) and 0.01255 (9.23–12.74 m), respectively and it connected two tanks. The water level in the downstream tank was kept below the pipe outlet. The upstream tank supplied the pipe with water. The experiment started from steady state. At the upstream, a constant discharge of 0.0042 m^3/s was kept. Due to the relatively steep slope of the pipe, the flow at the inlet was supercritical while the flow regime at the downstream end depended on the water level varying during experiment. The hydraulic jump was generated by a sudden gate closure at the downstream end of the pipe in the first phase of the experiment. Then the jump was going upstream making the pipe pressurization. When the jump was near the upstream end, the downstream gate was open, leading to a sudden decrease of the water surface. In this second phase a fast transient, in the form of a negative wave, returned the flow to its initial conditions.

For the numerical simulation of test no.1 the pipe was discretized into 255 nodes with Δx = 0.05 m length of the spatial step. The simulation was carried out with the time step equal to Δt = 0.005 s, ensuring the stability of the solution. The varying bottom line of the pipe was replaced by constant average value of the bottom slope (0.017) in the simulation. The comparison between results obtained by Capart *et al.* [3] and own calculations is presented in Figures from 2 to 5. The graph on Figure 2 shows the variation in time of piezometric head for three measuring points located 3.06 m (C3), 5.50 m (C4) and 7.64 m (C6) from the upstream end of the pipe, which were originally presented by Capart *et al.* [3]. The computed results presented in this picture corresponds to the solution obtained using Pavia Flux Predictor scheme. The own numerical calculations for the same control points are presented in Figure 3. It can be observed that water levels and the speed of the wave front seem to be in a good agreement. However, it can be seen that the results obtained with classic and improved McCormack schemes differ. The standard scheme produces the spurious oscillations near sharp pressure (water level) fronts, while the improved version of the scheme ensures quite smooth solution.

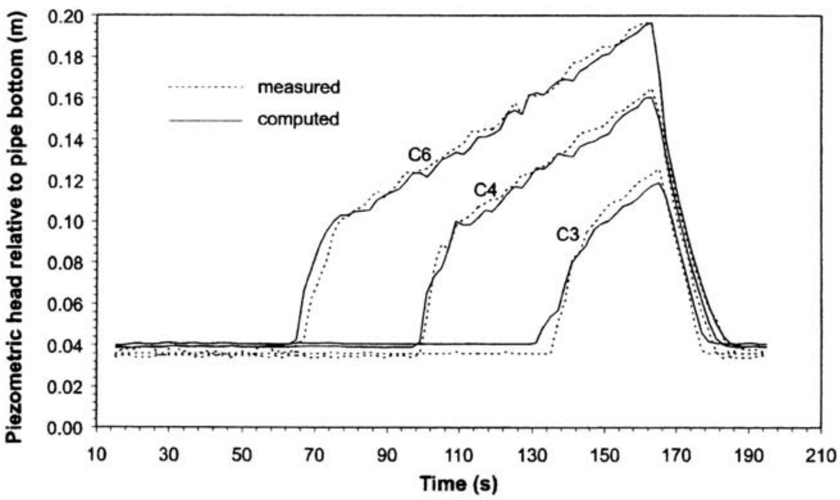

Fig. 2 Test no. 1. Piezometric head measured and calculated at sections C3, C4 and C6 by Capart *et al.* [3]

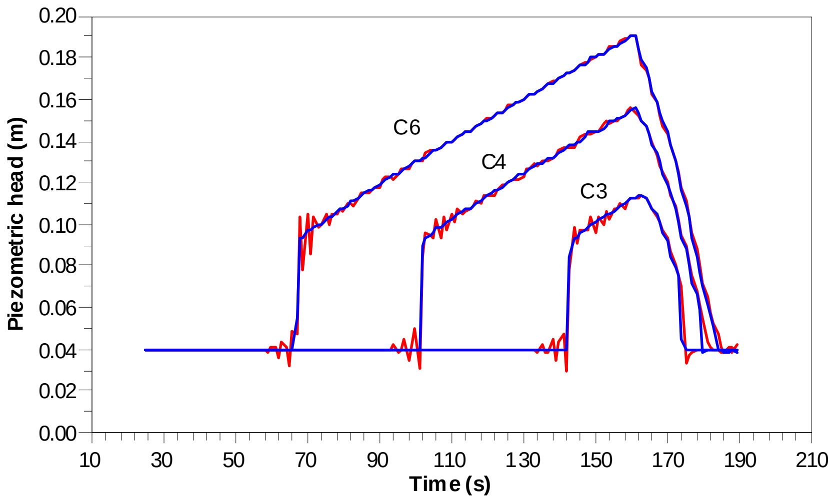

Fig. 3 Test no. 1. Piezometric head calculated at sections C3, C4 and C6 using classic (red line) and improved (blue line) McCormack scheme

A comparison between piezometric profiles computed and observed during the first period of the experiment is presented in Figures 4 and 5. The graphs present the piezometric lines at the same times. In these pictures the formation and progression of hydraulic jump can be seen. Analyzing the piezometric profiles for different time steps it can be seen that at the first moment after jump occurrence the free surface water flow exists along the pipe. Then, together with the rising water level at the outflow section of the pipe, the pressurization process can be observed. It can be seen that the agreement between originally measured and computed results and obtained using McCormack schemes is quite good, at least at the qualitative level. Finally, it can be found that again the solution obtained using the improved McCormack scheme better fits the previously published results than its classical version.

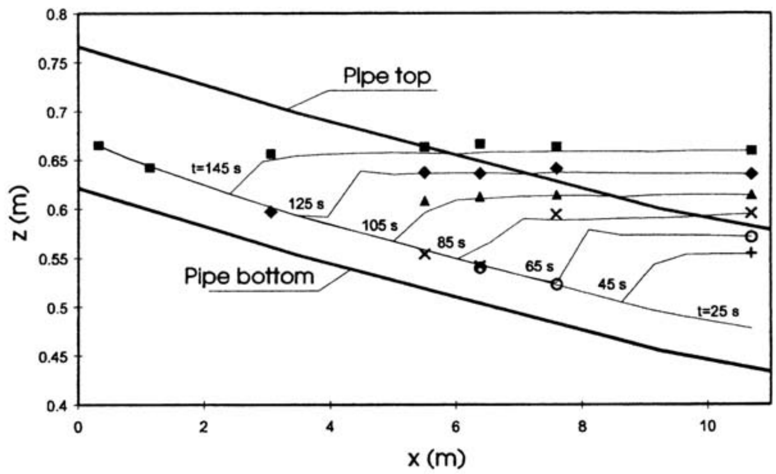

Fig. 4 Test no. 1. Piezometric profiles measured (markers) and calculated (solid line) by Capart *et al.* [3] for the first period of the experiment

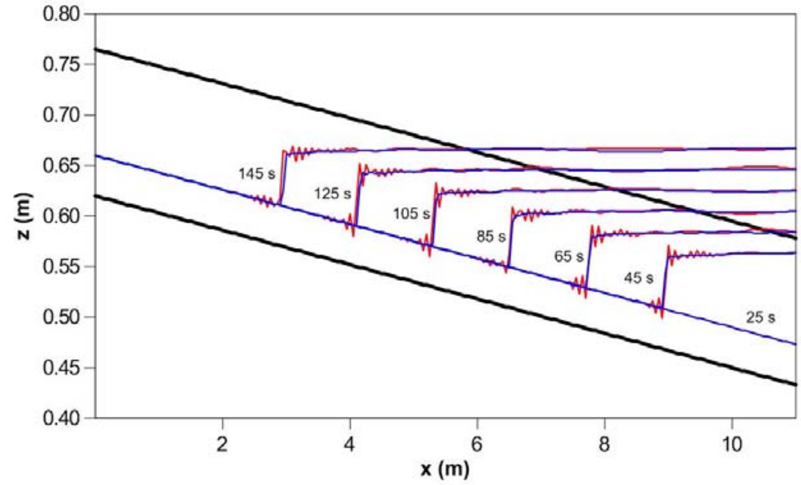

Fig. 5 Test no. 1. Piezometric profiles calculated using classic (red line) and improved (blue line) McCormack scheme for the first period of the experiment

3.2. Test no. 2

The second test is based on the own laboratory measurements. The experimental set-up was built at the hydraulic laboratory of GUT. The tests were carried out in a rectangular open channel with concrete bed and glass walls, where inside the circular pipe was located. The laboratory set-up featured a 10.55 m long pipe with a 0.15 m inner diameter. The Manning roughness coefficient was equal to 0.009 s/m$^{1/3}$. The pipe consisted of two segments of the same bottom slope equal to 0.005 and it joined two tanks. The water level in the downstream tank was initially kept below the pipe outlet. The upstream tank supplied the pipe with water. The experiment started from steady state. At the inflow section, a constant discharge of was kept. The scheme and the pictures of the laboratory set-up are presented in Figure 6.

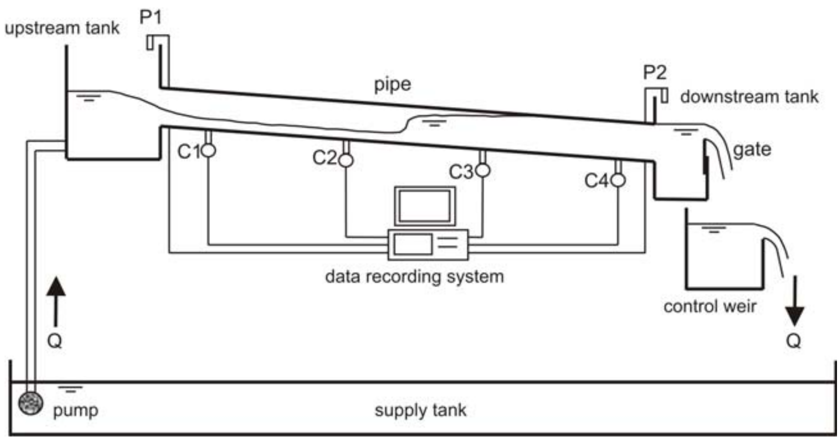

Fig. 6 Test no. 2. Scheme of the laboratory set-up

The pressurization of pipe flow and the water surface front migrating upstream the pipe were generated by a sudden gate closure at the outflow section of downstream tank. The variation of pressure head in time was measured using hydrostatic probes at two points along the pipe located at 1.0 m (C4) and 6.54 m (C2) from the downstream end of the pipe. Additionally, the water surface levels in both tanks were investigated by ultrasonic sounds (P1, P2). The other

two probes locations (C3 and C1), shown in the Figure 6, were not analyzed in the experiment presented in this paper.
The steady flow in laboratory hydraulic system was measured with the control weir at the outlet section of the laboratory channel. The constant initial inflow rate was equal to 0.0055 m^3/s. The water depth along the circular conduit was equal to 0.062 m. The unsteady flow and pipe pressurization was caused by sudden maneuver at the outflow section. In the first phase of the experiment, after the sudden gate closure, located in the downstream tank, the hydraulic jump was generated inside the pipe. Then the jump was going upstream making the pipe cross-section fully filled with water. Finally, the pressurization process made the pipe working as a submerged culvert. During the last period of the experiment the gate at the downstream tank was open in short time, leading to a sudden decrease of the water surface. In this third phase a fast transient, in the form of a negative wave, returned the flow to its initial conditions.
In order to simulate the one dimensional water flow in the single pipe using FDM improved McCormack scheme, the conduit had to be transformed into numerical mesh. The 10.55 m long pipe was discretized into 212 computational nodes with Δx=0.05 m length of the spatial step. The simulation was carried out with the time step equal to Δt=0.002 s, ensuring the stability of the solution. The bottom line of the pipe was represented by constant average value of the bottom slope (0.005) in the simulation. The numerical simulation of transient flow was carried out for the same initial condition as it was described for the laboratory experiment. The hydraulic jump, starting the pressurization process, was generated at the pipe outflow section by sudden closure of the gate in the downstream tank. The measured variation of the water level in this tank, presented in the Figure 7, was used as downstream boundary condition for the flow simulation. It can be observed that pressurization of the outflow section of the pipe began at time t=14.5 s and finished at t=70s.

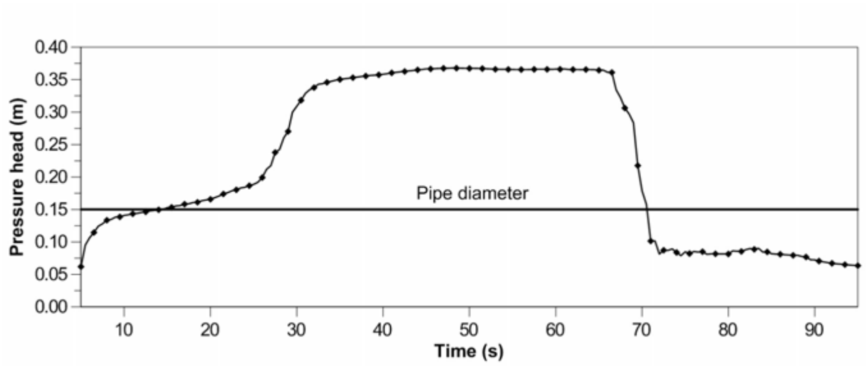

Fig. 7 Test no. 2. Downstream boundary condition - pressure measured at control point P2

The comparison between measurements and results of numerical simulation are presented in Figures from 8 to 14. The graphs in Figures 8, 9 and 10 show the variations in time of pressure head for three measuring points C4, C2 and P1, respectively. It can be observed that just after gate closure the hydraulic jump was formed inside the conduit what can be seen as a sudden rise of the water depth in the pipe. The hydraulic jump migrated upstream the pipe in free surface flow conditions, reaching the control point C4 and C2 at t=9 s and t=16 s. After t=25.5 s the hydraulic jump reached the upstream tank (point P1), starting the filling it with water (Fig. 10). Due to continuous rising of water level in the downstream tank the pressurisation of the pipe began after hydraulic jump occurrence. This effect can be observed at point C4 and C2 at time t=17 s and t=25 s, respectively, when the pressure head exceeds the pipe diameter. Both described hydraulic phenomena, related to rapidly varied and transient water flow, were reproduced in the numerical simulation properly. Some discrepancy between measurements and computations is observed at the beginning of the pressure pipe flow period what is the result of numerical solution oscillation due to Preissmann slot technique application.

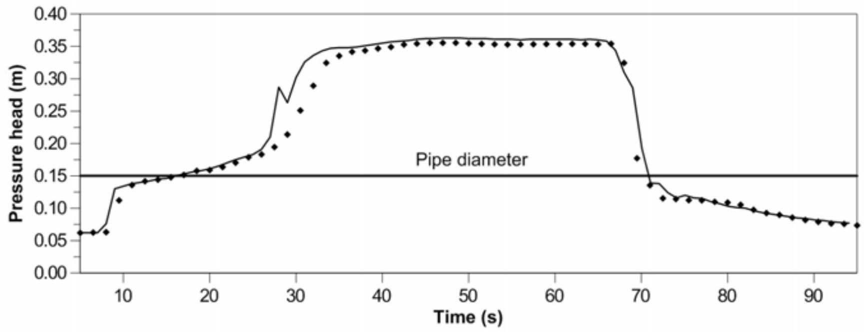

Fig. 8 Test no. 2. Measured (markers) and computed (line) pressure at control point C4

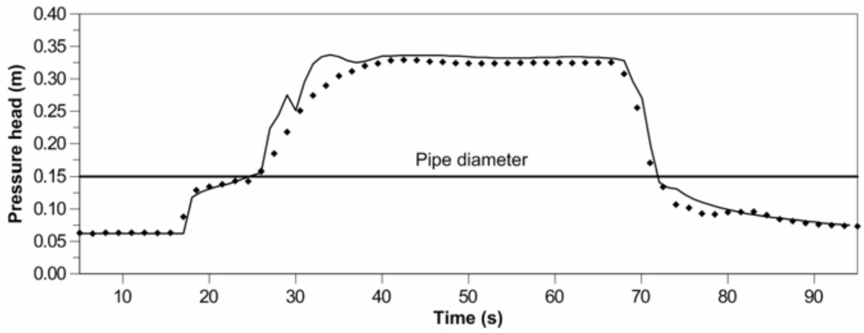

Fig. 9 Test no. 2. Measured (markers) and computed (line) pressure at control point C2

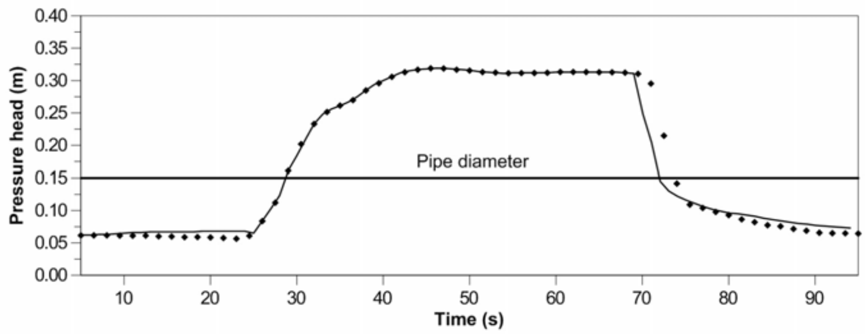

Fig. 10 Test no. 2. Measured (markers) and computed (line) pressure at control point P1

After full pressurization of the pipe the water flow became steady. During this period the piezometric pressure along the pipe is almost the same due to small velocity of the flow and low value of the pipe friction coefficient. The flow changed again into the unsteady at t=69 s due to opening of the gate located in the downstream tank. The sudden decrease of the pressure and transition from the pressure into the free surface flow can be also observed for control points C4, C2 and P1 (Figs. 8, 9, 10). First it can be seen at C4 section where pressurisation finished at t=70 s. Next the free surface occurred at point C2 at t=72 s and finally at section P1 at t=74 s. The worst quality of the numerical simulation was obtained for the point P1, where the upstream tank was located during laboratory experiment, which was not precisely represented by the inflow boundary condition imposed for the calculation.

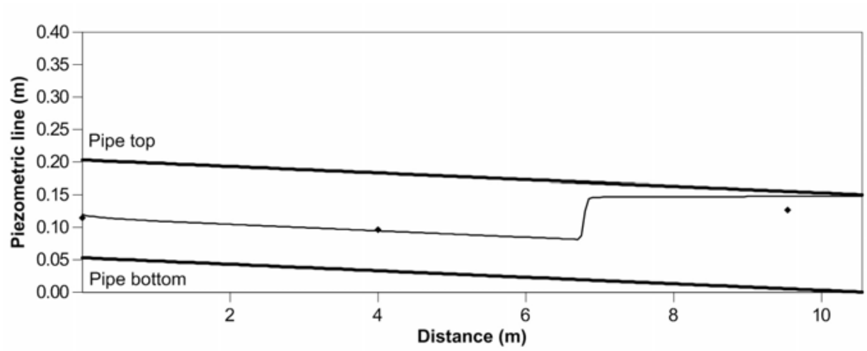

Fig. 11 Test no. 2. Measured (markers) and computed (line) pressure profile for t=10s

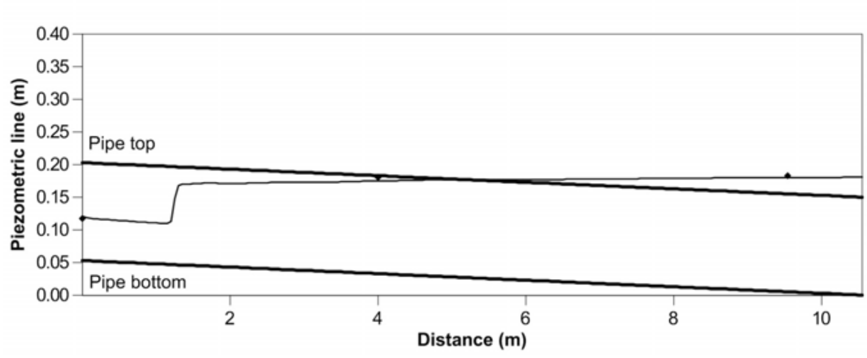

Fig. 12 Test no. 2. Measured (markers) and computed (line) pressure profile for t=20s

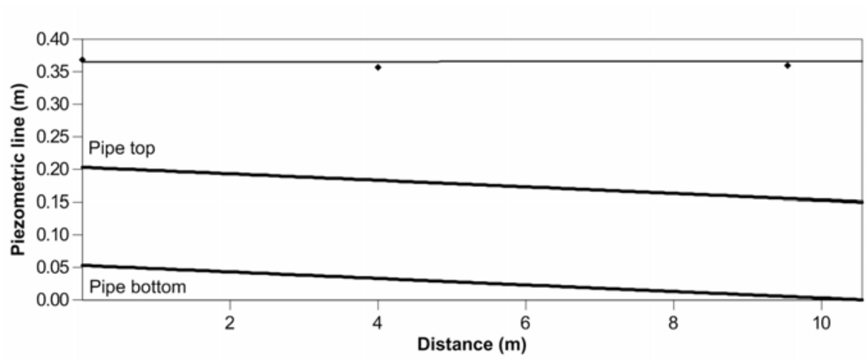

Fig. 13 Test no. 2. Measured (markers) and computed (line) pressure profile for t=50s

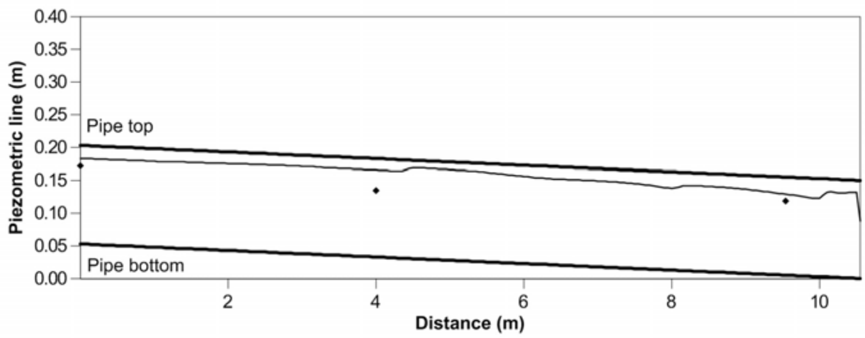

Fig. 14 Test no. 2. Measured (markers) and computed (line) pressure profile for t=75s

The processes of hydraulic jump migration and pressurization of the pipe can also be investigated in Figures 11, 12, 13 and 14. There are comparisons between computed and measured pressure profiles presented for some experiment moments. In Figures 11 and 12 the hydraulic jump moving upstream can be seen at t=10s and t=20s. It can be also observed that after 20 seconds of simulation upper part of the pipe is partially filled with water and lower part is pressurized. In the next graph (Fig. 13) the steady flow state in submerged condition observed 50 seconds after gate closure is presented. Finally, the pressure profile in a form of free water surface for t=75s is presented in Fig. 14. For all cases presented in these figures, the quite good agreement between calculations and measurements was obtained, what proves the proper solution of unsteady water flow equations for transient flow by improved McCormack scheme. The results disagreement can be observed only after sudden release of water (Fig. 14) what can be explained by poor representation of outflow boundary condition during this experiment period.

4. CONCLUSION

The main aim of this work was to test a numerical model for simulating transient and transcritical flow in storm sewers systems using the 'Preissmann slot' approach for the treatment of pressurized flows. The numerical solution of the Saint–Venant equations for one-dimensional flow based on the improved McCormack scheme was investigated in the paper. The results of numerical simulations of the flow in the single pipe of storm sewer were presented and analyzed. They were compared to the laboratory measurements carried out at GUT hydraulic laboratory. The own experiment presented in this paper allows to see the numerical simulations satisfying and numerical scheme effective.

Acknowledgements
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COARSE SCREENINGS WITH SELF - CLEANING ABILITY ON INTAKE STRUCTURES OF SMALL HYDROPOWER PLANTS

Ing. Miroslav Tvrdoň[1], Ing. Ján Rumann, PhD.[2]

Abstract

An important part of any small hydropower plant is the intake structure. Its main function is to provide sufficient discharge of water for the hydropower plant and to prevent the large floating objects carried by the stream to get to the hydropower plant. In many cases, this structure requires a weir construction across the riverbed which is often environmentally unacceptable. In cases where such a construction is not possible, another kind of the intake structure has to be used, for example a bank water intake. An important part of these intakes is the coarse screening which prevents floating objects to endanger the operation of the hydropower plant. These floating objects are captured on screenings and afterwards they should be cleaned (removed from the screening). The cleaning is usually manual. The article deals with an unconventional design of these coarse screenings, which due to its design, orientation and shape have a self-cleaning ability.

Keywords

water intake, coarse screenings, intake structure, small hydropower plant

1 INTRODUCTION

An intake structure which takes water from the river to the power plant is a part of every small hydropower plant. The intake structure should keep floating objects in the water from getting to turbines, therefore the intake structure is equipped with coarse screenings.

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2 SCREENINGS

Screenings are an essential part of every intake structure build as a part of hydraulic or hydropower engineering structures. For hydropower plants, their purpose is to prevent that mechanical impurities and floating objects are dragged along with water into the hydraulic system of the structure, which could endanger or complicate the operation of the hydropower plant. The main purpose of screenings is to prevent blockage or flow deterioration in the hydraulic system of the turbine and thus protect the system from mechanical damage. In addition, a number of additional requirements are put on the screenings and their supporting construction, e.g. the ability to resist the design loads, to allow the operational maintenance and cleaning, not to induce too large hydraulic resistance to the flow, etc.

2.1. Coarse screenings

Coarse screenings are part of the intake structure and their main task is to prevent of large floating objects to get to the hydraulic system of a hydropower plant. This means particularly the floating trees, branches, and ice blocks etc. Coarse screenings are usually accessible from a bridge or from the upper surface of the baffle wall, which is designed to prevent the surface floating impurities penetrate into the conduit hydropower plant.

Coarse screenings are frequently proposed as a metal rack with vertical gaps between rakes of 300–600 mm. To ensure a smooth wrapping at various manipulations with hydropower plant turbines is appropriate to propose screenings of thick-walled tubes with a diameter of 80–150 mm. The wall thickness of pipes of screenings depends on their range and load. Screenings are designed to withstand the full blockage, which means that the rack must withstand the total load of hydrostatic pressure in the cross section.

The flow of water through the coarse screenings induces hydraulic losses. These losses depend on the shape of the rods of the coarse rack (presented by the loss coefficient) and the flow velocity. It is therefore recommended that the water flow velocity v_0 in the cross section of coarse screenings does not exceed 0.6 to 0.7 m.s^{-1}. For hydraulic losses induced on coarse screenings, an equation can be derived:

$$h_{zh} = \beta \left(\frac{s}{b} \right)^{4/3} \frac{v_0^{\,2}}{2g}, \tag{1}$$

where:

- h_{zh} the hydraulic losses to coarse screenings [m],
- β shape coefficient of screenings (for pipe screenings $\beta = 1.79$),
- s outer cross-sectional diameter of the rods [m],
- b distance between the screenings [m],
- v_0 average velocity in the inlet opening of the coarse screenings without considering the screennings area [m.s^{-1}].

Losses determined by equation (1) are valid only for clean screenings. Silting-up of the screenings causes significant increase of the hydraulic losses. Therefore, it is necessary to clean the screenings, which is very tiring work, because it is mostly done manually. In some cases it is necessary to shut down the turbines in the hydropower plant (stop the flow) and so reduce the power by which the floating objects are pressed against the screenings. In some of the larger hydropower plants sensors that automatically report the difference in water level are mounted in the front and behind the screenings. After reaching the set maximum difference equal to the permissible size of losses, the operator is advised to clean the screenings.

In addition to hydraulic losses caused by the coarse screenings, it is necessary to calculate hydraulic losses arising in the entire intake. These losses can be expressed by the equation:

$$h_{zv} = \varsigma_0 \frac{v_0^2}{2g}, \tag{2}$$

where:

- h_{zv} the hydraulic losses in intake [m],
- v_0 average velocity of water in the inlet cross section [$m.s^{-1}$],
- ς_0 intake loss coefficient

Based on these data, the total hydraulic losses h_{zc} can be calculated as follows:

$$h_{zc} = h_{zh} + h_{zv}, \tag{3}$$

It should be noted that in equation (3) totally free, unblocked coarse screenings are considered.

2.2. Possibilities of making the screenings features for SHPP more efficient

Presently there are several types of screenings designs for SHPP. Criteria for the selection of the suitable design depend mostly on the quantity and attributes of the floating objects carried by the water, weather conditions, demands for self-service, design requirements and economic costs.

Cleaning and maintenance of coarse screenings are not yet do without heavy and unhygienic manual labor. The coarse screenings of some SHPP in the critical periods cause a loss to water head up to 0.5 m just because of the unresolved way of their prompt.

In order to make the SHPP more efficient it is desirable to relieve the intakes and screenings from the inefficient mechanical cleaning. To ensure an easy cleaning of the coarse screenings, it is necessary to change their orientation from a vertical position to a horizontal position. One way is to use own hydraulic resistance arising from the gradual silting of the screenings. For example, if we determine the acceptable level of screenings silting (and the corresponding force ratios), the higher forces at higher screening silting levels can be used to force rotation, flip, tilt or move the screenings in order to make them clean and then the cleaned screenings return to the original position.

2.3. Coarse screenings of bottom water intake with self-cleaning ability

The main design characteristic of bottom water intakes is a channel in bottom weir construction, which is covered with fine screenings with a significant slope in the direction of water flow. The water flows in the channel through the screenings and so gets rid of the floating impurities then it flows to sedimentation tank and from there to the pressure conduit of the turbine.

During operation the water flows in the bottom channel up to the design discharge and the floating impurities are captured on the screenings and are gradually accumulated at the end of the screenings. At high discharges (above the design value), the water flow also over the weir (not just in the channel) and it takes the accumulated impurities from the screenings. This provides self-cleaning ability.

To ensure adequate operating characteristics of such bottom water intakes in addition to their correct hydraulic design several recommendations must complied, based on the experience gained from the operation of such constructions:

- intake structure should be designed in a straight section of stream,
- the maximum design flow rate should not exceed 8 $m^3.s^{-1}$,
- optimal slope of the screenings towards the water flow direction can be considered a slope of 20%, the use of screenings made of fiberglass rods anchored only on the upstream side and with free ends on the opposite side can reduce the slope to about 10% or less,
- screenings field are made principally from individual rods,
- cross-sectional shape should be circular, drop-shaped or optionally wedge shaped; appropriate construction materials for the screenings are laminates, reinforced plastics or wood (for smaller intakes), these materials are almost anti-freeze and flexibile and they enhance the self-cleaning ability,
- for operation and repair of screenings is necessary that the individual screenings fields are easily removable,
- the entrance wall of the feeder tray before screenings have been proven installation increased spillway edge (min. 10 cm) for example in the form of increased upstream wall of the crown, in the form of special precast or in the form of a removable low dam beams, since, if the water reaches the screenings and the relatively small height, causing them to vibrations, thereby increasing the self-cleaning effect.

3. CONCLUSION

Coarse screenings are an important part of each intake structure of SHPP. They prevent the floating objects to get to the turbines, and thereby endangering the operation of the hydropower plant. Therefore it is important to design them so that they are as effective as possible and capture floating objects. These captured objects must be subsequently removed from the screenings. To avoid manual cleaning self-cleaning screenings can be designed.

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FILTRATION MATERIALS USED IN GROUND WATER TREATMENT

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Abstract

Approximately 86 % water in water supply systems are from ground water sources. In many cases this water has to be treated to be suitable with requirements listed in Government Regulation of Slovak Republic No. 496/2010 for drinking water. There are presented filtration materials with their main properties, composition and application, which are used for removal of iron and manganese from water (Greensand, Birm, MTM, Klinopur-Mn, Everzit-Mn, Cullsorb-Mn) and materials used for removal of heavy metals (GEH, CFH18, Bayoxide E33, Asmet, DMI 65). Most of these materials are commonly applied in water treatment, but some of them are new.

Keywords

Filtration materials, iron-based sorption materials, removal of iron, manganese and heavy metals, drinking water

1 INTRODUCTION

For the supply of drinking water in Slovakia are used mainly groundwater resources (87.3% of inhabitants are supplied with drinking water from underground resources). In terms of quality of ground water which is used for drinking purposes the main indicators are the amount of iron, manganese, ammonium, heavy metals (e.g. arsenic, antimony, copper, lead), etc. Furthermore, we can also classify here the content of CO_2, hydrogen sulphide and microbiological quality of water. In our conditions the treatment is mostly related to undesirable concentrations of iron, manganese and heavy metals. These technological processes of treatment are demanding not just from the aspect of investment but also from the aspect of operating costs. Due to this fact the engineers are continually looking for ways to streamline these processes [1].

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2 MATERIALS USED IN CONTACT FILTRATION

By adding potassium permanganate on the surface of the filtration material occurs the formation of film layer which works as a catalyst for oxidation so the filtration material is coated with higher metal oxides, which cause the oxidation of dissolved iron and manganese [2-5]. This special filtration is called a contact filtration, filtration on manganese filters. Currently around the world there are different types of materials with formed surface oxidation layer. Some of these materials are: Klinopur Mn, Greensand, Birm, Culsorb M, Everzit Mn and material MTM. Furthermore, we made an experiment with materials: Klinopur-Mn, Greensand and Birm to compare their effectiveness in manganese and iron removal process.

In Slovakia, an acceptable concentration of iron and manganese in drinking water is defined under the Regulation 496/2010, the limit value for manganese in drinking water is 0.05 $mg.l^{-1}$ and for iron it is 0.2 $mg.l^{-1}$.

2.1. Klinopur-Mn

We can find Klinopur-Mn with trading name zeolite on the eastern part of Slovakia. Its surface is industrially activated with potassium permanganate ($KMnO_4$) solution. The main rock-forming mineral of this material is the clinoptilolite. On the surface of the clinoptilolite is industrially produced layer from manganese oxide which allows this material to use it in contact filtration. The advantage of this material is its lower density which reduces the amount of wash water compared by using the commonly used modified filtration sand.

In addition to clinoptilolite in this natural mineral are present resin, cristobalite and transitional forms between cristobalite and opal. These clinoptilolite type of Zeolites are able to keep own structure in wide range of pH, from 1,0 to 11,5 (generally it is not valid to zeolites, but the ratio of Si/Al is important). Slovak zeolite with primary cation K^+ has a very good thermal characteristics and it can withstand temperatures up to 600 C° [6].

2.2. Birm®

Birm is a granular filtration material (imported from the USA) which is primarily used to remove iron and manganese from water. It is a specially developed material containing MnO_2 film on the surface (catalyst). It is recommended to use Birm for lower iron concentrations (to Fe^{2+} concentration of 6,0 $mg.l^{-1}$ and Mn^{2+} about 3,0 $mg.l^{-1}$) and for household water treatment. It can also be used in gravity or pressure filters. Birm reacts as an insoluble catalyst to increase the reaction between dissolved oxygen and iron compounds which are commonly found in the water in dissolved form and they cannot be removed by simple filtration. After contacting the raw water with Birm there is rapid oxidation of divalent and trivalent iron and then occurs the production of iron hydroxides in the clots. Subsequently, precipitated Fe and Mn hydroxides (pH of 8 to 9 is required for Mn removal) are easily removed by filtration.

Filter medium is cleaned by backwashing. There is no need of chemical regeneration. In cleaning process the time of backwashing and wash water velocity are important factors. Long service life is also one of the advantages of this medium.

2.3. Greensand®

Greensand is a mineral from zeolite origin (imported from the USA). This type of filtration material is made from sand which is activated by potassium permanganate. The final product is a granular material that is coated with layer of MnO_2 and with other higher oxides of manganese.

Greensand with activated surface known as the manganese greensand is used in water treatment technology for removal of dissolved iron and manganese. Dissolved iron and manganese are oxidized and precipitated by contact with higher oxides of manganese on the surface of granulated greensand. After filtration are precipitates removed from a filter surface by back-washing. After exhausting of the oxidative capacity of manganese greensand, filtration bed must be regenerated with solution of potassium permanganate ($KMnO_4$).
Physical characteristics of filtration media are listed in the table 1.

Tab. 1 Physical characteristics of filtration media

	Klinopur-Mn	Greensand	Birm
Colour	Dark brown	Black	Black
Density [$kg.m^{-3}$]	2200-2440	2400-2900	2000
Bulk density [$kg.m^{-3}$]	1600-1800	1360	750-800
Granularity [mm]	0.3 – 2.5	0.25 – 1.0	0.4 – 2.0

2.4. The experiment

Experiments to compare the efficiency of filtration materials have done in the Water treatment plant Kúty (it has an underground water source). The raw water in this area according to the requirements of the Government Regulation of Slovak Republic (No. 496/2010) does not meet the limit values of iron, manganese, ammonium and aggressive carbon dioxide (CO_2).
The experimental equipment – filtration columns were placed right behind the aerator Inka (1st sampling point) and were placed on the section between the filter and lime dosing (2nd sampling point). During experiments the quality of raw water (amount of Fe and Mn) was monitored on the feed side and also the quality of treated water was monitored on the outlet side of each column. The basic parameters of water are listed in the table 2 and scheme of treatment process in WTP Kúty is on figure 1.

Tab. 2 The basic parameters during the pilot test

Parameter	Raw water	1st sampling point	2nd sampling point
Fe [$mg.l^{-1}$]	2.28 – 5,16	0.90 – 3.87	0.96 – 4.22
Mn [$mg.l^{-1}$]	0.82 – 1.12	0.816 – 1.092	0.168 – 0.,524
pH	6.64 – 6.98	6.81 – 7.14	8.40 – 8. 62
Oxygen (% saturation)	6 – 7	59 – 60	56 – 57

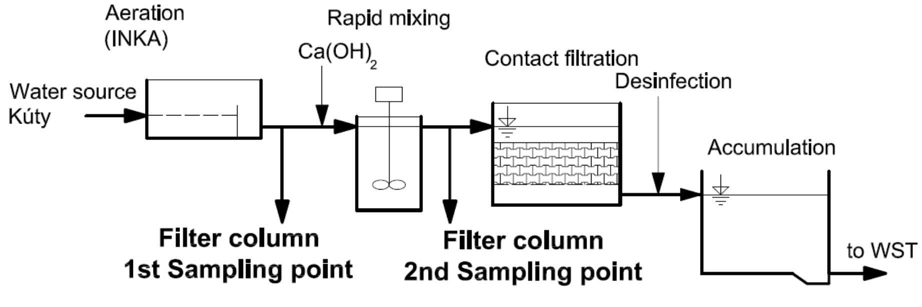


Fig. 1 Scheme of WTP Kúty (1st and 2nd sampling point)

Results and discussion

The results of the removal of Mn and Fe from the water in 1st and 2nd sampling points show the best Figure 2 and 3, where there are illustrated concentrations of manganese and iron in aerated water. The graphs are also shown manganese limit value (0.05 $mg.l^{-1}$) and iron (0.2 $mg.l^{-1}$) in drinking water legalized according to the Government Regulation No. 496/2010. The arrows on the graphs are the regeneration time of filter material.

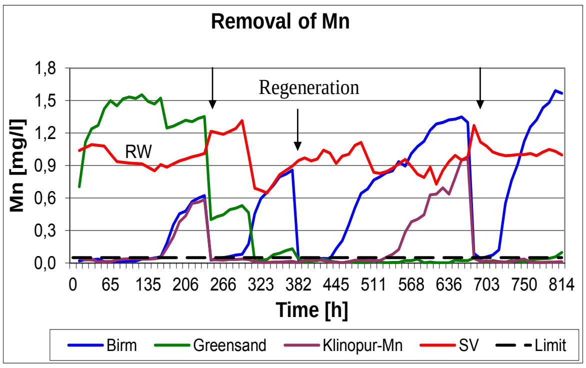
1st Sampling point

The filtration materials were washed continuously (approximately every tree days) by back flow of water (according to the amount of collected precipitated ferric hydroxide). Over the time as it is shown in figure 2 (graph of removing Mn) when water passed through the Klinopur-Mn, manganese concentration at the beginning in treated water have increased the value up to 0.05 mg.l-1 because at this point the cartridge from Klinopur were regenerated with potassium permanganate solution (0.5 % solution). After the regeneration the values of dissolved manganese in treated water meet the Government Regulation of Slovak Republic No. 496/2010.

Material Greensand, after progressive washing and regeneration gained its required properties and its efficacy has been gradually improved. Material Birm was at the beginning of the experiment effective, however then it started to lose the required effect and also regeneration with $KMnO_4$ which wasn't sufficient enough. The results have been shown, that on the removal efficiency of Mn from aerated water has a significant effect of pH value as well as the high manganese concentration at the inlet to the filter columns.

2nd sampling point

The results of the removal of Mn and Fe from the water at the second sampling point (between the filters and lime dosing, pH about 8.4) are shown on figure 3 which shows the concentration of manganese and iron before entering into the filter columns and after passing through the different filter materials. The pH value modification of water had a significant impact on the efficiency of removal manganese from the water. As it is shown in Figure 3 (removal of Mn) the values of manganese have been decreased at the inlet into the filtration columns (0.168-0.524). In this case of Klinopur-Mn, at the outlet from filtration columns have been achieved by the significant treatment where Klinopur-Mn even after 783 hours did not exceed the limit value of manganese (0.05 $mg.l^{-1}$). The effectiveness of other filter materials (Birm, Greensand) was also increased significantly.



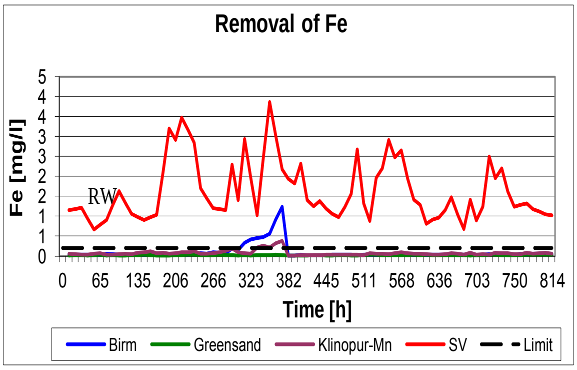


Fig. 2 Removal of manganese and iron during the water filtration in 1st sampling point

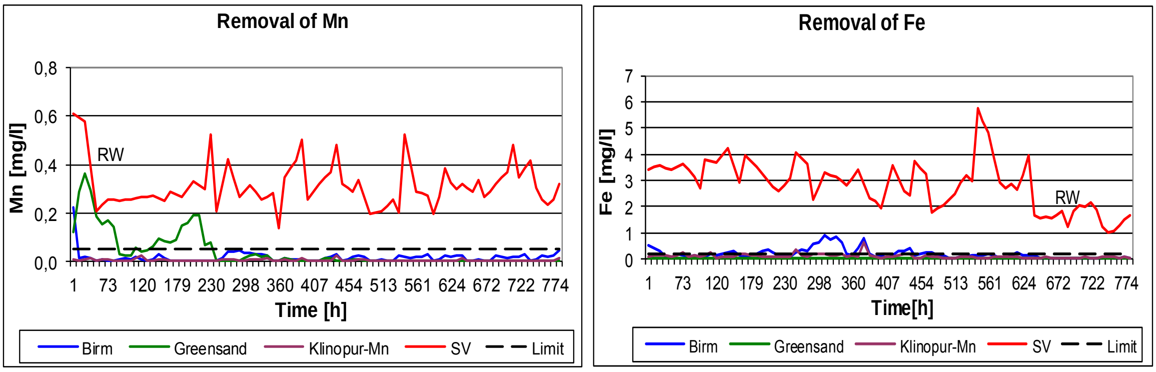


Fig. 3 Removal of manganese and iron during the water filtration in 2nd sampling point

The inclusion of aeration of raw water before filtration column increases an efficiency of the filter material and ensures trouble-free water treatment. With increasing the oxygen content in raw water an oxidation occurs from Fe_2^+ form on Fe_3^+ form, which can be caught on the filters. Dissolved oxygen also increases an oxidation surface of Klinopur-Mn, which increases its oxidative capacity required for the removal of manganese. Improving the efficiency of iron and manganese removal from water and also to protect the filter materials are preferred to use a dosage of alkaline agents (e.g. lime milk) in the treated water to achieve higher pH value (any value above 7.6).

3 SORPTION MATERIALS FOR REMOVAL OF HEAVY METALS

Sorption is a simple (regarding its operation) and effective method of heavy metal removal mainly because of a wide range of sorption materials (natural and synthetic sorbents) that can be used in this process. The most frequently tested sorbents are as follows: iron oxides and oxihydroxides, activated alumina, sand covered by iron hydroxide, activated carbon, magnesium hydroxide, media containing TiO_2 or MnO_2 layers on the surface and other [7, 8].

Adsorption by iron oxides and oxihydroxides (Bayoxide E33, CFH12, CFH18, GEH, etc.) represents an efficient and cost-effective method for the removal of heavy metals from water. These materials are very efficient in the removal of arsenic from water. The objective of this study was to verify the sorption properties of these sorption materials for the removal of antimony from selected water resources and compare their efficiency. In Slovakia, an acceptable concentration of heavy metals in drinking water is defined under the Regulation 496/2010. The limit concentration for antimony is 5 $\mu g.l^{-1}$. This limit value is in accordance with the WHO Recommendations (WHO, 2004) and the EU Directive (98/83/EC, 1998).

3.1. Bayoxide E33

Bayoxide E33 is a granular iron oxide-based medium. It was developed by Severn Trent in cooperation with Bayer AG for removal of arsenic and other contaminants from water. The advantage of this material is the ability to remove As(III) and As (V) together with iron and manganese. Under high pH conditions, high levels of vanadium, phosphate (>1.0 mg/L) and silica (>40 mg/L) can present interference and reduce the media's adsorption capacity for arsenic. Bayoxide E33 has a capacity to treat water with As concentration of 11÷5000 $\mu g.l^{-1}$ and Fe concentration of 50÷10 000 $\mu g.l^{-1}$ [9, 10].

3.2. CFH12 and CFH18

CFH12 and CFH18 are granular sorption materials on the basis of iron oxyhydroxides. They were developed by Kemira Finland as efficient products for removal of arsenic and other contaminants from water by adsorption. The advantage is their high adsorption capacity (4.9 g As^V per 1 kg of material), higher efficiency at lower costs, providing that the adsorption capacity is fully used (optimum filtration, backwash and pH). CFH 12 and CFH18 differ from each other by their grain size [11-12].

3.3. GEH

The GEH was obtained from the supplier (GEH Wasserchemie, Germany). GFH is a sorption material developed by the Department of Water Quality Monitoring of the University of Berlin for the purpose of arsenic removal from water. Treatment technology includes contaminant sorption process using granular ferric hydroxide placed in the reactor through which treated water flows [13-16].

The table 3 includes basic physical-chemical properties of the sorption materials used in the removal of heavy metals, especially for removal of arsenic.

Tab. 3 Physical and chemical properties of selected sorption materials

Parameter	Bayoxide E33	CFH12 a CFH18	GEH
Matrix/ Active agent	Fe_2O_3>70% and 90,1% α-FeOOH	FeOOH Fe^{3+} >40%	$Fe(OH)_3$ and 52-57% β-FeOOH
Physical Form	dry granular	dry granular	moist granular
Color	amber	brown-red	dark brown
Bulk density [$g.cm^{-3}$]	0.45	1.12-1.2	1.22-1.29
Specific Surface Area [$m^2.g^{-1}$]	120-200	120	250-300
Grain Size [mm]	0.5-2.0	0.32-2.5 or 0.8-1.8	0.32-2.0
Grain Porosity [%]	85	72-80	72-77
pH	6.0-8.0	6.5-7.5	5.5-9.0

3.4. The experiment

Technological tests were carried out at the facility of the Slovak Water Company, Liptovský Mikuláš Branch Company, in the locality of Dúbrava (former chlorine plant) with a spring capacity of approximately 40 $l.s^{-1}$. Based on the groundwater analyses the antimony concentrations in the raw water ranged from 55 to 62 $\mu g.l^{-1}$ during experiments. No other heavy metals were presented in the water. The objective of this work was to verify sorption properties of granular filter materials (GEH, CFH12, CFH18, Bayoxide E33) in the water resource of Dúbrava within the process of antimony removal from water.

Model Filtration System

The effectiveness of antimony removal was verified using adsorption columns containing selected sorption material. Adsorption column was made of glass with a diameter of 5.0 cm.

A height of media was 50 cm, and the total height of glass column was 80 cm.

Raw water (Brdáre spring) passed through the filtration system and the concentration of antimony was monitored in raw and treated water at the outlet of filtration columns. Simultaneously, the flow rates were measured at the outlet of each column. The system of several valves was used for feeding the water for filtration system (from top to bottom) and for filter backwash (from bottom to top) as well as for regulating filtration rates.

Results and discussion

The first stage of the model tests was aimed at monitoring of the efficiency of sorption materials GEH, CFH12 and Bayoxide E33 for the removal of antimony from water. The concentrations of antimony in raw water ranged from 55 to 62 μg.l^{-1} (average of 58,29 μg.l^{-1}), media height were 50 cm and the filtration rates were 4.7-5.3 m.h^{-1} for GEH, 4,3-4,9 m.h^{-1} for Bayoxide E33 and 4,3-5,1 m.h^{-1} for CFH12.

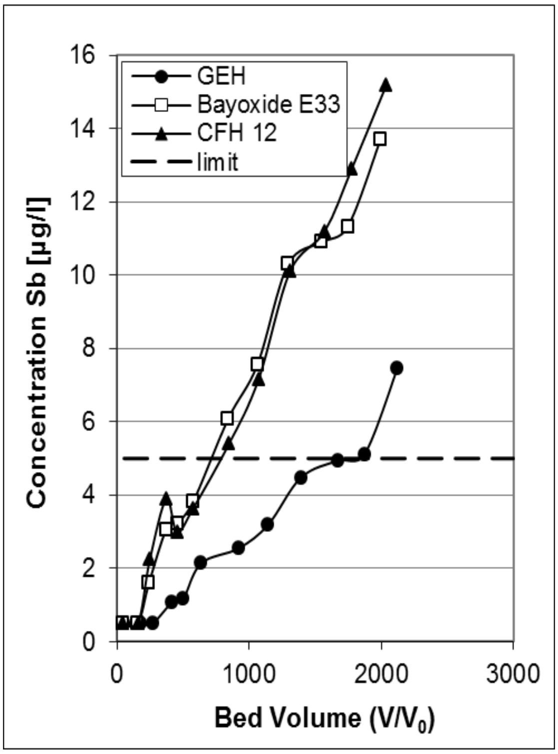
The figure 4 shows the concentration curve of antimony at the outlets of adsorption media in relation with V/V_0 ratio (where V is a volume of treated water that flowed through the column in a given time and V_0 is a volume of adsorbent media). This ratio is known in the literature as "Bed Volume".

The figure 5 presents the values of V/V0 ratios for each sorption material when reaching the limit concentration of antimony (5 μg.l^{-1}). Moreover, the figure indicates the adsorption capacity for the values of V/V_0 ratios. Considering the minimum differences in filtration rates and based on the results presented in the figures 2 and 3, it can be concluded that GEH is the most suitable material for antimony removal compared to other sorbents used in the test. The following V/V_0 ratios were measured for the antimony concentration (5 μg.l^{-1}) at the outlet of adsorbent media: V/V_0 = 1700 for GEH, V/V_0 =790 for CFH 12 and V/V_0 =715 for Bayoxide E33. The adsorption capacities were as follows: GEH = 83.6 μg.g^{-1}, Bayoxide E33 = 91.4 μg.g^{-1} and CFH 12 = 42.4 μg.g^{-1}.

Within the frame of experiments were studied chemical composition of used sorption materials. Chemical composition were determined by the Institute of Inorganic Chemistry of the Faculty of Chemical and Food Technology of the Slovak University of Technology, the values are listed in the table 4.

Tab. 4 Chemical composition of selected sorption materials

Material	Compound in mass [%]								
	MgO	Al_2O_3	SiO_2	P_2O_3	SO_x	K_2O	CaO	TiO_2	Fe_2O_3 (FeOOH)
E33	0.97	6.59	12.75	0.34	0.31	0.37	2.01	0.91	75.28
CFH12	3.75	0.45	1.18	-	8.49	0.27	2.72	0.50	82.65
CFH18	5.19	0.48	1.47	0.28	4.58	-	1.41	0.30	86.29
GEH	-	1.74	3.05	0.21	0.54	0.08	0.18	-	91.92



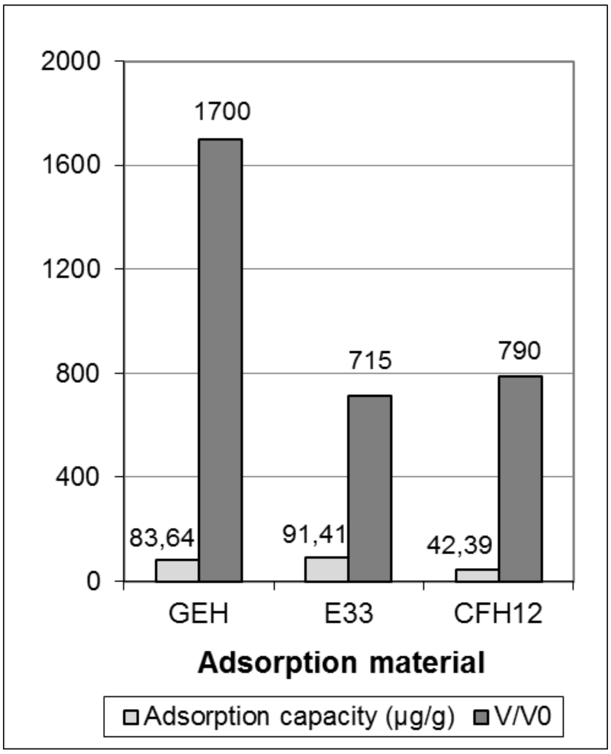


Fig. 4 The concentration of antimony at the outlets of adsorption media in relation with V/V_0 ratio (average concentration of Sb in raw water 58,29 $\mu g.l^{-1}$, filtration rate 4.3-5.3 $m.h^{-1}$)

Fig. 5 The values of V/V_0 ratio and adsorption capacity for each sorption material when reaching the limit concentration of antimony (5 $\mu g.l^{-1}$)

Performed technological tests of ground water from the spring in the locality of Dúbrava showed that the use of sorption materials can possibly decrease the content of antimony in water to the values limited by the Government Regulation No. 496/2010 for drinking water. The results proved that the materials CFH12, CFH18 and Bayoxide E33 can be also used to decrease the concentration of Sb in drinking water below the limit value of 5 $\mu g.l^{-1}$. The adsorption capacities and V/V_0 ratio are lower for these sorption materials. Based on the pilot tests, the most suitable sorption material for antimony removal from water is GEH.

4 CONCLUSION

The quality of raw water is different for every location, at the beginning of consideration of a new water treatment plant or reconstruction and modernization of an existing water treatment plant should be requirements proceeded for modern technological processes which have been documented by a reliable data (especially based on good theory and well-made pilot experiments).

Acknowledgement

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PLANNING WASTEWATER COLLECTION AND TREATMENT IN AGGLOMERATIONS BELOW 2.000 PE IN PROTECTED AREAS

G. Ćosić-Flajsig[1], B. Karleuša[2] and B. Kompare[3]

Abstract

Settlements with less than 2.000 inhabitants, generally with rural character, account for approximately 60 % of the total population of the Republic of Croatia, and cover 70 % of its territory. There are 469 preliminary agglomerations in the Republic of Croatia below 2.000 PE and account for 15 % of Croatia's population. It is important to emphasize that 45 % of those settlements with less than 2.000 inhabitants are part of agglomerations larger than 2.000 PE. Simultaneously, a large number of inhabitants in agglomerations below 2000 PE live in protected areas and 410 agglomerations below 2.000 PE discharge the waste water into a sensitive areas, or discharge wastewater into small recipients without temporary flow or into the underground. The stricter water protection, by implementation of "adequate treatment level" and by "combined approach", are prerequisites for the achievement of good water status and fulfilling several environmental protection objectives for river basin.

Keywords

agglomerations below 2.000 population equivalent PE, small urban waste water treatment plants UWWTPs, protected areas, combined approach, environmental protection objectives for river basin, good water status

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1 INTRODUCTION

The general demographic picture of the Republic of Croatia is that of relatively poor population density, 78.5 inhabitants per km². Croatia belongs to more sparsely populated European countries with a negative growth rate. Settlements with less than 2.000 inhabitants, which are generally of rural character, account for approximately 60 % of the total population of the Republic of Croatia, and cover 70 % of its territory.

There are 763 agglomerations planned in the Republic of Croatia, of which 294 are larger than 2.000 PE (Population equivalent). Agglomerations below 2.000 PE account for 15 % of Croatia's population, but 410 agglomerations below 2000 PE discharge the waste water into a sensitive area. It is important to emphasize that 45 % of the settlements with less than 2000 inhabitants are part of agglomerations larger than 2.000 PE.

Simultaneously, a large number of inhabitants in agglomerations below 2.000 PE live in protected areas for which there is a stricter water protection regime in place, and this protection is achieved by fulfilling several environmental protection objectives for water bodies that are receiving water for wastewater, in accordance with which "adequate treatment level" shall be implemented.

The paper will offer an overview of results, including analysis, for:

- agglomerations below 2.000 PE and settlements below 2000 inhabitants
- European and national legislation related to sewage system and waste water treatment for agglomerations below 2.000 PE and effluent limit values with regard to receiving water and environmental impact assessment,
- different approach to definition of pollution sources and biodegradable loads for agglomerations below 2.000 PE,
- wastewater treatment and sludge disposal technology with regard to the requirement for implementation of "adequate treatment level".

2 SETTLEMENTS BELOW 2.000 INHABITANTS AND AGGLOMERATIONS BELOW 2000 PE

2.1 Settlements below 2000 inhabitants

In settlements below 2.000 inhabitants, mostly of rural type, about 39.5 % of all Croatian inhabitants live (Fig. 1 and Tab. 1). That is 1.664.400 inhabitants [1].

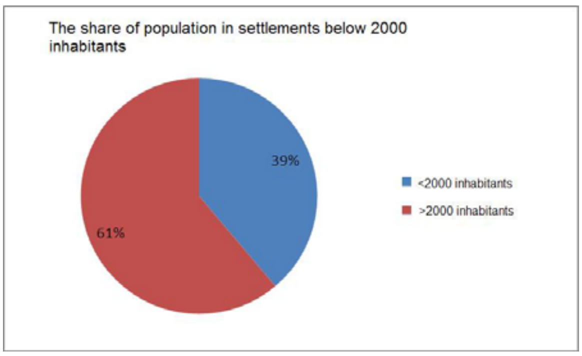


Fig. 1 The share of the population in settlements below 2.000 inhabitants and total number of inhabitants [1]

Tab. 1 Settlements related to the N° of inhabitants [1]

	N° of settlements	N° of inhabitants	%	
			settlements	inhabitants
Together	**6.756**	**4.284.889**	**100,00**	**100,00**
below 2000	**6.535**	**1.664.400**	**96,73**	**39,5**
2 001-5 000	143	434.201	2,12	10,13
5 000-10 000	39	264.060	0,58	6,16
10 001-50 000	31	617.909	0,46	14,42
50 001-100 000	5	320.651	0,07	7,48
100 001-200 000	2	295.505	0,03	6,90
200 001 and more	1	688.163	0,01	16,06

The number and percentage of connected and not connected inhabitants to sewage system, related to the size of settlement, is presented in Tab. 2.

Tab. 2 Analysis of the connected and not connected to sewage system inhabitants of the settlements below 2000 inhabitants [1]

Size of the settlement	Connected		Not connected		Total
N° of inhabitants	N°	%	N°	%	N°
<2.000	**111.717**	**6**	**1.640.828**	**94**	**1.752.545**
2.000-10.000	267.602	37	455.883	63	723.485
10.000-50.000	501.527	74	172.036	26	673.563
>50.000	1.042.126	81	245.741	19	1.287.867
Total	1.922.972	43	2.514.488	57	4.437.460

There are 6.535 settlements below 2.000 inhabitants of the total number of 6.756 settlements, that is 96.73% (Tab. 2).

Tab. 3 Analysis of the relation between inhabitants and settlements below 2.000 inhabitants and the total number [1]

	N° of settlements	N° of inhabitants	%	
			Settlements	inhabitants
Total	**6.535**	**1.664.400**	**96,73**	**39,5**
Without inhabitants	150	-	2,22	-
Below 100 inhabitants	2.653	113.914	39,27	2,66
101 – 200	1.318	192.193	19,51	4,49
201 – 500	1.448	461.114	21,43	10,76
501 – 1.000	658	462.788	9,74	10,80
1.001 – 1.500	195	240.133	2,89	5,60
1.501 – 2.000	113	194.258	1,67	4,53

Related to Tab. 3 and Fig. 2 and 3, the settlements of less than 100 inhabitants have the highest percentage in the total number of settlements, about 39,27%, but the settlements from

200 to 500 inhabitants and from 500 to 1.000 inhabitants have the highest percentage of inhabitants, each group about 10%.

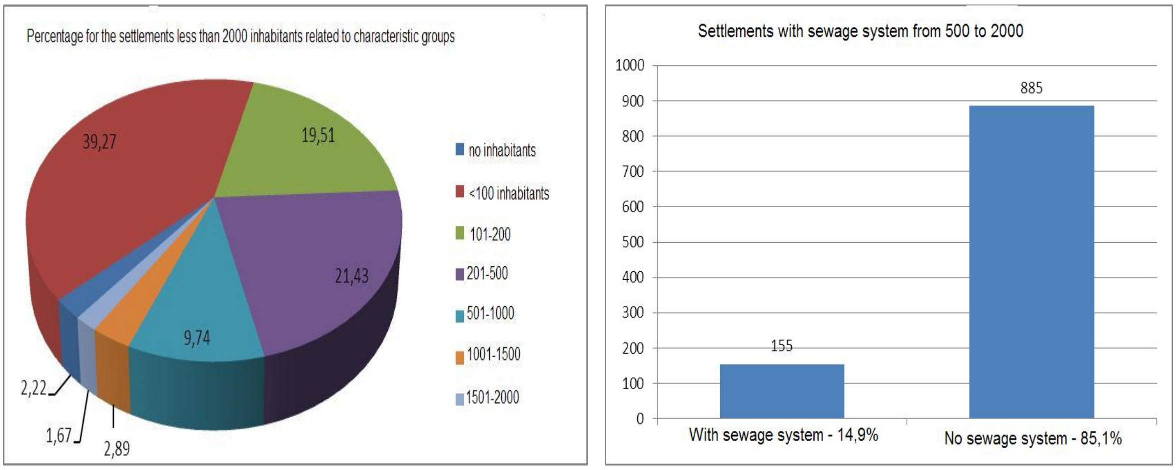


Fig. 2 and 3 Percentage for the settlements less than 2.000 inhabitants related to characteristic groups and settlements with sewage system from 500 to 2.000 inhabitants [1,2]

From the above graphical presentation (Fig. 2 and 3), the largest percent of the population lives in settlements of 201-500 inhabitants and 501-1000 inhabitants. The smallest percentage of settlements is of settlements below 100 inhabitants, even the number of these settlements is the largest in the total number of settlements.

The biggest problem of the settlements below inhabitants in Croatia is the lack of the sewage system and wastewater treatment plant. The total number is 1.752.545 inhabitants from settlements below 2000 inhabitants, but just 111.717 inhabitants is connected to the sewer system (6,3 %) and just 49.974 inhabitants are connected to wastewater treatment plant (3%).

Small settlements below 2.000 inhabitants are not connected to sewage systems and at the same time in them a great number of population of Republic of Croatia lives (Fig. 4).

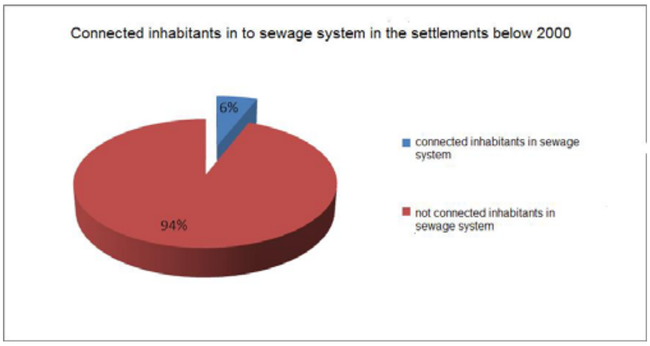


Fig.4 Connected inhabitants in to sewage systems in the settlements below 2.000 [2]

Only 155 settlements have the sewage system: 79 settlements have separate system, 59 settlements have combined system and 17 settlements have mixed system.

2.2 Agglomerations below 2.000 PE

The selected concept of identification of agglomerations in the Republic of Croatia implies that an area of one agglomeration is served by one collecting system and one waste water treatment plant assessed as the most appropriate in terms of the present situation. A more detailed analysis of current state, carried out during the negotiating process, has provided more precise data and estimates of the percentage of population connected to public waste water systems.
The collection of waste water from 4.437.460 inhabitants of the Republic of Croatia living in a total of 6.762 settlements can be organized in 763 systems, i.e. agglomerations. Only 294 agglomerations are larger than 2.000 PE.
However, the greatest number of agglomerations are very small ones, below 2.000 PE (469 out of 763). The requirements of the Urban waste weather treatment directive (UWWTD) refers primarily to agglomerations larger than 2.000 PE.
Out of the total estimated existing maximum load, agglomerations larger than 2.000 PE will encompass around 93%. These agglomerations encompass the total of 1.783 settlements with 3.547.000 inhabitants which should connect directly to the sewerage system, representing 80% of Croatia's population according [1].
The spatial coverage of agglomerations and their loads will in the future be adjusted to the changes of spatial conditions, i.e. to the changes in the number of users, economic trends, but also to the financial capacities and the standard of living. The definition of agglomerations thus becomes a continuous process, an integral part of planning and managing this activity, coordinated with generally accepted water protection principles, with the aim of minimizing the impacts on water and environment in a wider sense, with minimum costs, coordinated with the capacities of users.
More precise planning of the scope and load of individual agglomerations, including a consideration of realistic capacities of development, operation and maintenance of the system's facilities, will be the result of subsequent more thorough analyses which will be carried out during the preparation of individual feasibility studies which will study an area in much more detail than at this top planning level with information available so far.
Graphical presentation of designated agglomerations in Croatia comes from the negotiating platform in the pre-accession process to the EU, are identified that an agglomeration = one public sewerage system. Based on the analysis on the planned public sewerage system preliminary agglomeration with the size and spatial coverage were made.

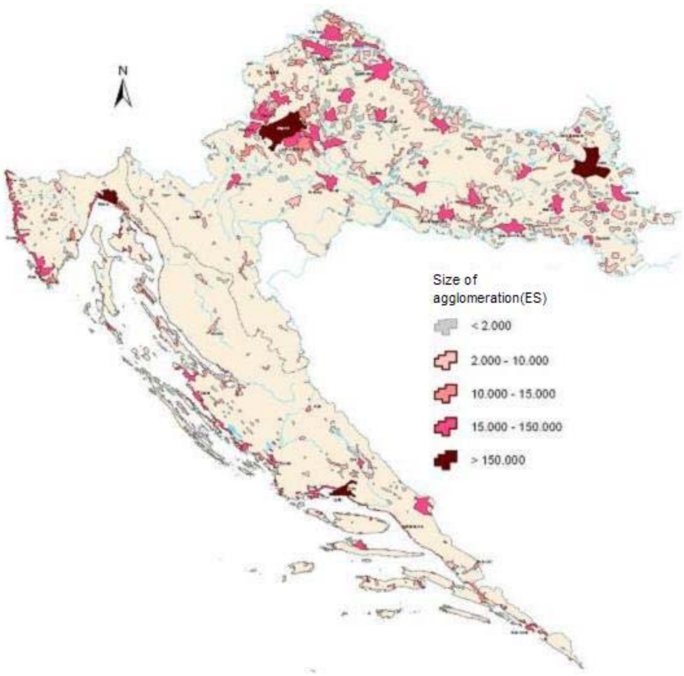


Fig. 5 Spatial distribution of agglomerations [3]

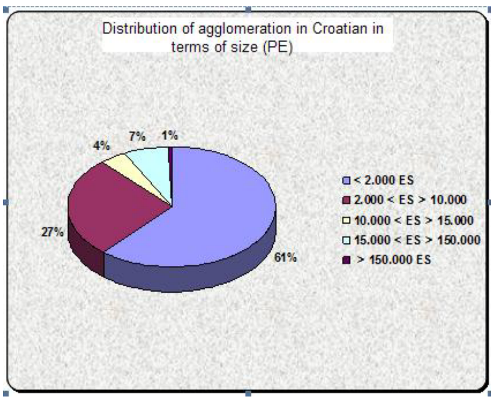


Fig. 6 Distribution of agglomeration in Croatian in terms of size (PE) [3]

It is anticipated that 88 % of Croatian citizens are connected to the public sewage systems. According to data from 2010 [3], about 50 % of population is connected and only 27% is connected to wastewater treatment plants.

3 PROTECTED AREAS

The protected areas are areas with special protection proclaimed related to legislation for the purpose of protection of surface water and groundwater, also as water ecosystems and ecosystems dependent on the water.
The register of protected areas includes the following types of protected areas:
- areas designed for the abstraction of water intended for human consumption;
- areas designated for the protection of economically significant aquatic species;
- bodies o water designated as recreational waters, including areas designated as bathing waters,
- nutrient-sensitive areas, including areas designated as vulnerable zones and areas designated as sensitive areas;
- areas designated for the protection of habitats or species where the maintenance or improvement o the status o water is an important factor in their protection, including Natura 2000.

3.1 Presentation of the agglomerations in sensitive areas

According to the requirements of Directive 91/271/EEC and the Implementation Plan for Water Utility Directives (2010) [3], also as the level of the wastewater treatment, analyses of agglomerations bigger than 10.000 PE in sensitive areas have been prepared and presented on the map (Figure 7).

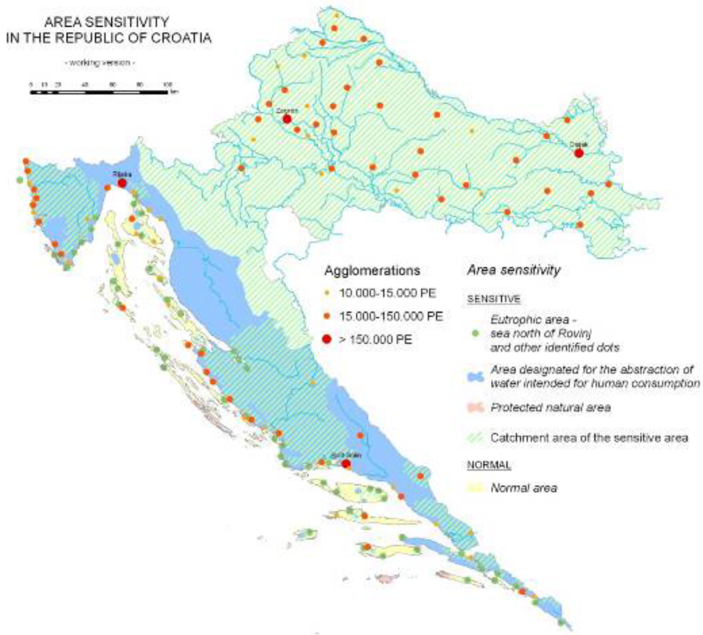


Fig. 7 Map of the sensitive areas and agglomerations bigger than 10.000 PE [4]

Related to Croatian Waters (data from 2010.), 410 agglomerations below 2.000 PE discharge wastewater in sensitive area, with 359.316 PE. Total planned potential load is 449.983 PE. Tab. 5 and 6 present the type of sewage systems and the level of the wastewater treatment for agglomerations below 2.000 PE.

Tab.4 Number of agglomerations below 2.000 PE in sensitive areas regarding the type of sewage system [5]

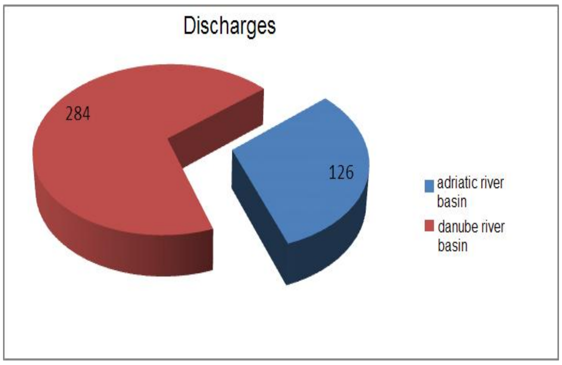
Number of agglomerations	Sewage systems			
	combined	mixed	separate	Not defined
Sensitive areas	19	57	289	45

Tab. 5 Number of agglomerations below 2.000 PE in sensitive areas regarding the level of wastewater treatment [5]

Number of agglomerations	Level of the wastewater treatment		
	pre-treatment	1. level	2. level
Sensitive areas	19	57	289

Related to data of Croatian Waters from 2010 [5] , it is planned that about 87 % of agglomerations below 2.000 PE will have recipients in sensitive areas with total load of 359.316 PE.

Also, related to Fig. 8 and 9, from the total number of 410 agglomerations below 2.000 PE, 284 (69 %) of agglomerations will have discharge in sensitive areas of Danube River Basins and 126 (31%) agglomerations will have discharge in sensitive areas of Adriatic Sea Basin.



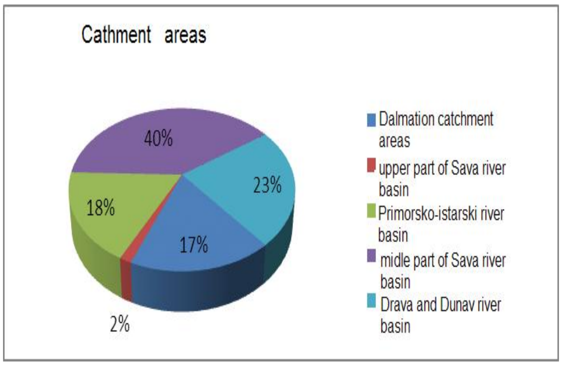


Fig. 8 and 9 Discharges of agglomerations below 2000 into sensitive areas divided to river basins [Hrvatske vode, 2010] Discharges of agglomerations below 2000 into sensitive areas divided to catchment areas [5]

The presentation has been prepared related to the requirements of the Ordinance of limit values emissions in wastewater (NN 81/10) [6], definition the level of wastewater treatment and „adequate wastewater treatment level" for agglomerations below 2.000 PE.

That means wastewater treatment by any process that provide good water status of the recipient water body. Related to the ecological water status, wastewater management can have a great impact on the water body quality, specially for small rivers with low flow and groundwater in karstic area.

4 EUROPEAN AND NATIONAL LEGISLATION FOR AGGLOMERATION BELOW 2.000 PE

The Urban Waste Water Treatment Directive (91/271/EEC) is one of the legislative core elements of water protection policy in Europe. Adopted in the year 1991 this Directive regulates discharges of urban waste water from larger villages and towns and explicitly specifies, which kind of treatment must be installed in EU-27. The Directive applies to agglomerations generating a pollution load of more than or equal to 2000 population equivalents (PE). Around 22.900 settlements generate an organic pollution load of approximately 604 million PE all over Europe and have to be equipped with collecting and treatment systems for their urban waste waters according to the Directive. Furthermore, the Directive foresees the designation of sensitive areas (mostly sensitivity to eutrophication due to nitrogen and/ or phosphorus) and application of more stringent treatment to the urban waste waters in these areas and their relevant catchment areas.
In order to meet the relevant water quality objectives, Article 7 of the UWWTD stipulates the need to implement an appropriate waste water treatment for agglomerations less than 2.000 PE for discharges to fresh-water and estuaries, respectively for discharges of agglomerations less than 10.000 PE to coastal waters.
Also, as other countries of European Union, Croatia had transposed and implemented into the Croatian water policy all statements of the Urban Waste Water Treatment Directive (91/271/EEC).

4.1 European experience related to planning wastewater treatment in agglomerations below 2.000 PE

Adopted in the year 1991 this Directive regulates discharges of urban waste water from larger villages and towns ("agglomerations"). Principally the Directive requires that all European agglomerations with a size of more than or equal to 2.000 PE are equipped with collecting and treatment systems for their waste waters.
In order to get insight into waste water treatment infrastructure for agglomerations less than 2,000 PE the European Commission granted a “Study on small scale waste water treatment in Europe” , final report, BOKU- SIG, Vienna, 2011 [7]
In the scope of the study the term „small scale“ refers to all available technologies for waste water treatment up to a capacity of 2.000 PE
The overall objective of this study is to get an overview of the waste water treatment of agglomerations with less than 2.000 PE in EU-27 Member States, including in particular information on the number and loads of agglomerations below 2.000 PE (best estimates), percentage of waste water collected in sewage systems, percentage of waste water treated onsite (best estimates), percentage of collected waste water treated in an urban waste water treatment plant (UWWTP) (best estimates), agglomerations' waste water treatment schemes, mid-term planning perspectives (10-15 years) for the waste water collection and treatment schemes of agglomerations below 2.000 PE. (including financial perspectives) as well as reference to national or regional legislation related to small scale waste water treatment.

The objective of this study is to complement the already available information of the waste water situation for agglomerations ≥ 2.000 PE for the assessment of potential needs for further action in agglomerations below this threshold. This information about small scale waste water treatment related to population ratio living in rural respectively urban areas (Figure 10) can support the European Commission in defining future priorities e.g. in the field of EU funding instruments.

From the environmental point of view, waste water management can have a significant influence on the water quality in water bodies. The link between the Urban Waste Water Treatment Directive and the Water Framework Directive is therefore of high importance. In particular in small river catchments with low water flow also small scale waste water systems in agglomerations below 2.000 PE can considerably pollute the water bodies. If designated water bodies fail the good biological and/or good chemical status, then waste water treatment measures in small agglomerations can be applied to improve the water quality.

The knowledge about small scale waste water treatment facilities in EU-Member States is very limited. This information is not part of the regular UWWTD reporting and therefore no structured data and information is available on European level.

A high effort for the data management of agglomerations less than 2.000 PE is necessary on the side of EU-Member States. On the other hand, it is important for the European Commission to get appropriate information on small scale waste water treatment in addition to the information gained through the regular reporting under the UWWTD. In particular, if the connection rate of the population to waste water treatment facilities in agglomerations ≥ 2.000 PE is under a certain amount, this information can be valuable for further measures.

To receive this information a study was carried out including a voluntary questionnaire, which was sent out to Member States, a literature review as well as a GIS based approach to assess agglomerations/settlements from geographical data. The use of a GIS-methodology supporting the assessment of derivation of agglomerations in the sense of the UWWTD was tried to be achieved by an approach combining different data sets available across Europe. The main limitation of such a methodology is that the settlement areas cannot be compared with agglomerations in the meaning of the UWWTD.

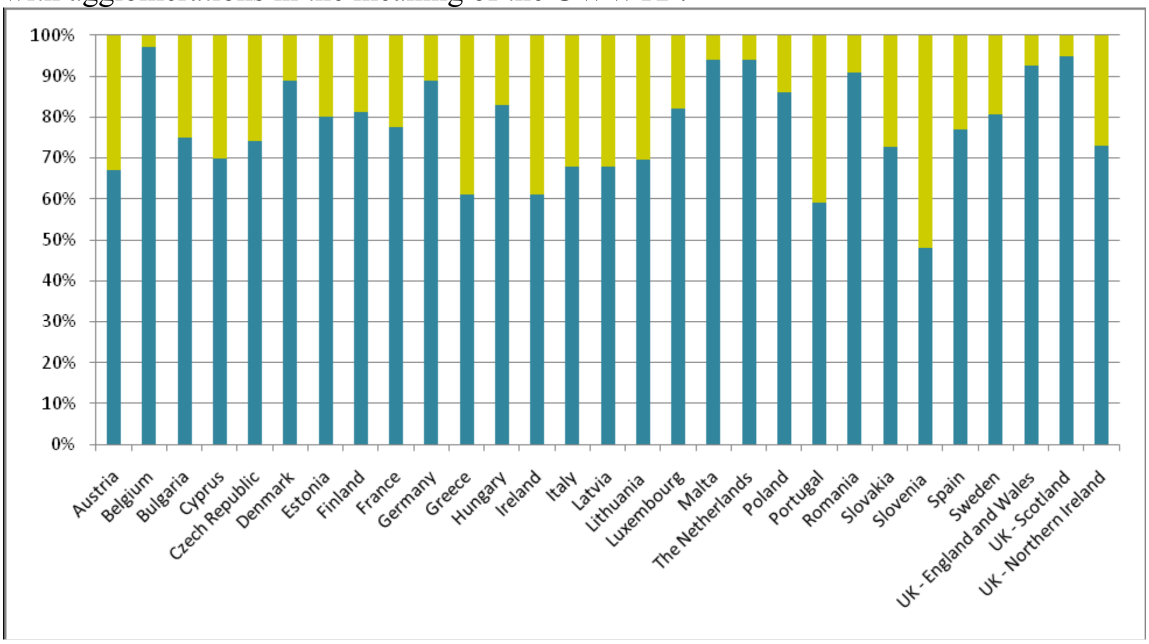


Fig. 10 Population ratio living in rural (green) respectively urban (blue) areas [7]

4.1.1 Principles of the wastewater treatment for rural agglomerations below 2.000 PE

In general rural areas are large and isolated areas of an open country with low population density, small population size, and distance from high population density and big size-major agglomerations. The inhabitants of rural areas or of small towns are classified as rural. The Organization for Economic Co-operation and Development (OECD) distinguishes between two hierarchical levels: local and regional. At local level the municipalities are deemed to be rural if they have a population density of less than 150 inhabitants/km². At regional level, larger functional or administrative units are divided according to the degree of rurality, i.e. according to the percentage of the regional population living in rural areas.

Nevertheless it has to be said that data only focusing on settlement patterns are not the only criteria to indicate which waste water management system – small scale waste water treatment vs. Agglomerations > 2.000 PE. – should be applied among Member States. There are many other determining factors which need to be taken into account for the decision of implementing a certain waste water system:

- topography: plain areas, hilly or mountainous territory
- structure of settlements: single dwelling, dense built-up areas or scattered settlements
- density of inhabitants
- receiving water body: according to water quality indirect dischargers: type and amount of commercial dischargers in the settlement
- policy or subsidies

The quality of available data through all categories (population coverage, applied systems, legal background, technical standards, subsidies and perspectives) vary from very good to very poor. General data about population coverage and small scale waste water treatment was better available than data about technical standards and future perspectives.

The information on small scale treatment shows that the efforts in Central and Eastern European countries are focused to comply with the UWWTD, which means that waste water facilities for agglomerations <2.000 PE will follow as a next step after having covered the larger agglomerations in their implementation programs. In other Member States, efforts are concentrated on the issues related to the WFD and closing gaps in rural areas that still use old sanitation infrastructure (e.g. cesspools) to reach appropriate environmental objectives.

4.1.2 European experience related to planning wastewater treatment in protected areas

Experience from Austria - The Austrian Water Act in conjunction with four ordinances determines the requirements for small scale waste water treatment. The EN 12566 series, the Austrian standard ÖNORM B 2505 series and two guidelines are relevant for small scale waste water treatment. Subsidies are granted by the Federal Ministry of Agriculture, Forestry, Environment and Water Management.

In 1993, the Environmental Support Act (Umweltförderungsgesetz, UFG) prompted a re-structuring of Federal Government funding for water supply and waste water treatment, aiming at, among others, the enhanced development of waste water disposal in rural areas. Since then, investments have mainly been boosted through annuity and investment allowances.

Basic structure of the subsidy system for small-scale waste water treatment plants under the UFG: Sewage plants < 50 PE and max. 4 objects connected

- EUR 2.500 basis for a plant up to 15 PE
- EUR 140 for each additional PE
- EUR 20 per meter collecting system

Experience from Slovakia - The Water Act no. 364/2004 transposes the regulations of the acquis communautaire into legislation of the Slovak Republic. Among others this Act states the obligation for agglomerations < 2.000 PE in which sewage systems are built without appropriate treatment, to perform the treatment not later than 31 December 2015. This Act is followed by the Regulation of the Slovak Republic Government No. 269/2010 Coll. of 25 May 2010 determining the requirements for achieving a good water status. Herein, limit values of pollution parameters consider economically sustainable possibilities of technological solutions of waste water collection and treatment. The values for waste water discharge in agglomerations < 2.000 PE depend on the plant size (up to 20, 50 and 50-2.000 PE).

Experience from Romania - The legal framework for authorization in water management and water protection is represented by the Water Law No. 107/1996 (with subsequent amendments and completions) and the Ministerial Order No. 662/2006. According to these ordinances, the National Administration of Romanian Waters (NARW) and the River Basin Water Directorates (RBWD) are the competent authorities for issuing water management permits and licenses.

Related to licensing and permitting of waste water discharges from small agglomerations below 2.000 PE, NARW can appoint slightly higher or lower limits depending on the local conditions and water bodies status according to the technical legal provisions NTPA 001/2005 and NTPA 002/2005 (annexes of GO 352/2005). These local limits are then defined in water management licenses. The local branches of NARW are monitoring the quality of the discharges from waste water treatment and also the quality of receiving water bodies

4.2 Croatian experience related to planning wastewater treatment in agglomerations below 2.000 PE

European Standards for small scale wastewater treatment: EN 12566 series [8]

Over the past 20 years the European Union has been developing a new standard for small scale wastewater treatment. Croatia has adopted it. All packaged and pre-fabricated treatment systems (kits) sold now have to have EN 12566 certification which currently consists of 7 parts. The EN 12566 standards deal with a range of products including prefabricated septic tanks, soil infiltration systems, packaged and/or site assembled wastewater treatment plants and packaged filtration systems, etc. Some parts are still in preparation whilst others are now considered as finished and published for adoption as National Standards. The status of the individual parts is listed as follows:

- **EN 12566-1:** 2000: Small Wastewater Treatment Systems up to 50 PE – Part 1: Prefabricated Septic Tanks (Published).
- **EN/TR 12566-2:** 2005 Small Wastewater Treatment Systems up to 50 PE - Part 2: Soil infiltration systems (Published).
- **EN 12566-3:** 2005 Small Wastewater Treatment Systems up to 50 PE – Part 3: Packaged and/or site assembled domestic wastewater treatment plants (Published).
- **EN 12566-4:** 2008 Small Wastewater Treatment Systems up to 50 PE – 2007 Part 4: Septic tanks assembled in situ from prefabricated kits (Published).
- **EN12566-5:** 2004 Small Wastewater Treatment Systems up to 50 PE – Part 5: Pre-treated effluent filtration systems (Published).
- **pr EN 12566-6:** 2008 Small Wastewater Treatment Systems up to 50 PE – Part 6: Prefabricated treatment units for septic tank effluent (Draft).

- **pr EN 12566-7:** 2009 Small Wastewater Treatment Systems up to 50 PE – Part 7: Prefabricated tertiary treatment units (Draft).

In Croatia there is no planning approach for waste water management of agglomerations below 2.000 PE on national level although some next steps related to this issue had to be done as a part of water management plans.

There are activities on regional and local level. For example in the Region of Istria there is Istrian Water Protection System a company owned by the cities and municipalities of Istria, Region of Istria and Croatian waters, established for the project "Public sewage system and wastewater treatment for small settlements in the zones of sanitary protection of drinking water sources of Istria" because a very large part of the Region is sensitive area. Many small WWTP from 100 to 500 ES are in plan to be built and some are already built and in function.

5 DIFFERENT APPROACH TO DEFINITION OF POLLUTION SOURCES AND ENVIRONMENTAL IMPACT

5.1 General description of technologies

The results of the survey of small scale waste water treatment in Europe showed that the terminology for similar systems strongly varies. On the other hand, systems reported show differences in design, tackling the systematic allocation. To overcome these facts a functional description covering the main components from waste water production to the discharge to the receiving environment has been chosen.

The general description covers the main groups of reported technologies for small scale waste water treatment in Europe. Hence, the description covers a wide range of application, i.e. the application of the systems presented might vary regionally as well from common application to minor application. Since there are many varieties of design and process layout, the description covers the basic properties that characterize the technology.

5.2 General comments on small scale and decentralised treatment

The differentiations between small scale and full scale treatment or between decentralised and centralised treatment cannot be given sharply and uniformly. Since the related technologies applied are based on common principles, those definitions are merely based on conventions that are related to the regional or national conditions. The definition of „small“ in respect to waste water treatment systems is different in many countries depending on the settlement structures and the infrastructure development in general. The content of this catalogue intends to give an overview on plants of a capacity up to 2.000 PE, (in reference to agglomerations <2,000 PE, 91/271 EEC) therefore the term „small WWTP refers to this threshold.

The coverage of settlements of a single treatment plant gives information on the „character“ of the system in terms of decentralisation:

- „Single (household) system“– single connection and short conveyance distance
- „Cluster system“– small number of connections and medium conveyance distance
- „Centralised system“– high number of connections and high conveyance distances

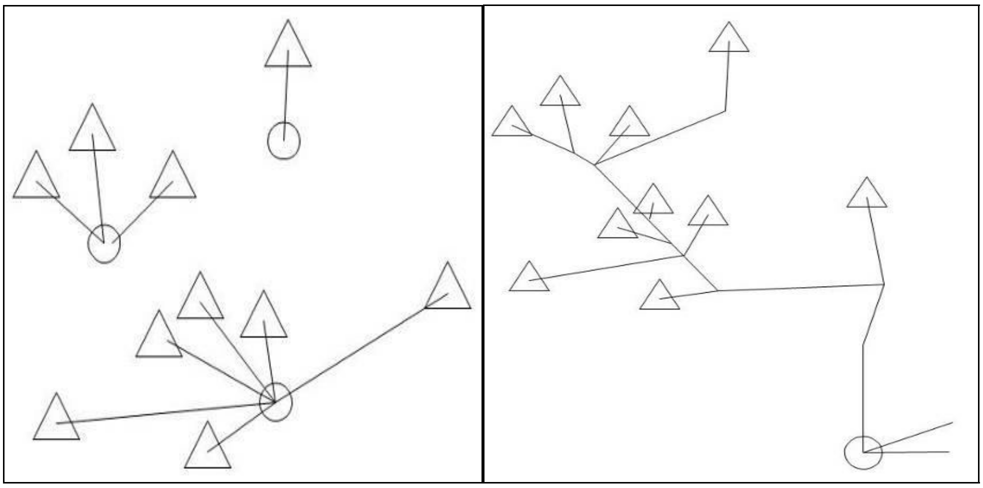


Fig. 11 Single and clustered systems (left; circles: treatment plants; triangles: houses) vs. a (semi) centralised variant for the same settlements on the right side [7]

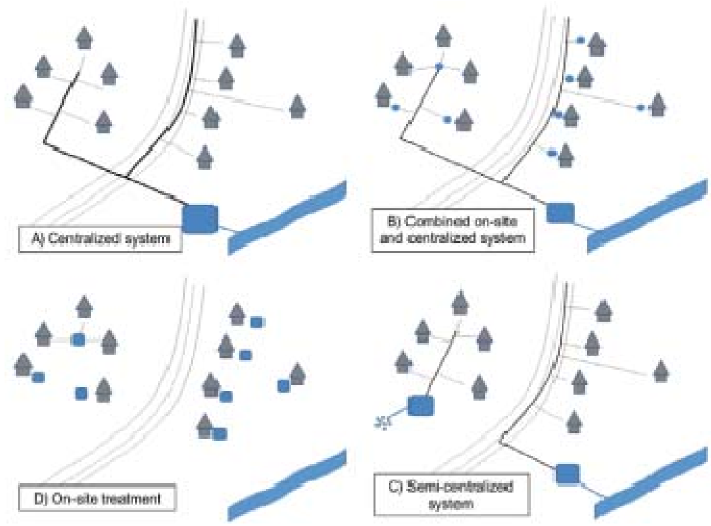


Fig. 12 Technical possibilities how to provide an agglomeration with sanitation and wastewater treatment [7]

Another aspect that may be related to the character of the waste water disposal infrastructure is the organisation of operation and maintenance of the installed systems. In general, for small or decentralized systems the systems owner(s) (e.g. single household or neighbourhood) are in charge of the operation and maintenance. This implies that the systems have to be easy enough to be operated by non-experts, or an adequate training is necessary. Different operation and maintenance models (e.g. maintenance contracts) allow an appropriate operation of more complex systems by outsourcing the services to external experts. In case of ownership and operation of even a small plant (e.g. 1000 PE) by a community, municipality or a public association can be seen as centralised system.

5.3 Characteristics of waste water

Waste water from small settlements is mainly originating from households or small businesses (trade waste water). For households the waste water is generated from different sources and can therefore be separated into the following components:

- Grey water: waste water from kitchen, bathrooms, showers, hand washing facilities and laundry without contact to human excreta.
- Brown water: faeces with flushing water.
- Yellow water: Urine.
- Black water: Brown water and yellow water.

The above made distinction is relevant for a number of decentralized treatment and disposal options where the different streams are treated differently. However, most of the mentioned methods described below are designed to treat the produced waste water as a whole and a differentiation is not necessary in those cases. The main constituents of waste water that are relevant for small systems are the organic compounds (organic carbon) that are given as sum parameters (chemical or biochemical oxygen demand) and the main nutrients (nitrogen and phosphorus). Beside that the content of suspended solids are of interest. Other parameters may be relevant when it comes to enhanced treatment requirements.

6 WASTEWATER TREATMENT AND SLUDGE DISPOSAL WITH REGARD TO THE IMPLEMENTATION OF „ADEQUATE TREATMENT LEVEL“

Depending on the number of households connected to a waste water treatment system, the characteristics change from single household owned systems to semi centralized systems with related sewer systems. However, the main components are the same and are presented in the following figure. There are two basic options: Onsite treatment of waste water (via single, clustered, or centralised systems) or storage and road transport to a remote facility.

Beside the main components, the figure below shows the difference between ways of treatment that are considered to be appropriate (generally secondary or tertiary treatment or transport to remote facilities) and ways that cannot be considered as appropriate but are found in practice.

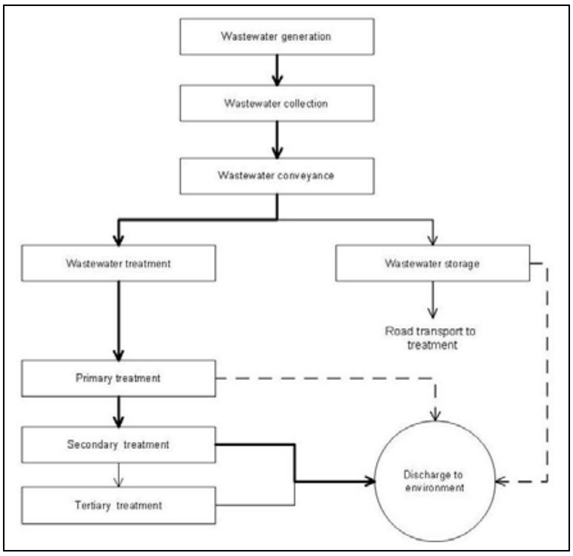


Fig. 13 Flow scheme of appropriate waste water treatment (bold lines) and non- appropriate treatment (dashed lines) relevant for agglomerations <2.000 PE [7]

The listing of systems shown in Tab. 6 is based on the technologies reported in the investigations. Only technologies in use in the EU are contained. The following table gives an overview and lists the technologies according to their appearance in the description. Note the collection of waste water is not described in detail since it is of minor importance in practice. Further, the disposal of sludge from treatment process is not shown. Sewage sludge has to be treated appropriately before use and disposed. In many cases this is achieved by periodic transport to larger centralized facilities.

Tab. 6 List of waste water treatment technologies for agglomerations less below 2.000 PE [7]

Component	Technology	Purpose / Characteristics
A- Collection of waste water in the household or small business		
	A1- Standard in-house piping	Collection of mixed waste water
	A2- Separation system	Collection of separate streams
B- Conveyance of waste water to storage or to treatment facility		
	B1- Gravity sewer line	Conveyance along gradient
	B2- Pressure sewer line	Conveyance against gradient
C1- Storage of waste water for periodic transport to treatment facility		
	C11- Cesspool	Watertight storage
C- Primary treatment for the removal of sand, grit and particulate matter		
	C1- Septic tank	Additional partial removal of dissolved organic contamination
	C2- Settling tank	-
D- Secondary treatment for biological removal of dissolved organic contamination		
Activated sludge systems- Biological treatment using suspended biomass.	D1- Conventional act. sludge	Separate biomass retention step.
	D2- Sequencing Batch Reactor	Single reactor setup.
	D3- Membrane Bio Reactor	Membrane biomass retention.
Fixed film systems- Biological treatment using attached biomass	D4- Trickling filter	Natural aeration of biomass.
	D5- Biological contactor	Mechanical aeration of biomass.
Natural systems – Extensive biological treatment	D6- Constructed Wetlands	Controlled wetland processes.
	D7- Waste water ponds	Controlled surface water processes.
E- Tertiary treatment for achieving further purification of secondary effluent		
Activated sludge systems- Biological treatment using suspended biomass.	D1-D3, D5	Implementation of nitrogen and phosphorus elimination processes in the main stage.
Natural systems – Extensive biological treatment	D6-D7	As additional stage after secondary treatment stage
Hygenisation - Removal of pathogens	E1 - UV- Disinfection	As additional stage after secondary/other tertiary treatment stage
Filtration – Removal of suspended and non-degradable matter	E2- Filter systems	As additional stage after secondary treatment stage
F- Disposal of appropriately treated waste water to the environment or re-use		
Discharge to surface water	F1- Discharge to surface water	Discharge via piping to receiving water body
Discharge to underground	F2- Soil infiltration systems	Infiltration of treated waste water
Re-use and discharge to underground	F3- Irrigation systems	Agricultural application of treated waste water

Single household:

- Application: One treatment plant per household.

- Operation and maintenance (O&M): The household owners are responsible for operation and maintenance.
- Example: A pre-fabricated membrane bioreactor with a capacity of 5 PE situated in the basement of a house.

Clustered households:

- Application: One treatment plant for a number of households.
- Operation and maintenance (O&M): The household owners agree on common efforts for operation and maintenance.
- Example: A vertical flow constructed wetland for 20 houses with a capacity of 100 PE situated at a commonly owned plot in the vicinity of the small agglomeration.

Community:

- Application: One treatment plant for all households in the community (centralized).
- Operation and maintenance (O&M): The community is responsible for operation and maintenance.
- Example: An activated sludge plant for a community with a capacity of 1,500 PE situated in the vicinity of a small river downstream of the community.

Temporary, there is possibility for co-financing from EU funds regional development to solve problems of wastewater treatment for agglomerations below 2.000 PE as part of rural development projects from.

7 DISCUSSION AND CONCLUSION

Information about small scale waste water facilities are usually not part of the regular data management on national level, but this information is necessary for local and regional water management issues.

For the (aquatic) environment, emissions from small scale waste water facilities can significantly influence the water quality elements of a water body. The achievement of the good water ecological status for surface and groundwater, relate the biodiversity of Croatia, will be great challenge. Therefore the UWWTD is strongly linked to the implementation of the Water Framework Directive and to potential measures to be applied.

The assessment of information on small scale treatment shows that the efforts in Central and Eastern European countries, also as in Croatia, are currently focused to comply with the requirements of the UWWTD. Consequently, waste water facilities for agglomerations < 2.000 PE will follow as a next step after having covered the larger agglomerations in the national implementation programs. Next step means the issues related to the WFD as well as on closing gaps in rural areas that still use old sanitation infrastructure (e.g. cesspools) in order to reach the appropriate environmental objectives.

The main small scale waste water treatment technologies implemented in the Member States are cesspools, septic tanks and various activated sludge systems (e.g. conventional systems, sequencing batch reactors (SBR)), followed by the application of fixed bed systems. In some Member States natural systems are used predominantly for waste water treatment in rural areas.

For pre-fabricated systems, EN 12566 is in force and has therefore already been implemented to national standards in most Member States, also as in Croatia. Further technical standards for small scale waste water facilities are only applicable in few Member States.

In the view of an integrated waste water management policy and under consideration of an efficient use of resources, the priorities for implementation and adaptation of waste water treatment facilities should follow a top down approach according to the size of agglomerations and the status of receiving waters.

There is no planning of the sewage systems and waste water treatment plants in agglomerations below 2.000 PE in protected areas in Croatia. Mostly, legislation and standards are prepared for agglomeration larger than 2.000 PE. There are activities on regional and local level (for example in the Region of Istria).

Related to presented analyses, it will be necessary to prepare appropriate measures related to wastewater treatment below 2.000 PE in protected areas. The measures had to be defined to avoid risk of no achieving good water status in protected areas.

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LEACHATE GENERATION IN THE MSW LANDFILL

K. Galbová[1] and I. Škultétyová[2]

Abstract

Waste management in the Slovak Republic is based on the dominant landfill, currently landfilled more than 82% of municipal waste produced. The main environmental impacts of such landfills, containing high amounts of biodegradable organic matter, are caused by emissions of liquid effluents and landfill gas. With no collection and treatment, leachates from landfills pollute groundwater and soils locally, while landfill derived methane contribute to climate change on a global level.

As water plays a key role in landfills, knowledge about its distribution and transport is fundamental for understanding the behavior of the landfill.

Keywords

municipal solid waste landfill, leachate, leachate production, water balance, modeling of landfill hydrology

1 INTRODUCTION

Predicting quantity of leachate is a critical parameter in the design of landfill construction. Created amount of leachate influences the way how to capture, collect and process the leachate.

In resolving that issue, it´s important to design optimal water regime of landfill order to ensure the elimination of potential environmental pollution.

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The hydrometeorological conditions in the area of the landfill and its surroundings are of high importance as they affect the hydrogeological status of the area, leachate production and subsequently the risk of contamination [4].
Extended section of the Zohor Landfill, located 26 km northwest of the Slovakian capital, Bratislava has been in operation since 2011 and its capacity is 240.000 m^3, which represents a total area of 1.42 hectares. Enabled annual amount of stored waste is 200.000 tonnes per year and it’s projected the landfill closure in the year 2018. Construction commenced in 2011, the site received about 200.000 tonnes of municipal solid waste in its first year of operation. This paper summarises the simulation of the water balance of the landfill.

2 QUANTITY OF LEACHATE

Leachate is the percolation of precipitation, surface drainage and irrigation water into the landfill including the biological and chemical reaction of waste being disposed at the landfill. Leachate formation is an indicative of increased moisture content, which is associated with enhancing biodegradation in landfills [2]. Leachate generation can be determined directly by collecting leachate production from landfill site that has leachate collection system.
Generally, water balance of landfill is used to estimate leachate formation. The water balance components include water inflow, water outflow and water store within landfill.

The amount of leachate generated from landfills over long time periods (e.g. years) can be predicted quite well using available water balance models (e.g. HELP [6]). The water balance components are presented in Figure 1.

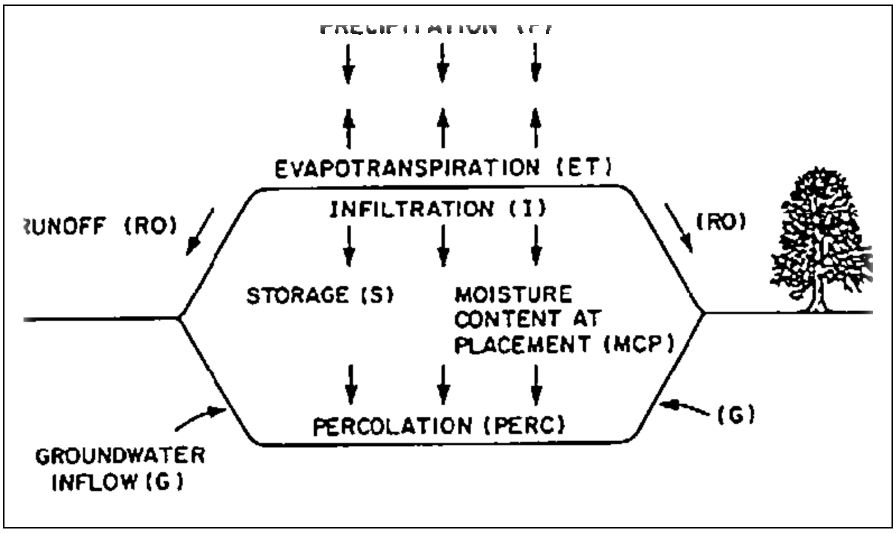


Fig. 1 Water cycle in a MSW landfill

Their quantity is influenced by the intensity of rainfall, the surface runoff, evaporation and the ability to accumulate the landfill body water. They calculate leachate discharge equal to the difference between precipitation and the sum of actual evapotranspiration, runoff, and water storage within the waste body, where by the later one is determined on the concept of field capacity (no water flow until the soil or waste reaches certain water content). However, the variation of the leachate discharge rate over time is much more difficult to describe, since it requires an understanding of the water flow processes inside the landfill [3].
Despite the need for obtaining information on the quantity of leachate from the landfill, they have so far not been sufficiently addressed. Process water balance for the landfill is more

complex than the water balance of natural origin, because in addition to natural conditions affect the quantity water and bio-chemical processes inside a landfill [3].

3 SIMULATION OF THE WATER BALANCES OF THE MSW LANDFILL

The site was proposed as a modern landfill which meets the standard requirements of according to the generally binding regulations. It was designed to receive a mix of non-hazardous commercial and domestic municipal solid waste.

It is situated in a warm climate and the average annual rainfall reached 702 mm. Total precipitations during the growing season are from 400 to 500 mm and in winter season from 200 to 300 mm. Monthly rainfall typically ranges from 30 - 100 mm, with distinctive occasional monsoons in June and July. The average year temperature is of 9.0 to 9.9 °C and an average sunshine duration is of 156.4 hours the solar radiation.
Estimates of leachate generation rates were simulated with varying heights of deposited waste for a certain year of operation of the landfill using the HELP model in conjunction with a simple water balance model. The assessment of leachate generation was for the 1.42 ha.

Hydrologic Evaluation of Landfill Performance Model

Determining water management of landfills need to understand leachate formation, factors influence leachate production, including model application. Hydrologic Evaluation of Landfill Performance (HELP) model is a tool to estimate of water balance for municipal solid waste landfill. Use of HELP model is recommended by the U.S. Environmental Protection Agency (US EPA) and required by most states for evaluating closure design of hazardous landfill non-hazardous waste management facilities.
Model application, the HELP model is classified as quasi-two dimensional because several one-dimensional models (percolation vertically, drainage and surface runoff horizontally) are coupled [1]. The model accepts weather data, soil and design data and uses solution techniques for water balance analysis. Generally, landfill system consists of the various combinations of vegetation, cover soils, waste cells, lateral drainage layers, low permeability barrier soils and synthetic geomembrane liners. The model facilitates rapid estimation of the amounts of runoff, evapotranspiration, drainage, and leachate collection and liner leakage.
HELP model still has limitation of application. For example, the model has limits on the arrangement of layers in the landfill profile. The physical characteristics of landfill are constant over the modelling period.
The model was applied for estimating the leachate generation in landfill and its response on rainfall variation. The HELP model version 3.07 was selected for simulation water balance of open MSW landfill. Significant inputs were collected and assessed for model procedure.

Data collection and analysis

• Weather data was recorded by Meteorological Station Kuchyňa. Seven years (2006 - 2012) daily rainfall, temperature, solar radiation, etc. were collected. The other significant data for HELP model was obtained from the previous experimental data, literature review and available data from model.

• Soil and landfill design data were followed the specific design and operation of open MSW landfill. Using the default data from model was considered.

Modelling procedure

A comprehensive review on HELP model and its application was performed. The procedure followed the HELP model user's guide Version 3 [5].The water balance of MSW landfill was modelled.

Since the model does not include the climatic data of the considered site Zohor were manually inserted into the model seven annual data on rainfall, temperature and solar radiation.

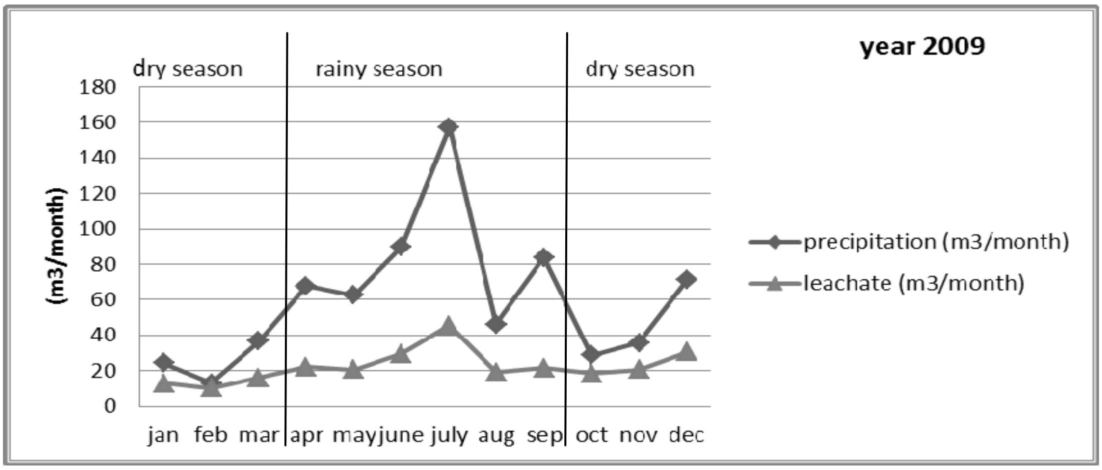
The simulation was composed profile of MSW landfills. Profile is composed of six layers, wherein the first layer is municipal waste, the amount of which varied during the seven years of operation of the landfill.

During the first year the height of the waste deposited reached 2.41 meters, in the second year, the amount of waste 4.82 m, in the third year of 7.23 meters, in the fourth year of 9.68 meters, the fifth year of 12.05 meters, in the sixth in the seventh year of 14.46 m and 16.87 m. The second to fourth layer is a drainage system and the fifth and sixth layer consists of sealing landfills.

In the simulations were used climatic data for 7-years from the meteorological station military airport Kuchyňa.

4 RESULTS

Leachate generation is not constant and it depends on the initial moisture content, decomposition of solid waste, and the influence of climate. Graph 1 presents the relationship between rainfall and cumulative leachate generation from landfill during the year 2009. This figure presents rainy season and dry season.

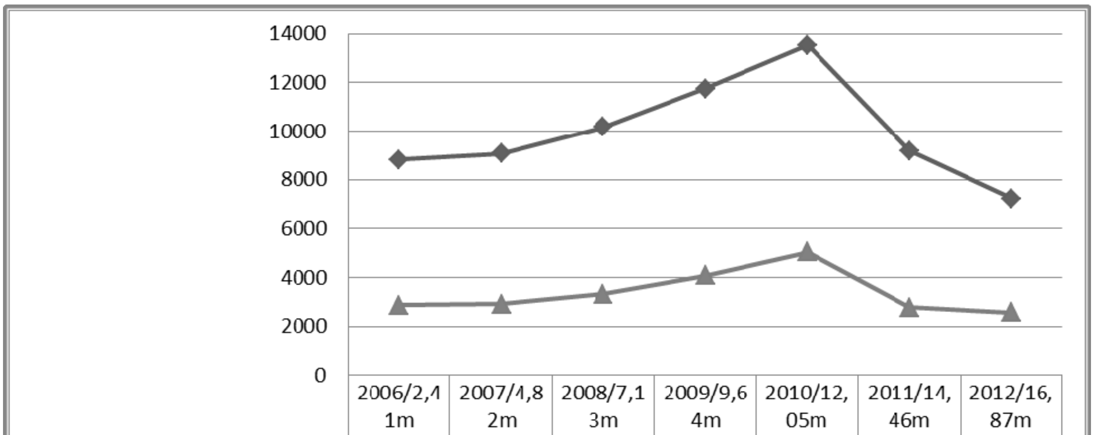


Graph 1 Values of precipitation and leachate for the fourth year of operation of the landfill.

Rainy season was started from June to mid-September 2009. High amount of leachate was generated which mainly based on initial moisture content of MSW, decomposition of waste and precipitation in this period. The leachate from landfill was significantly increased on July 2009 which had high intensive rainfall with long duration. The rainfall was rapidly infiltrated into landfill.

During the dry period was very less or no rainfall. Thus, in this period, the leachate was produced in small amount. The cumulative leachate generation from landfill was slowly increased.

Water management at landfill sites is an important issue. The open landfill simulation showed that the highest cumulative leachate generation during monsoon and leachate ceased out during the dry period due to heavy loss of moisture by evaporation.
After input all data, HELP model simulated the results such as daily, monthly and annual output of precipitation, runoff, evapotranspiration, lateral drainage collected and percolation/leakage through layer.
Graph 2 shows the output of the simulation based on the monthly average data of seven years (2006-2012). The model output illustrated that the cumulative leachate generation and the evaporation were 66 % of annual rainfall, respectively.



	2006/2,41m	2007/4,82m	2008/7,13m	2009/9,64m	2010/12,05m	2011/14,46m	2012/16,87m
precipitation (m3/month)	8828.14	9083.74	10178.56	11743.4	13528.34	9184.56	7229.22
leachate (m3/month)	2865.89	2908.18	3330.34	4082.7	5045.03	2775.57	2571.27

Graph 2 Production of leachate during operating years

The change in water storage in landfill was varied depended upon the infiltrating rain, evaporation and leachate generation components. Use the HELP model was performed complex simulation behaviour of the landfills. The results during the considered period provide the estimates of leachate and their variability throughout the year.

5 CONCLUSIONS

In Slovakia the open landfill approach is the most predominant waste disposal option creating considerable environmental problems. With the accelerated generation of waste caused by an ever increasing population, urbanisation and industrialisation, the problem has become one of the primary urban environmental issues.
This study aims to investigate water management in the open landfill. Water management consisted of storage and evaporation of leachate. The application of Hydrologic Evaluation of Landfill Performance (HELP) model for water management was determined.

The results of simulation water balance of open MSW landfill from HELP model indicate that the leachate generation was the main component of water balance (around 33.79 % of total precipitation) which needs to manage for open MSW landfill. Simulation of the leachate production can be predicted quantities for few decades.

Acknowledgments
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MOTION OF SUSPENDED PARTICLES IN VORTEX SEPARATOR

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Abstract
Vortex separators are designed for gravitational removal of suspensions from storm waste water. In this type of devices centrifugal force is applied to enhance the process of sedimentation. Vortex separators, like all objects for water and waste water treatment, must be designed with utmost care. Therefore, they can be classified as devices requiring additional verification of design calculus. As empirical experiments are expensive, time-consuming and need a prototype to be constructed, convenient numerical methods are chosen. Motion of suspended particles is described by a system of differential equations. The equations are solved numerically leading to creation of a computer program. The software can be used to calculate and visualize trajectories of specific particles and utilized to design vortex separators.

Keywords
Centrifugal force, particle trajectory, sedimentation, suspended particle, vortex separator, waste water treatment

1 INTRODUCTION

When designing technical objects, it is convenient to use computational methods as simple as formally possible and physically justified. Mathematical relations describing considered processes or phenomena in an algebraic form are especially desired in the design process. However, this formal simplicity is usually obtained at the cost of lower accuracy of the solution. That is why, it is always wise to have more precise, but complicated, methods at

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ones disposal. Usually, they come as a set of differential equations and are solved by numerical means. Such methods can be used to perform simulations of considered systems for various external conditions and states. Most often, simulations yield a full set of information needed to solve the analyzed problem. However, to design a completely new object, one needs to find a method for solving inverse problems.

2 MATERIAL AND METHODS

The paper involves issues related to design and operation of vortex separators. These devices are designed for gravitational removal of suspensions from storm waste water. Their basic characteristic is application of the centrifugal force to support the process of sedimentation. This force elongates particle detention time within the separator chamber improving its separation from the carrier liquid.

Methods for designing vortex separators presented in literature are questionable. Main doubts concerning improper description of fluid velocity field and balance of forces acting on a characteristic suspended particle have been discussed in works [1] and [5].

Force balance is employed in the structural method (determination of particle trajectory) which is the basic tool for describing operation of vortex separators. Particle trajectories are expressed by radius vectors $\mathbf{r_p}(t)$. Each trajectory is related to particle velocity $\mathbf{v_p}$ by the formula:

$$\frac{d\mathbf{r_p}}{dt} = \mathbf{v_p} \tag{1}$$

Description of particle velocity by Newton's second law yields a set of three equations [2], [6] and [10]:

- for radial direction:

$$\left(\rho_p + \alpha_s \rho\right) V_p \frac{dv_r}{dt} = \left(\rho_c - \rho\right) V_p \frac{u_t^2}{r} - C_D F_p \frac{\rho\left(v_r - u_r\right)^2}{2} \tag{2}$$

- for tangential direction:

$$\left(\rho_p + \alpha_s \rho\right) V_p \frac{dv_t}{dt} = C_D F_p \frac{\rho\left(v_t - u_t\right)^2}{2} \tag{3}$$

- for vertical direction:

$$\left(\rho_p + \alpha_s \rho\right) V_p \frac{dv_z}{dt} = \left(\rho_p - \rho\right) V_p g - C_D F_p \frac{\rho v_z^2}{2} \tag{4}$$

where: v_r, v_t, v_z – radial, tangential and vertical particle velocity component, V_p – particle volume, F_p – particle active cross-section, C_D – drag coefficient, g – gravitational acceleration, ρ_p, ρ – particle and liquid density, α_s – associated mass coefficient, t – time.

Velocity field of liquid flowing through vortex separator is described by a simplified method in order to get relatively simple, but accurate, relations [2], [5], [8]:

$$u_r = -\frac{Q}{2\pi H_e r} \tag{5}$$

$$u_t = Br^{-0.65} \tag{6}$$

where: u_r, u_t – radial and tangential liquid velocity component, Q – discharge, H_e – average active liquid depth in separator, r – radius, B – semiempirical multiplier.
According to laboratory research [2], [8] value of H_e can be calculated from the formula:

$$H_e = 0.5\left(H + h + d_{in}\right) \tag{7}$$

where: H, h – specific liquid depths in vortex separator, d_{in} – inlet diameter (Fig. 1).

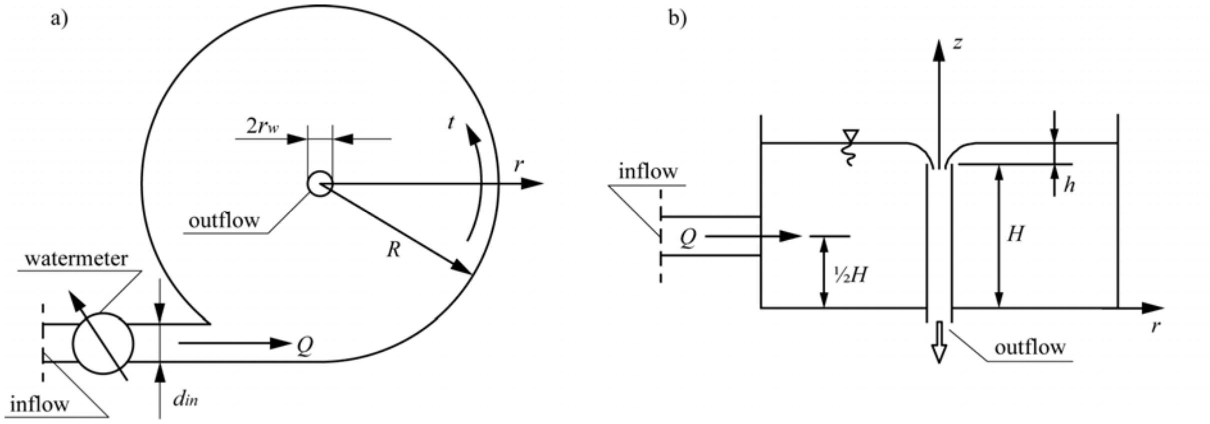

Fig. 1 Schematics and dimensions of vortex separator

Coefficient B is calculated from balance of energy flux delivered to the device by inlet pipe and energy flux dissipated by rotational motion of liquid [7], [8], [9]:

$$B = 4.63Q\left[Hd_{in}^{\ 4}\left(r_w^{-0.95} - 1.41R^{-0.95}\right)\right]^{-1/3} \tag{8}$$

where: r_w – outlet radius, R – separator outer radius (Fig. 1).

Thickness of water overfalling the outlet pipe orifice h can be calculated from the formula:

$$Q = \frac{4}{3}\pi\mu_p r_w\sqrt{2g}h^{3/2} \tag{9}$$

Value of overfall discharge coefficient μ_p is the same as for the "morning glory" overfall [3].

3 RESULTS

The presented set of Eqs. (1)-(9) can be used to describe vortex separator operation. Low acceleration of suspended particles allows to exclude the inertial force from relations (2) and (3). As a result, Eq. (2) takes the form:

$$v_r(r) = \frac{dr_p}{dt} = u_r + \frac{v_{so}B}{\sqrt{g}r^{1.15}} \tag{10}$$

where: v_{so} – free sedimentation velocity of a particle of diameter d_p [6], [10]:

$$v_{so} = \left[\frac{4(\rho_p - \rho) g d_p}{3C_D \rho} \right]^{1/2} \tag{11}$$

After transforming Eq. (3) one obtains:

$$v_\varphi(r) = r_p \frac{d\varphi_p}{dt} = u_\varphi(r) \tag{12}$$

where: v_φ – particle angular velocity, φ_p – particle angular coordinate, u_φ – fluid angular velocity. Division of Eqs. (11) and (12) by sides yields the equation describing shape of particle trajectory:

$$\frac{dr_p}{d\varphi_p} = -\frac{Q}{2\pi H_e B} r_p^{\ 0.65} + \frac{v_{so}}{\sqrt{g}} r_p^{\ 0.5} \tag{13}$$

Computational time of particle motion needs to be correlated with time of particle sedimentation t_{so} given by the formula:

$$t_{so} = \frac{H}{v_{so}} \tag{14}$$

Time of particle motion on a horizontal plane t_p (between inlet and outlet) can be easily calculated from Eq. (12) using Eq. (6). Single integration with initial condition $t_p = 0$ when $r_p = R$ results in the equation:

$$t_p = \frac{\varphi_p r_p^{\ 1.65}}{B} \tag{15}$$

Calculations are done in a number of steps:

- for subsequent time intervals particle coordinates and actual time of particle motion t_p are determined from Eq. (13);
- if $t_p < t_{so}$ and $r_p > r_w$ calculations are continued;
- if $t_p = t_{so}$ and $r_p > r_w$ calculations are stopped – particle settled on separator bottom.
- if $t_p > t_{so}$ and $r_p = r_w$ calculations are stopped – particle flew out from separator.

4 DISCUSSION

Sample calculations were performed for the data set: R = 0.50 m, H = 1.00 m, r_w = 0.05 m, d_{in} = 0.10 m, v_{so} = 0.02 m/s and for three values of discharge (Tab. 1). The highest discharge

Q_1 resulted in particle motion time shorter than particle sedimentation time ($t_{p1} < t_{so}$). This means that characteristic suspended particle was removed from the separator (its trajectory is marked by a solid line in Fig. 2). For the smaller discharge Q_2 particle motion time was still shorter than particle sedimentation time ($t_{p2} < t_{so}$). Even though motion time was distinctively longer than in case of Q_1, particle was again removed from the separator (dashed line). Finally, the lowest discharge Q_3 yielded particle motion time longer than particle sedimentation time ($t_{p3} > t_{so}$). In this case particle settled on the separator bottom (dotted line, sedimentation point PS).

Tab. 1 Times of particle motion and sedimentation for different discharges

i	Discharge Q_i [m^3/s]	Time of particle motion t_{pi} [s]	Time of particle sedimentation t_{so} [s]
1.	0.05	8.6	50
2.	0.01	43	50
3.	0.005	86	50

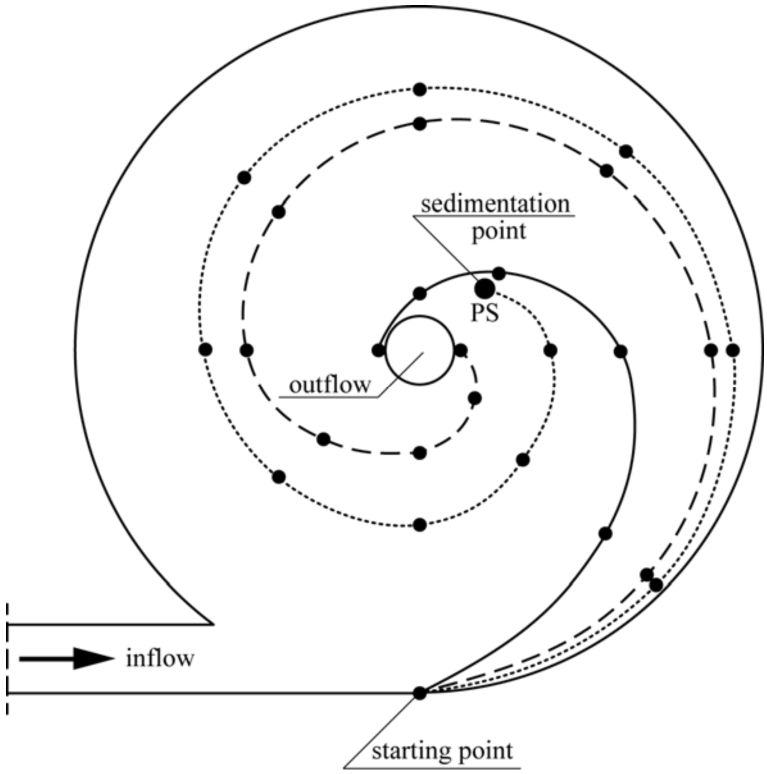

Fig. 2 Sample trajectories of characteristic suspended particle

While describing motion of suspended particles one needs to take into account the sign of derivative of Eq. (13). Right side of the Eq. includes a sum of two terms. The first term (negative one) describes the stress force $\mathbf{F_N}$ that directs particle towards the outlet, while the second (positive one) – the centrifugal force $\mathbf{F_0}$ reduced by the transversal drift. When the stress force is bigger than the centrifugal force particle is directed towards separator axis (negative derivative). When modulus of derivative decreases, influence of the centrifugal force increases. When derivative equals zero, particle reaches a balance point – its horizontal motion ceases and particle falls to the bottom. On the other hand, a positive derivative in Eq. (13) means that the centrifugal force is stronger than the stress force and directs particle towards the outside wall (separator acts like a cyclone).

5 CONCLUSION

The key element of every method for designing mechanical devices for suspension removal is proper description of trajectories of characteristic particles. The paper presents a set of equations allowing to determine particle trajectories in vortex separator. Simplification of these equations yielded relations convenient for calculations and better understanding of the sedimentation phenomena.

Acknowledgements

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ENERGY SAVING POSSIBILITIES IN WATER SUPPLY PUMPING SYSTEMS

Ivan Halkijević [1], Živko Vuković [2] and Marin Kuspilić [3]

Abstract

When the water is pumped through a centrifugal pump the most of the consumed electric energy is spent on reaching the pressure values required by the system. Since the maximum demand occurs in a relatively short period of time, for most of the time water supply system with high demand variability will be characterized with a pressure higher than required. Considering the usual centrifugal head/flow characteristics of the pump, low demand results with high pressure values and thus unnecessary energy consumption. Different demand/pressure relationships are usually considered by changing the rotational speed of the pump. Recently the most popular type of variable speed control became the variable frequency drive. The usual way to regulate the pressure in a water supply system with speed controlled pumps is with constant or proportional pressure values. This paper deals with the comparison of energy consumption when constant pressure or proportional pressure control is used. The possibility for energy savings are examined using the numerical model of pumped water supply system of the city Velika Gorica in Croatia.

Keywords

Energy savings, pumped water supply system, pump speed control

1 INTRODUCTION

Increasing the pressure in water supply systems is usually performed by applying centrifugal pumps. The operating principle of the centrifugal pump is the conversion of the electric energy into the mechanical energy. Most of the available mechanical energy is manifested as

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the kinetic energy (rotational energy) by the rotation of the pump shaft. Increase in water pressure is the result by the conversion of the kinetic energy into the potential energy of the mass of water.

For closed water supply systems the sufficient pressure is realized by using pumps. Closed water supply systems are systems without a storage water tank that regulate the system pressure. These systems are used in locations where storage tank is either infeasible or undesirable or large hydropneumatic tanks are impractical, [14].

The electric energy consumed by the pump is spent on reaching the pressure values required by the system. The critical point regarding the required pressure is realized with the appearance of maximum demand. Since the maximum demand occurs in a relatively short period of time, for most of the time water supply system with high demand variability will be characterized with a pressure higher than required causing the energy consumption to be higher than necessary.

The pump power consumption, P [kW], is defined as:

$$P = \frac{9.81 \cdot Q \cdot H}{\eta} \tag{1}$$

where Q [m^3/s] is flow rate, H [m] is head and η [1] is efficiency. Graphically, power is represented by the area under the rectangle defined by the operating point on a pump performance chart.

The amount of energy, E [kWh], used by the pump is proportional to the power and the operating time, t [h], of the pump:

$$E = P \cdot t \tag{2}$$

In order to reduce the energy consumption it is necessary to reduce either the power or the operating time of the pump. In the case of a closed water supply system it is not possible to reduce the time of operation since the pressure in the system is required during the entire day. The only parameter that can be affected is the power of the pump.

The overall efficiency of a centrifugal pump is the ratio of the output (water) power to the input (shaft) power and is depending upon a particular pump design, including impeller design, volute design and hydraulic losses. For a given pump the efficiency depends on the operating performance and is affected by a different flow rate and a different head. Changes in the flow and the head of the pump are related to each other by the pump QH-curve and directly influence the energy consumption.

When selecting a pump for a given application, it is important to choose the one which has the duty point in the high-efficiency area of the pump. Otherwise, the power consumption of the pump is unnecessarily high. Sometimes it is not possible to select a pump that fits the designed duty point because the demand of the system changes or the system curve changes over time.

2 CONTROLLING PUMP PERFORMANCE

Many water supply systems require a variation of flow demand or pressure and either the system curve or the pump curve must be changed to get a different operating (head/flow) point. Since the pump is usually oversized and is operating inefficiently for other duties then maximum, it is necessary to adjust the pump performance so that changed system requirements are met.

The most common methods of changing pump performance, Figure 1, are [9]:

· Throttle control – the pump runs continuously and a valve in the pump discharge line is fully open or partially closed to adjust the flow to the required value
· Bypass control – the pump runs continuously and a valve in the bypass line attached to the outlet can be used to adjust the pump performance
· Impeller trimming – the pump performance will change by modifying the impeller diameters in the pump meaning
· Speed control – the pump performance parameters (flow rate, head and power) will change with the varying impeller speeds.

Choosing a method of adjusting the pump performance is based on an evaluation of the initial investment together with the operating costs of the pump. Compared to the other methods, which can be carried out during operation, modifying the impeller diameter has to be done in advance, before the pump is installed.

Traditional control methods – throttling valves or bypass lines – waste energy and frequently increase operating and maintenance costs, because valves reduce the flow, but not the energy consumed by the pumps.

The variable speed control method is very efficient way of adjusting pump performance exposed to variable flow rates and pressures. This is because the power consumption reduces as the pump’s speed reduces. Compared to the other methods, the speed control method also makes it possible to extend the performance range of the pump above the nominal speed level of the pump.

The most commonly used device for pump speed variation (reduction) is variable speed drive (VSD) [10], [13], [1]). With the introduction of pump with integrated VSD, it became possible to operate the pump at different impeller speeds, which made it possible to realize other relations between flow and head and between flow and power.

By far the most popular type of VSD is the variable frequency drive (VFD). VFDs are a type of adjustable speed drives used to control the motor (rotational or impeller) speed of an alternating current electric motor by adjusting the frequency and voltage applied to the motor. During the last decade, VFDs have become increasingly popular for the pump characteristics control [5].

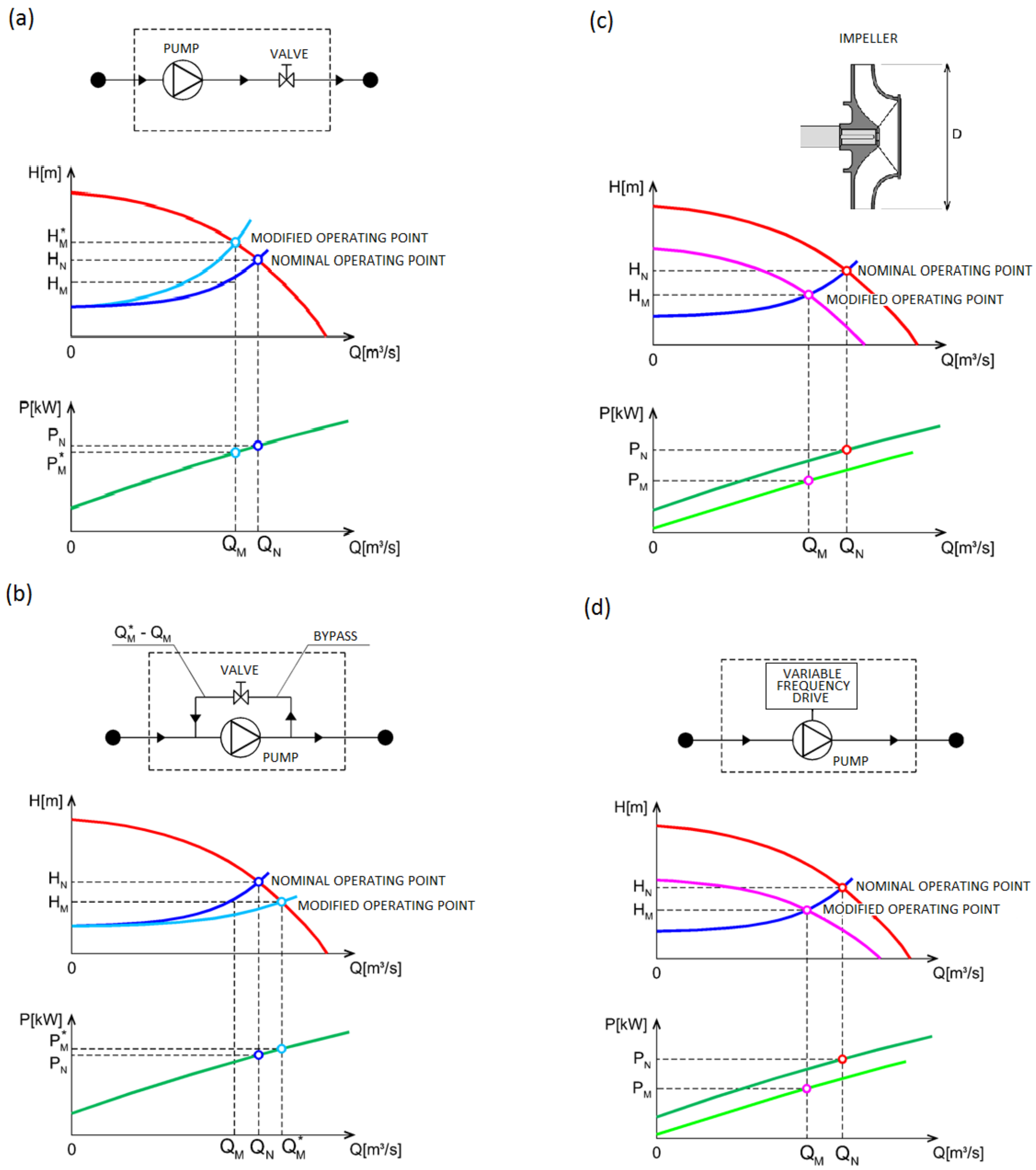

Fig. 1 Pump control methods [12]
(a) Throttle control; (b) Bypass control; (c) Impeller trimming; (d) Speed control

The use of VFD control offers several advantages [7], [6]. The most significant benefit is the potential to reduce electrical energy consumption and demand from motor – driven processes [2]. VFD also has the potential to reduce system maintenance and related costs. Control with a VFD affords the capability to "soft start" a motor, which means that motor can be brought up to its running speed slowly rather than abruptly starting and stopping. Soft starting motor results in less mechanical stress on the equipment and, over time, less maintenance is required. Similarly, running the motor at lower speeds extends the lifetime of other equipment components, including shafts and bearings.

As mentioned earlier, the best possible way of adapting the performance of a centrifugal pump is by means of speed control of the pump. Figure 2 shows an example of the performance curves of a speed-controlled pump in an open and a closed system. The first set of curves shows the QH-curves and the second one shows the corresponding power consumption curves.

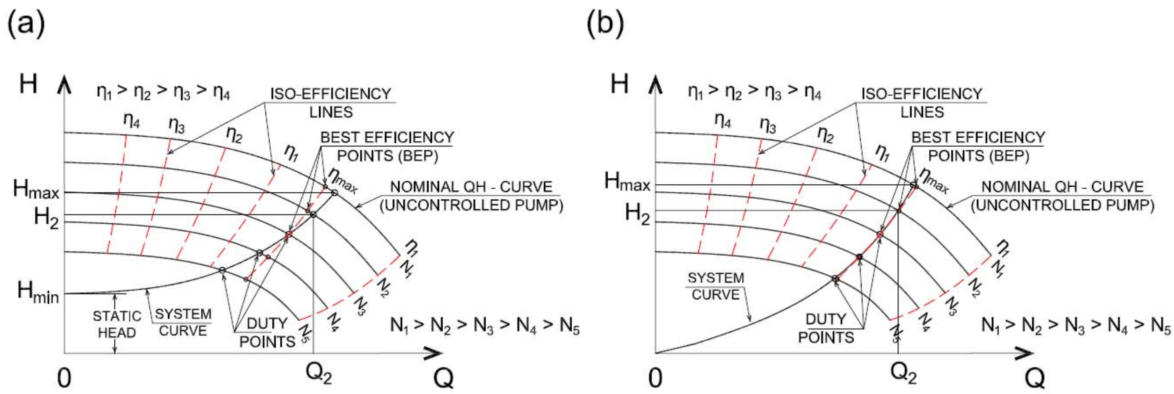

Fig. 2 System characteristic curve together with the pump performance curves for a speed-controlled pump. The units are arbitrary
(a) Open water supply system; (b) Closed water supply system

As it is indicated in the diagram, it is possible to point out a specific duty point and find out at which speed can the duty point be reached and at which power consumption level. Points of equal efficiency on the curves for different speeds are jointed to make the iso-efficiency lines, Figure 2.

In an open system, which is a pump system with a static head, the system curve does not follow the curve of constant efficiency. Instead, it intersects them. This means that the pump efficiency changes along with the speed of the pump. This is quite the contrary to a pump system without a static head, it is a closed system, in which the pump efficiency remains fairly constant with changing speed.

In a system where static head is high the operating point for the pump moves relative to the lines of constant pump efficiency when the speed is changed. The reduction in flow is no longer proportional to speed. A small turn down in speed could give a big reduction in flow rate and pump efficiency, which could result in the pump operating in a region where it could be damaged if it ran for an extended period of time even at the lower speed. The drop in pump efficiency during speed reduction in a system with static head, reduces the economic benefits of variable speed control.

Reducing the speed in the friction loss system moves the intersection point on the system curve along a line of constant efficiency. The operating point of the pump, relative to its best efficiency point, remains constant and the pump continues to operate in its ideal region. The affinity laws are obeyed which means that there is a substantial reduction in power absorbed accompanying the reduction in flow and head, making variable speed the ideal control method for systems with friction loss.

3 SPEED-CONTROLLED PUMP SOLUTIONS

The usual regulation solutions of the pressure in the water supply systems with speed-controlled pumps are [2], [5], [11]:
1. Constant pressure control – water supply systems with constant pressure,
2. Proportional pressure control – water supply systems with pressure proportional to the flow.

3.1. Constant pressure control

With constant pressure regulation set pressure, H_{set}, occurs at the beginning of the pressure pipe, right at the outlet of the pump. The set pressure must provide the sufficient pressure for the critical (authoritative) consumer when maximum demand occurs. Therefore, the pressure in the pumping system, except for the maximum demand, will be higher than required. This excess of pressure will be more conspicuous with higher flow resistance. Constant pressure control is recommended if the flow resistance in the water supply system is low.

Figure 3 shows the performance curves of a speed-controlled pump delivering constant pressure to the system. The supply pressure is constant and independent of the flow in the whole range of $0 – Q_{max}$. *A1 – A4* are different duty points.

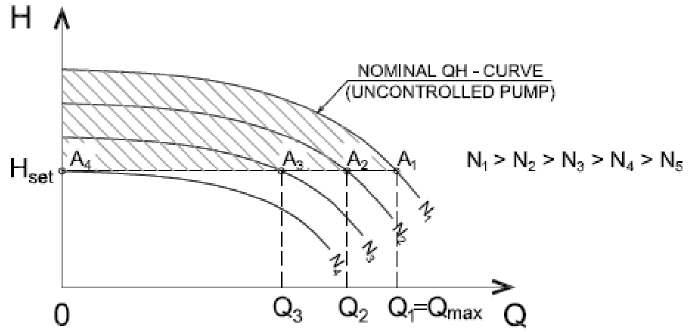

Fig. 3 Constant pressure control [2], [5].

The speed of the pump is reduced from N_1 to N_4 in order to ensure that the required head is $H_1 = H_2 = H_3 = H_4 = H_{set}$.

The shaded area on Figure 3 represents the decrease in head compared to an uncontrolled pump.

3.2. Proportional pressure control

If the resistance in the water supply systems is not negligible, proportional pressure control is recommended. This pressure control also maintains the constant pressure in the system, but not at the beginning thus at the end of pressure pipe, where critical consumer is defined.

Figure 4 shows the performance curves of a speed-controlled pump delivering proportional pressure to the system. The pressure increases when the flow increases and vice versa. *A1 – A5* are different duty points.

Different slopes of the proportional system control curves can be selected to fit the pump to overcome the resistance in the system, in which it is installed.

The system control curve between two operating points H_{max} and H_{min} is usually defined with a straight line, figure 4(a). H_{max} is the design duty head of the pump and H_{min} is the head at minimum flow which is set. However, the system control curve can be modified as a quadratic curve between H_{max} and H_{min}, figure 4(b).

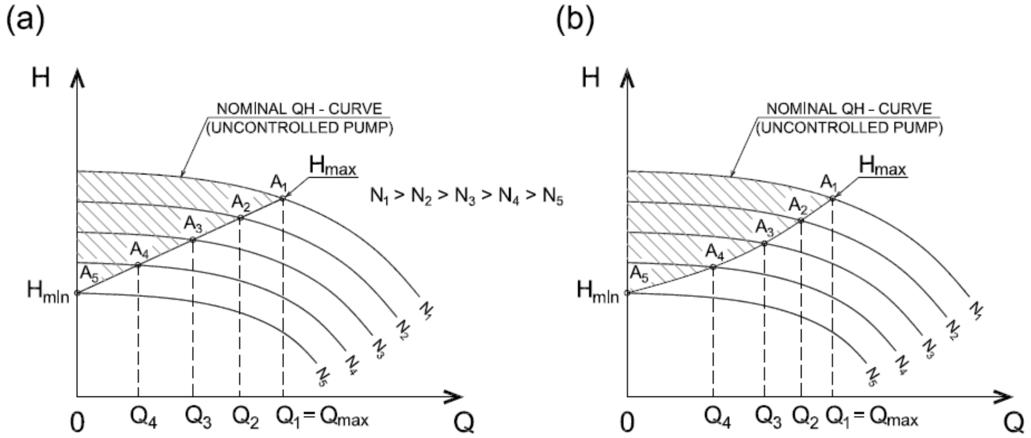

Fig. 4 Proportional pressure control, [2], [5].
(a) linear pressure control; (b) quadratic pressure control

The speed of the pump is reduced from N_1 to N_5 in order to ensure that the required head is $H_1 > H_2 > H_3 > H_4 > H_5$.

The shaded area on figure 4 represents the decrease in head compared to an uncontrolled pump.

4 ENERGY COST SAVINGS

The first step in calculating the potential energy savings is determining the load requirements for the system.

Many water supply pumping systems require a variation of flow demand or pressure. A helpful way to show the flow demand is to use a duration diagram [13]. A duration diagram shows how many hours during a year is a given flow rate needed.

The water supply system must be able to deliver the peak flow, but, from an economic point of view, it is also important to know at which flow rates will the system operate most of the time.

The second step is defining the energy cost savings by comparing energy costs of traditional control methods (e. g. throttling valves or bypass lines) and the energy costs of variable speed control method. This calculation should be done at each specific flow rate from the duration diagram.

As previously mentioned, the typical method for flow control is the use of a throttling valve. Partially closing the valve adds another resistance in the system, raising the system losses and the system curve. The flow rate will now be determined by the point B where the new system curve crosses the pump curve, Figure 5.

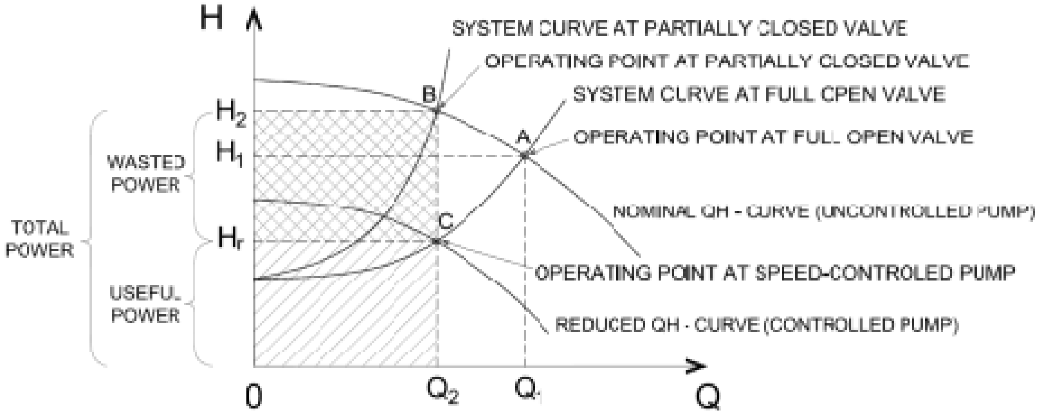

Fig. 5 Effect on system curve with throttling valve and speed-controlled pump [4], [8].

Suppose that we have estimated a new operating condition as Q_2 and reduced head, H_r. In fact, what we actually want is to operate at point *C*. To achieve this we need to reduce pump speed.

In the first case, using throttling valve, the amount of energy the system consumes is proportional to the flow rate, Q_2, and the head, H_2.

In the second case, using speed-controlled pump, the amount of energy the system consumes is proportional to the reduced head, H_r, and flow rate, Q_2.

The difference:

$$Q_2H_2 - Q_2H_r = Q_2\,(H_2 - H_r) = Q_2\,\Delta H \tag{3}$$

represents the energy cost saving of the pumping system. In other words, this difference represents wasted power.

To determine the total cost savings of operating the pump during a year, the running cost savings at each operating condition during a year must be calculated and summated.

Consequently, there is an opportunity to achieve energy cost savings by using the variable speed control method which reduces the power to drive the pump during periods of reduced demand.

The possible savings are evaluated using numerical model of Velika Gorica city’s water supply system

5 NUMERICAL EXAMPLE

The effects of variable speed control are theoretically examined using a numerical model of Velika Gorica city’s water supply system. The total length of the water supply network is about 500.0 [km] supplying the total population of 42.500 and several industrial facilities throughout three municipalities. The system is supplied through the pumping station CS 2 with the intake capacity of 200.0 [l/s]. Two additional pumping stations are interpolated in the system to ensure the required head for distant parts of the system.

This analysis was performed only for the central part of the system in which the pressure is controlled by the main pumping station CS 2. This part of the system has the characteristics of a closed water supply system with a static head around 10.0 [m]. Two pumping stations for distant parts of the system were not analyzed.

The installed pump in the pumping station CS 2 has power input of 190.0 [kW] and operating point Q/H = 200.0 [l/s] / 70.0 [m]. These characteristics largely exceed maximum hourly demand that does not have significant seasonal variation.

The numerical model was designed using the computer program EPANET, version 2.00.12. The analyses were carried out on the calibrated model. Measured data included 80 discharge gauge points and 65 pressure gauge points in total.

Three scenarios were analyzed. The first scenario analyses the uncontrolled pump where the pump adapts to the system demand requirements, Figure 6.

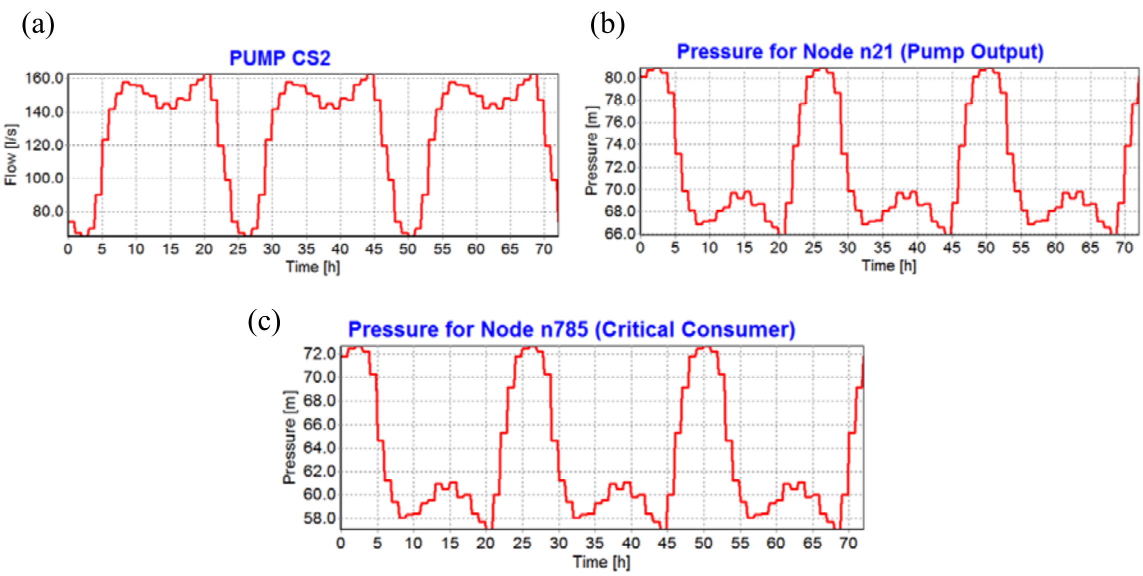

Fig. 6 Uncontrolled pump scenario
(a) Pump flow; (b) Pump output pressure; (c) Critical consumer pressure

Uncontrolled pump operation analysis is characterized with the average pressure in the system ranging from 6.5 to 7.5 [bar], total efficiency ($\eta_{pump} \cdot \eta_{motor}$) of 65.5 [%] and the operating cost of 326.8 [EUR/d] for the 100.0 [%] of time in operation.

Second and third scenario assumes variable speed controlled pump. Considering the required fire-fighting pressure (2.5 [bar]), a minimum pressure value of 3.0 [bar] for the minimum demand and 3.5 [bar] for the maximum demand would secure the need for regular supply of the critical consumer as well as the fire-fighting conditions.

Second scenario assumes constant pressure control for critical consumer. Since the critical consumer requires a pressure of 3.5 [bar] for maximum demand, this pressure value is considered during the whole simulation time, Figure 7.

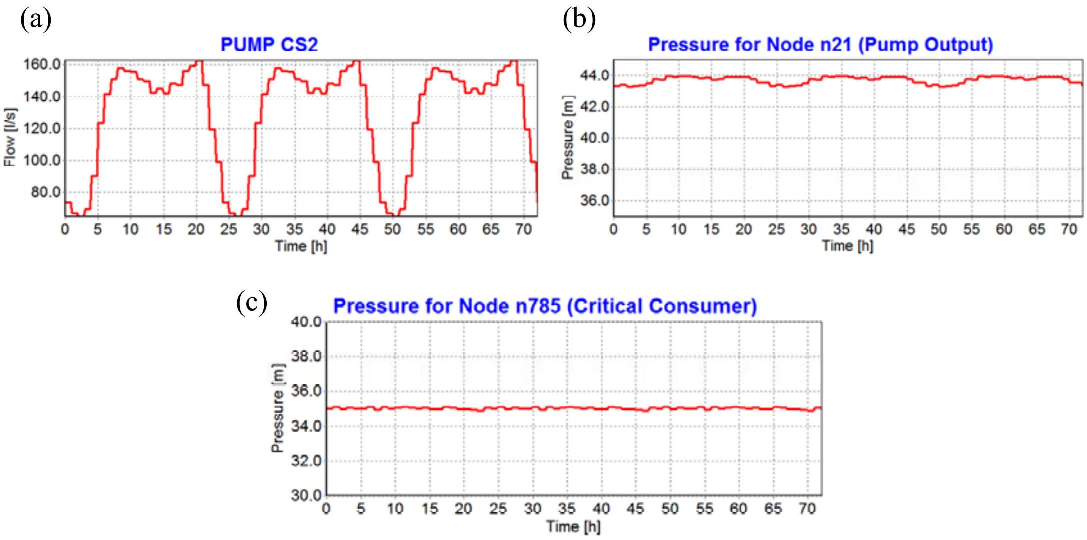

Fig. 7 Constant pressure control scenario
(a) Pump flow; (b) Pump output pressure; (c) Critical consumer pressure

Variable speed pump with constant pressure control is characterized with the average system pressure of 4.0 [bar], total efficiency ($\eta_{pump} \cdot \eta_{motor} \cdot \eta_{VFD\ drive}$) of 57.0 [%] and the operating cost of 268.9 [EUR/d] for the 100.0 [%] of time in operation.

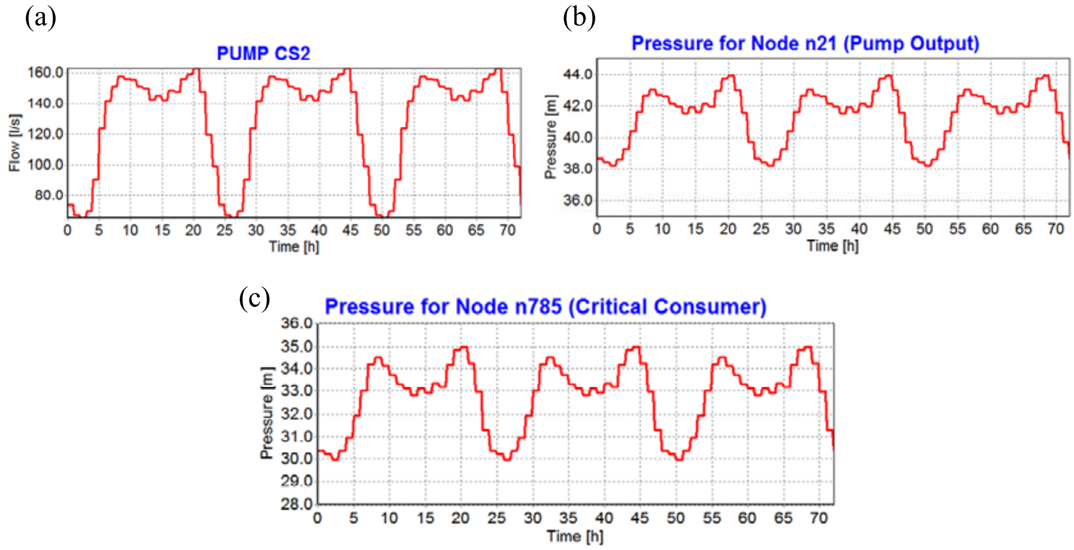

Fig. 8 Proportional pressure control scenario
(a) Pump flow; (b) Pump output pressure; (c) Critical consumer pressure

Introducing proportional pressure control with a pressure ranging from 3.0 to 3.5 [bar], whereby the higher pressure is realized with increased demand, results with operating cost of 231.2 [EUR/d], average system pressure of 3.6 [bar], total efficiency ($\eta_{pump} \cdot \eta_{motor} \cdot \eta_{VFD\ drive}$) of 61.1 [%] and 100.0 [%] of time in operation.

6 CONCLUSION

Commonly used methods for pump control in water supply systems include throttle control, bypass control, impeller trimming and pump speed control. Throttling valves are often used to regulate flow or pressure in water supply pumping systems. However, flow control with throttling sacrifices energy efficiency and results in unnecessary costs.

Variable speed drives provide an economically sound and operationally effective solution for flow or pressure control and reduced power consumption, as well as water loss [3]. The usual pressure regulation solutions in the water supply systems with speed controlled pumps are constant pressure control and proportional pressure control.

The comparison between the constant pressure control and proportional pressure control for the system with high head losses and small static head, indicates that proportional pressure control results with lower running costs of the pump then constant pressure control does.

To determine which pump control solution is most cost effective engineer needs to perform a life-cycle cost analysis for a given system.

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SEWAGE SLUDGE TREATMENT AND REUSE

I. Mahríková.[1]

Abstract

Sludge treatment is the last step of waste water treatment before it´s reuse. Sewage sludge is a waste product, which comes into being in the process of wastewater treatment and so underlies under Waste Low. Sludge often consist toxic chemical matters, which we can classify as a dangerous waste. Hygienic inspection is the technological process of sludge treatment, which is used in the case, when the sludge properties are not fulfilled the request of valid European legislation for next sludge reuse in agriculture, or other fields of treatment. In continuation text are described present methods of hygienic inspection used in Slovakia and neighbouring countries.

Keywords

Hygienic inspection, methods, reuse, sewage sludge disposal and treatment.

1 INTRODUCTION

Sewage sludge definition is: it is a disposal from urban wastewater treatment. We define is as a disperse system, which consists from the suspended, colloid matters. The main part creates suspended matters with characteristics concentration 5-50 g.l-1. The sludge consistence is liquid till greasy. Sludge creates from 1 till 2% of waste water volume and contains 50-80% of primary pollution. The sludge as waste product from various steps of wastewater treatment is no stabile, it is the reason of the next sludge treatment, before it´s using as a lateral stuff.

2 SEWAGE SLUDGE CHARACTERISTICS

The basic sludge characteristic is volume of suspended solids. This characteristic we verify by transpiration by temperature 105 °C as a weight ratio p_{ss}. To the volume of suspended solids we calculate also the matters dissolved in sludge, which are not streamed off and stayed into the suspended solids. Its ratio in the suspended solids is low from 0,5 do 1,0 g.l^{-1}. Weight ratio of water in sludge is marked as p_v and it is a supplement to suspended solids of the sludge.

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General is valid $p_{ss} + p_w = 1$. Volume of suspended solids and water in sludge is expressed in per cent as [5]:

$$100p_{ss} + 100p_w = 100\% \quad (2.1)$$

Volume of suspended solids of primary sludge is usually about 2,5%. Volume of suspended solids of redundant sludge is from 0,5 – 1,0%. By process of sludge thickening is possible to increase volume of suspended solids to 4 – 6%, but the sludge is still liquid. After following stabilization and dewatering becomes sludge to solid consistence, similar to the soil. The suspended solids are from 20-50%. Next by thermic treatment is possible to get 90% suspended solids in sludge.

There are two components into suspended solids – organic and mineral. Ratio of organic p_{sso} is settled as a loss by stripes by temperature 550°C. The organic matter burn up and mineral stayed no distributed as a residue after burning. Ratio of mineral suspended solids is supplement to the organic and is valid [5]:

$$100_{psso} + 100p_{ssa} = 100\% \quad (2.2)$$

Specific suspended solids sludge production SPs [$g.PE^{-1}.d^{-1}$] by middle (high) loaded activation is [5]:

by raw sludge 85 (79) $g.PE^{-1}.d^{-1}$

by anaerobic stabilized sludge 55 (52) $g.PE^{-1}.d^{-1}$

Specific sludge production SP_k [$g.PE^{-1}.d^{-1}$] is established as a ratio of suspended solids sludge production SPs and volume of suspended solids sludge p_{ss} [5]:

$$SP_k = \frac{SPs}{p_{ss}} \quad (2.3)$$

Sludge suspended solids density ρs [$kg.dm_{-3}$] is possible to derive from following relations [5]:

$$\rho = \frac{m}{V}, \quad V_s = V_{so} + V_{sa}, \quad m_s = m_{so} + m_{sa}, \quad p_{so} = \frac{m_{so}}{m_s} \quad (2.4)$$

Sludge suspended solids density depends from [5]:

$$\rho_s = \frac{\rho_{so} \cdot \rho_{sa}}{(1 - p_{so}) \cdot \rho_{so} + p_{so} \cdot \rho_{sa}} \quad (2.5)$$

where

ρ_s [$kg.dm^{-3}$] sludge suspended solids density

ρ_{so} [$kg.dm^{-3}$] sludge suspended solids density of organic ratio (cca 1,0 $kg.dm^{-3}$)

ρ_{sa} [kg.dm^{-3}] sludge suspended solids density of mineral ratio (cca 2,5 kg.dm^{-3})

Vs , Vso , Vsa [dm^{-3}] volume of sludge suspended solids and its ratios

ms , mso , msa [kg] weight of sludge suspended solids and its ratios

Analogue is possible to derive sludge density ρk, which depends from density of suspended solids ρ_s and density of water ρ_w [5]:

$$\rho_k = \frac{\rho_s \cdot \rho_w}{(1 - p_s) \cdot \rho_s + p_s \cdot \rho_w} \qquad (2.6)$$

When we consider, that ρw ≈1,0 kg.dm-3, is possible to reduce the equation (2.6) [5]:

$$\rho_k = \frac{\rho_s}{(1 - p_s) \cdot \rho_s + p_s} \qquad (2.7)$$

Next parameter, which we use especially for liquid sludges is suspended solids concentration X [kg.m^{-3}]

$$X = 1000. ps . \rho k \; (2.8)$$

The quality of redundant sludge depends from the type of sludge and from parameters of treatment technologies. From this point of view one of the important characteristic is sludge Θ_x. It is the amount of activated sludge in system reach at daily sludge production – time of biomass restraint in the system. Next parameters characterizing thickening and dewatering properties are sludge index KI and appearance of fibrous organism and foam production. Sludge index KI [ml.g^{-1}] is volume of settled sludge after 30 minutes long sedimentation reach at concentration of sludge suspended solids X. We can say, that KI values till 100 [ml.g^{-1}] indicate good settled sludge, by values over 200 [ml.g^{-1}] gives out to sludge washing out and to production of fibrous microorganisms.

One of the most important sludge characteristics is hygienic aspect. Sewage sludge is a waste product, which comes into being in the process of wastewater treatment and so underlies under Waste Low. This act regards all stuff, which has at least one dangerous property to the dangerous waste. The producer of this waste is obligatory disposal the sludge as a dangerous waste. Lot of sludge produced on the WWTP has one dangerous characteristic. It is infectivity. Infectivity is caused by pathogen bacterial. Their amount depends from the geographic, climate and demographic factors. The main source of pathogen bacterial is excrement of ill people or animals. Majority of these bacterial is disposal by the treating process, but without perfect sludge hygienic inspection, the small amount stays still in the sludge. By the using of sludge in agriculture can expose these pathogenic germs healthy of living organisms. By the pathogenic organisms which occurs in the waste waters belong:

- viruses
- germs as a Salmonella, Escherichia coli
- protozoa
- lateral worms

Sludge often consist toxic chemical matters, which we can classify as a dangerous waste. There are some organic matters and heavy metals. The source of heavy metals is Cadmium, Chrome, Copper, Mercury, Lead and Zinc. We can find this matters into the industrial wastewaters from the metal and leader industry, from the wet and dry deposition.

3 MEDHODES OF SEWAGE SLUDGE DISPOSAL AND HYGENIC INSPECTION

The most using process of sewage sludge disposal in Slovakia is application into the soil. The controlled application must be done according to the valid legislation (Act no.188/2003 Coll. and Act no. 136/2000 Coll.). Hygienic inspection is the technological process of sludge treatment, which is used in the case, when the sludge properties are not fulfilled the request of valid Slovak legislation. The main part of waste, microorganisms and viruses proceed after treatment into the sludge. By hygienic inspection improved the hygienic and sanitary properties of the sludge. We can use for it various methods as a pasteurisation, aerobic, thermophilic stabilisation, chemical methods-with using CaO, by gamma radiation or irradiation and incineration.

3.1. Pasteurisation

Pasteurisation is a process, when the sludge is heated during relative short time on a temperature 70°C. Process of pasteurisation was established in Switzerland. The goal of it is elimination of germs and viruses especially of salmonella. Pasteurisation no replaced process of sludge stabilisation. It has to be combined with some other process of stabilisation. The process of pasteurisation is high effective by killing of various types of viruses especially on entherogerms and salmonella. It is no effective by thermo tolerant types of life forms as a certain types of viruses and spores.

3.2 Irradiation

There are only marginal experiences of irradiation of sewage sludge in the Europe. By the irradiation are most reduced salmonella and entherogerms. By the operational point of view is interesting, that the irradiated sludge has better dewatering properties as a no irradiated sludge.

3.3 Aerobic thermophilic stabilisation

Aerobic thermophilic stabilisation was developed for the sewage sludge stabilisation. High level of aeration reinitiate by biological processes in such intensity, that the warmth generated by the process keeps the temperatures needed for the disinfection. We know two concepts of aerobic thermophilic stabilisation:

- aeration by clear oxygen
- aeration by air oxygen

By the using of clear oxygen, are the average temperatures about 60-80°C. The disinfection effect will be comparable with the pasteurisation and we can reach also better results then by pasteurisation. When is the process supplied with oxygen from the air, the reached temperature is 40-60°C. In this temperature interval is the process of disinfection reduced and there is needed longer time for deactivation of pathogens. To find the optimal relation between aeration and temperature is not easy.

3.4 Composting

Composting is a process, which is depending on aerobic reduction of organic matter by thermophilic germs. Sludge is mixing with filling mass, what served to the increase of porosity for better aeration, for reduction of content of humidity and improve of relationship between C:N. There are often combined all three functions in one product. For example: straw, wood peel or household waste. There can be used also no degradable material as a plastic. It was developed many types of composting processes, so is really difficult to define average data for requested disinfection effect. Generally we can speak about two basic processes:

- Composting in composting lagoon
- Composting in bioreactor

By composting in composting lagoon is the sewage sludge in a role of filling mass. For requested sanitary effect is necessary to achieve the critical temperature for requested time distance. For elimination of salmonella in summer period is the critical time established on 6-7 weeks.

3.5 Lime treatment

Lime is often added into the swage sludge before dewatering. That caused increase of pH level, it can reached value from 9-13 in dependency of lime supply and sludge characteristics. Vegetative germ cells (coliform germs or salmonella) are fast reduced to nothing by value of pH from 9 till 10. According to Strauch and Berg, pH level has to be higher than 11,5 for reaching of requested efficiency. By these ph values will be destroyed most of viruses. Destruction of the viruses is not effected directly by ph effect, but by unlocking of free ammonia by pH level around 12. Complete destruction can be reached by adding of quicklime into the dewatered sludge. At this moment the temperature increase till 70-80 °C and all pathogens died very fast. The ain destruction factor is temperature.

3.6 Incineration

Incineration is one of the most effective processes of sewage sludge treatment and sanitation. By the still growing amount of sludge it is a perspective solution for sludge disposal. But the process of sewage sludge incineration is not easy to realise in really conditions. The dewatered sludge contents about 75% of water. The complication is also high value of heavy metals in sludge. It can cause creation of vapours by higher incineration temperatures. One of the perspective methods of sewage sludge incineration was founded fluid layer. The fluid layer create thermal homogeneous environment, which is reach on oxygen with strong abrasive influence on the sludge elements. The thermal degradation has a lot of advantages. At first it is volume reduction about 87%. The ash from the incinerated sludge is inorganic and mostly is it sterile matter. So we can use it in building industry. The heavy metals are strong bounded on sorbate and so they are not washed up. The gas emissions are deep under allowed limits. Also the process of thermal degradation is friendly to the environment. By the treatment on WWTP we can skip the sludge treatment from the technology of waste water treatment and replace it with thermal sludge degradation.

Fig. 1 Incineration station based on sludge burning on fluid layer

The financial cost of the sludge processing, however, the overall economic balance significantly affects the treatment of waste waters, which represented more than half of the total cost of waste water treatment. The production of the sewage sludge, cannot be avoided, innovative technologies can reduce the amount of sludge or energy use. The significant step by sludge treatment is sludge reuse. One of the possibilities is slow thermal decomposition of sludge, which is able to use maximum of its energy potential. The sludge is not perceived as a waste, but as the raw material, which is the energy – valuable product. It is possible to produce high quality energy products from the sewage sludge. Namely gas, liquid and solid fuels, which can be used directly on WWTP. There are involved several methods for production of high efficient fuels from redundant sludge. These solid materials, in the form of pellets, can be used in industry as a fuel in the ballast furnace, or also by electric energy production, when is thermal energy change in cogeneration units to the electric energy. It can be sold to the other industrial or living areas. The main benefit of this process is, that we don´t load the environment with large amount of redundant sludge and we produce the clean environment friendly energy, which is essential for our life.

Tab. 1: Methods of sludge hygienic inspection

Sludge hygienic inspection – methods and efficiency

Bacterial effect						
Type of process	Death factor	Bacterium	Viruses	Vermins	Product stability	Notes
Pasteurisation	temperature 30- 70°C	good	unstable	good	post- weakly before- good	Combination with stabilisation needed
Irradiation	ionized radiation 300 krad	good	weakly	good/average	unstable	Combination with stabilisation needed
Aerobic thermop. st. - oxygen - air	temperature 60 - 80 °C 40 - 60 °C	 good good/weakly	 good/unstabl weakly/unsta	 good weakly/good	probably good	Effect depends on mixing characteristics
Composting - in composting lagoon - in bioreactor	temperature 40 - 60 °C 50 - 80 °C	 weakly/good good	 weakly/unsta weakly/good	 weakly/good good	good	Effect depends on mixing climate characteristics
Lime treatment - slack lime - no slack lime	High ph level and free ammonia pH till 12 pH till 12 temperature till 80 °C	good good	averag/good good	unstable good	good by pH > 10	High pH can destroid epidemie of various vermins eggs
Incineration - on grate - on fluid layer	temperature 800 - 900 °C	good	good	good	good	High technology investition

4 CONCLUSION

Wastewater treatment involves the generation of large volumes of sludge and other waste which management in an economical and environmental acceptable way has become a matter of increasing importance during the last few years. While the technologies and processes to reduce sludge generation are being widely studied, contributions relative to economic aspects are much more limited. However, when WWTPs operators face to the implementation of these technologies, not only technical aspects must be considered but also influence on environment regarded. Technical solution, economical aspect and environment friendly, that are the three highlights, which have to bee in balance. We can see, that is possible to use various methods for sludge disposal. But our goal is to choose the best alternative from the technical, environmental and last and but not least economical point of view. Each WWTP has its own sludge treatment and also specific condition for sewage sludge disposal and hygienic inspection. We have to find the optimal solution for the people and for the environment.

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RISK ASSESSMENT FOR LANDFILL

I. Škultétyová[1] and Š. Stanko[2] and K. Galbová[3]

Abstract

Municipal waste management is one of the most important services in urban areas. A number of technical, demographic, financial, institutional, economic, and social factors contribute to the failure to waste management. It is difficult to adopt a general good waste management system which would be as successful in all locations. Facilities used for waste disposal are final element for waste management in these systems. Landfills were the most widely used equipment in the past and they are still in many countries now although they representing the highest risk of environmental pollution and health hazards. Pollution of surface water or groundwater pollution due to landfill leachate and its percolation is one of the negative impacts of landfills. Leachates are generated in the landfill body from various sources and under the influence of a number of conditions. Risk assessment is a key process in the future evaluating landfill developments and therefore an example of the methodology for risk assessment of landfill contamination is describes in this article.

Keywords

Environment, flood, landfill, leachate, risk assessment, solid waste

1 INTRODUCTION

The municipal wastes are disposed by various ways, but at least acceptable way from the waste hierarchy, human risk and environment is the landfilling. The landfilling is the simplest

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and very often the cheapest way of solid waste disposal. This fact predicts designation, that many parts of the world now and will at next, to solve waste disposal by the landfilling.

The last thirty years many authors designed the models, which modelling the leachate liquid flow through the municipal landfill. There are the models such as: HELP - Hydrologic Evaluation of Landfill Performance; FILL - Flow Investigation of Landfill Leachate; LandSim or EPACMTP - EPA's Composite Model for Leachate Migration with Transformation Products. The other models are based on risk analyses evaluation mentioned by various literature searches. The every of these evaluation systems evaluate the relative level of risk by one or more parameters – diffusion of pollution by underground waters, surface waters or atmospheric dispersion, burning and explosiveness, the direct contact with harmful pollutants, the type of landfilling waste, or the changes in landfill body and others. In the operational landfill period and after the landfill close, the influence of the landfill is relatively very good predicted. The influence in the flood period, there are no so many experiences. The contamination from the landfill is possible to estimate base on methods which exploit the evaluation of risk level.

2 LANDFILLING OF SOLID WASTE

Landfilling of solid waste remains the most widely used waste management method. The landfill means a technical facility which is characterized with physical, chemical and biological processes. The location and characteristics of the site will determine the extent and nature of the impact of a municipal landfill on public health and the quality of the surrounding water, air and land resources. Many communities are having difficulties siting new landfills, largely as a result of increased citizen concerns about the potential risks and aesthetics associated with landfilling in their neighbourhood. To reduce the health and environmental risk there were developed minimum criteria that solid waste landfills must fulfil. [7].

3 THE LANDFILLS IMPACT ASSESSMENT SYSTEMS

The impact of the landfill on human health and the environment is largely well predictable during landfill operation and post-disposal operations. This is documented in numerous published studies but the flood impact on landfill is still poorly understood [8]. In both cases, the contamination from landfills are estimated by the methods of systems for assessing the risk.

The other models were based on risk analyses evaluation described in many literature [5]. The every of these evaluation systems evaluate the relative level of risk by one or more parameters – diffusion of pollution by underground waters, surface waters or atmospheric dispersion, burning and explosiveness, the direct contact with harmful pollutants, the type of landfilling waste, or the changes in landfill body and others. In the operational landfill period and after the landfill close, the influence of the landfill is relatively very good predicted. The influence in the flood period, there are no so many experiences. The contamination from the landfill is possible to estimate base on methods which exploit the evaluation of risk level.

The risk of flooding is determined for landfill sites in Austria using areal data about flood risk zones HORA (Hochwasserrisikozonierung Austria) in Austria. The HORA-data set provides information about potential flood inundation zones along rivers for discharges with a return period of T=30 years, T=100 years, and T=200 years [4]. The site coordinates of the landfill and information about designated flood risk zones are integrated in a geographic information system (GIS), which is used for a categorization of flood risk exposure [2]:

- endangered landfills, are situated within a designated flood risk zone,

- probably endangered sites, are less than 150 m away from a designated flood risk zone,
- probably not endangered sites, do not have a designated flood risk zone within a distance of 150 m around the landfill coordinates.

The results of the evaluation of flood risk exposure of Austrian MSW landfills indicate that one third of the landfills are located in flood prone areas (assigned either to the category "endangered" or "probably endangered" of flooding), with more than 10 % being located directly in a potential inundation zone corresponding to a recurrence interval flood of T=30 years. Only a small portion of the endangered and probably endangered sites have flood protection facilities. Whereas around 60 % of active sites (only around 5 % of the investigated landfills) have some kind of flood protection facilities (i.e. dams), less than one third of the closed sites in flood prone areas reported technical flood protection. [2].

4 RISK ASSESSMENT METHODOLOGY OF LANDFILL SITES CONTAMINATION IN FLOOD ZONES

Evaluation of the proposed systems to determine the degree of risk of contamination from landfills is carried out mainly on the basis of components, including:

- **source** landfill, which is characterized by the amount and potency of waste contaminants released from landfilling waste,
- **pathway** characterized by the migration of contaminants from source to receptor,
- **receptor** the landfill vicinity, which is described by different characteristics permitted of contaminant transport (soil, groundwater, groundwater users - human beings, livestock, crops, and sensitive environment).

Singh [5] proposed the system for a hazard rating evaluation (see a conceptual diagram in Fig. 1) in which the hazard rating (R) of a waste disposal site is given by the following relationship:

$$R = R_Z \cdot R_P \cdot R_R \qquad (1)$$

where R – hazard rating; R_Z - is source hazard rating; R_P – is pathway hazard rating; R_R – is receptor hazard rating.

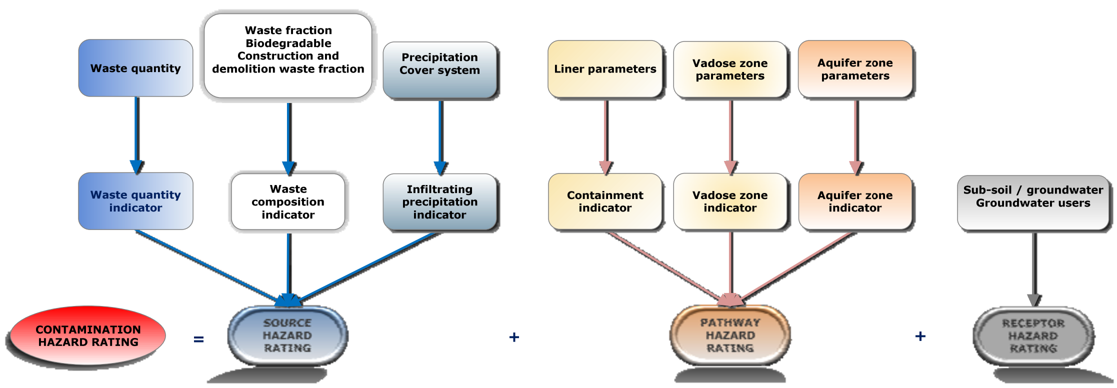

Fig. 1. Diagram of hazard rating evaluation system

Source hazard rating **(R_Z)** is a combined measure of the amount and toxicity of the contaminants contained in a landfill and the potential for their leaching out from the fill. While the amount of contaminants will be directly proportional to the waste quantity at a site, the toxicity of contaminants will depend on the type or composition of the waste. The potential for leach-out of contaminants from a fill depends on the amount of precipitation that infiltrates into the waste.
Pathway hazard rating **(R_P)** - the waste contaminants carried by the infiltrating rainwater, and released from a fill, move through three distinct pathway media namely containment zone (base liner), if any, vadose zone, and aquifer zone, before reaching a recipient groundwater well. In each medium, contaminant movement is retarded by way of physical barrier or attenuation mechanism; as such the contaminant loading within a pathway medium decrease as the contaminant moves from the source to a receptor.
The receptor hazard rating **(R_R)** is equal to the summation of the indicators for sub-soil/groundwater, and various groundwater user categories (e.g., human population, livestock, crops, and sensitive environment).

5 RESULTS AND DISCUSSION

As illustrated in figure (Fig. 2) landfilling of solid waste remains the most widely used waste management method. Landfills are placed near the cities and lowland areas that may be at risk from flooding in many cases. The risk of contamination from the landfill involve probably with existence of flooding and the consequences of this event - flooding of the landfills body, hydrodynamic effects on landfill and environmental impacts [1].

Fig. 2. Municipal waste landfilling rates in 32 European countries, 2001 and 2010 [3]

There are illustrated the landfills localization in Slovakia in the map (Fig. 3). The system's ability to evaluate a risk of contamination from Slovakia landfills depends on the amount of information taken into the system.
A hazard rating system must be sensitive both to change in site conditions as well as change in the type of waste. The site characteristic parameters of landfills, which were applied in the

existing systems, are site topography/slope, landfill area, landfill height, cover system, active life, liner and leachate removal system, annual precipitation, evapotranspiration, depth to aquifer, vadose zone permeability, aquifer permeability, aquifer thickness, groundwater gradient, distance to nearest well, population using groundwater, crop area using groundwater, livestock using groundwater, sensitive environment using groundwater, alternate source of water supply available. The type of waste is municipal solid waste but i.e. hazardous waste, construction and demolition waste too.

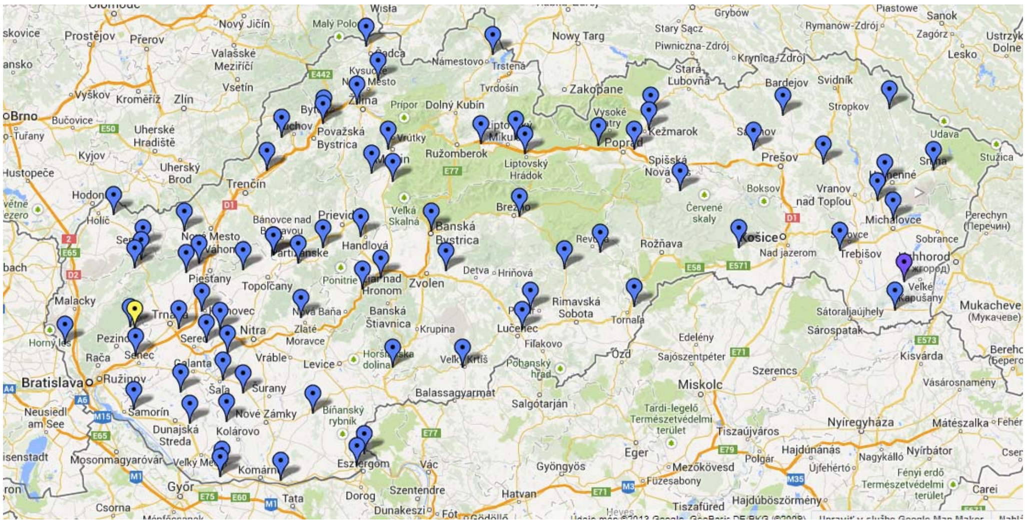

Fig. 3. Landfills in Slovakia [6]

The input data of parameters and their relative importance weights with scale of 0 - 10 for the evaluating system have been identified based on literature review and expert opinions. A rating of „0” indicated that the parameter was not important at all, hence should not be considered, whereas the rating of „10” was to be assigned to the most important parameter.

Tab. 1 System Parameters and Their Relative Importance Weights

Parameters		Relative importance weights
Cover components	Surface grade (top deck) [%]	0.10
	Vegetative/top soil layer thickness [m]	0.15
	Drainage layer thickness [m]	0.15
	Clay layer thickness [m]	0.30
	Geo-membrane thickness [mm]	0.30
Containment system	Thickness of clay layer [m]	0.40
	Thickness of HDPE geomembrane [mm]	0.35
	Leachate collection & removal system	0.25
Aquifer zone	Aquifer thickness [m]	0.30
	Aquifer permeability [m/sec]	0.35
	Groundwater gradient [%]	0.35
	Distance to nearest groundwater well [m]	-

6 CONCLUSION

Municipal landfills pose large and long-term potential risk to human health and the environment. The assessment methodologies are described in many literary sources and environmental risk assessment of municipal waste landfills are implemented in several studies, whereby data on the impact of landfills in operation, respectively after operation under normal circumstances, are dominate. Description of the flood negative impacts on the landfill currently seems to be inadequate despite the risk assessment system for landfill flooding is already developed by some countries of the European Union.

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WATER RESOURCES ENGINEERING PRINCIPLES AND PRACTICE IN THE PROVINCE OF ONTARIO, CANADA WITH A FOCUS ON PRACTICAL STORMWATER MANAGEMENT APPLICATIONS

P. Spal[1]

Abstract

Evolution of stormwater management in Ontario, Canada has changed dramatically from mid-1970 and has progressed to a comprehensive strategy protecting the natural attributes of the watershed. Implementation of water resources engineering and principles are governed by Federal, Provincial and Municipal Authorities and designs in Ontario have to meet specific regulatory targets. Practical examples are presented regarding the hydrological and hydraulic modelling and end-of-pipe stormwater management facility design.

Keywords

Canada, Ontario, City of Ottawa, IBI Group, stormwater management, regulatory requirements, hydrological, hydraulic, modelling, stormwater management facility.

1 INTRODUCTION

During the past twenty years, the philosophy of stormwater management in Ontario has changed dramatically. In the mid-1970s, it was generally thought that the best way of handling stormwater was to provide conveyance systems to quickly and safely move stormwater away from a subject area. However, it became apparent that although this approach achieved its objective, significant negative impacts associated with flooding and erosion of downstream drainage system were associated with this practice.

In recognition of this, various regulatory agencies (i.e. Ontario Ministry of Natural Resources and Conservation Authorities) subsequently required the introduction of stormwater management (SWM) facilities as an integral part of a development plan. The purpose of

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SWM facilities was to reduce peak out flow rates from each new development to pre-development conditions. Although it was apparent that implementation of this approach, on a site-by-site basis, would achieve the required goals and objectives, watershed-wide impacts were unknown.

In the mid-1980s, concepts of urban drainage began to evolve toward a more holistic view of protecting the natural attributes of the watershed. SWM facility design expanded to provide water quality and erosion control and became more integrated into the overall ecological and environmental watershed system.

The development of an area is preferably investigated through a Subwatershed Study, which is a detailed analysis of the area’s existing conditions, including hydrologic, hydrogeological, and geotechnical characteristics, as well as terrestrial, aquatic and ecological conditions. The study also investigates potential development scenarios or planning for the area. The Subwatershed Study becomes a catalogue of the area and a reference for the developers and designers for future serviceability studies.

The stormwater management concept alternatives are usually developed during the Master Servicing Studies (MSS). The MSS outlines proposed development, including conceptual servicing of water, sanitary, storm sewers and SWM facilities. Master Servicing Studies may become part of the Class Environmental Assessment (EA) report for that particular development area. The EA process, detailed in Municipal Class Environmental Assessment by the Municipal Engineers Association (October 2007) [1], requires certain information be provided in the report, including existing conditions, potential alternatives in the design, and public consultation. Generally, most Master Servicing Studies are subject to an EA. The complexity of an EA is outlined in schedules ranging from the simplest Schedule A to more complex B and C up to the most complex Individual EA Schedule. SWM systems are typically undertaken through Schedule B or C.

In some circumstances, depending on the history of the studies completed for a specific area, the Class EA requirements for SWM facilities may be satisfied through the Ontario Planning Act. Under this scenario, the SWM facility is also subject to public consultation during specific periods of review, as specified in the Planning Act and the municipal review process.

The Master Servicing Study identifies the preferred SWM system alternative and provides guidance for the further design of particular components. Depending on the size and complexity of the proposed development, a sequence of more detailed SWM designs is prepared and documented in SWM reports which typically include pre-design drawings. To facilitate construction, the SWM systems are provided with a design brief and detail design drawings.

2 APPROVALS AUTHORITIES

2.1. Federal Authorities

Depending on the project complexity, as well its location, SWM systems may require a formal authorization and review under the Canadian Environmental Assessment Act (CEAA), the relevant federal agency that takes the lead in administering the process. For SWM projects in Ottawa, the lead agency is typically Fisheries and Oceans Canada, the National Capital Commission, or in some cases, Parks Canada, Transport Canada or Environment Canada.

2.2 Provincial Authorities

SWM systems fall under the Ontario Environmental Protection Act (EPA) and as such each SWM system requires an Environmental Compliance Approval (ECA). The ECA certificate is issued by the Ontario Ministry of the Environment (MOE) under Section 20.2 of Part II.1 of the EPA. The approval is issued for the treatment and disposal of sewage (including stormwater) by municipal and private systems. Environmental Compliance Approvals are required for SWM facilities since they discharge to surface water or, less commonly, to groundwater. SWM facilities designed to serve a single lot or parcel of land (excluding industrial land) that discharge to a storm sewer are exempt from requiring an ECA.

The ECA application for SWM facilities is reviewed by MOE engineers and technical staff to assess the project’s compliance with Section 20.2 of the EPA and other applicable provincial regulations and guidelines. The MOE review is focused primarily on the requirements of the MOE Stormwater Management Planning and Design Manual (March 2003) [2] and how it has been applied to the design. In some cases other provincial ministries are involved, such as Ontario Ministry of Natural Resources and Ontario Ministry of Transportation.

Water resource management on a watershed basis is regulated by Conservation Authorities (CA). In Ontario, CAs are community-based environmental experts who use integrated, ecologically-sound environmental practices to manage the province’s water resources on a watershed basis; maintain secure supplies of clean water; protect communities from flooding; and, contribute to municipal planning processes (that protect water). A signed approval letter from the CA is required by the MOE in order to grant an ECA.

Frequently, most of the construction works related to SWM systems is in proximity to lakes and rivers. As a result, more specific permits are required to facilitate the construction works. The permits are issued by CAs under specific provincial regulations related to development, interference with wetlands, and alterations to shorelines and watercourses. Relevant permits must be applied for through the CA, supported with project documentation, including drawings.

2.3 Municipal Approvals

SWM systems constructed within the City of Ottawa will eventually become part of the City of Ottawa infrastructure. This is similar to other municipal infrastructure, such as water mains, storm sewers, sanitary sewers and pump stations. Considering the assumption of SWM facilities, the City of Ottawa has an interest in ensuring that their regulatory requirements are met through review and comments on the design. Review of SWM facility design includes calculations and modelling, with a strong focus on the facility’s functionality and required maintenance.

3 REGULATORY REQUIREMENTS

In the Province of Ontario, guidance for the planning and design of stormwater management systems was developed by the MOE and published in the above-noted Stormwater Management Planning and Design Manual (March 2003) [2]. The main aspects of the regulatory requirements are presented in the manual as environmental design criteria, including those listed below.

- Water Balance
  Targets are related mainly to maintaining or enhancing infiltration and baseflow.
- Water Quality
  Targets are related to mitigation of negative impact of urbanization on recipients and reductions of pollutant loads such as total suspended solids, nutrients and thermal pollution.
- Erosion Control/Geomorphology
  Fluvial geomorphology assessment is typically required to assess the required targets to prevent degradation of the receiving streams,
- Water Quantity
  Water quantity attenuation of urban runoff is required to reduce peak flows to pre-development levels. In some cases, more stringent quantity controls may be required to alleviate existing flooding problems.

The City of Ottawa developed SWM facility design guidelines, entitled Stormwater Management Facility Design Guidelines and Standards (IBI Group, 2013) [3]. The premise of the guidelines is to reinforce design parameters that better reflect local conditions and requirements, as well as to detail constraints and present opportunities focused on SWM facility design. The guidelines provide complementary information to the MOE Manual [2].

4 STORMWATER MANAGEMENT SYSTEMS

To achieve the above-noted regulatory requirements, the recommended stormwater management strategy is to provide an integrated treatment train approach by providing control at the lot level, during conveyance, followed by at the end-of-pipe.

Lot level and conveyance controls are applied at the individual or multiple lot level, as well as within the stormwater collection system. They are typically only suitable for small drainage areas and are categorized as either storage or infiltration controls. Storage controls (such as roof, surface or underground storage) reduce peak runoff rates, while infiltration controls (such as lot grading, vegetated swales, infiltration trenches and pervious pipes) aid in mitigating the impacts of urbanization by allowing runoff to infiltrate on a localized basis. Lot level and conveyance controls are considered integral components of an overall stormwater management strategy and are frequently employed to maintain the natural hydrologic cycle to the greatest extent possible.

End-of-pipe controls receive water from a conveyance system and discharge to a receiving water body. They commonly service the entire tributary sewershed area. End-of-pipe controls are usually required for flood and erosion control and water quality improvement; however, lot level and conveyance controls can be used to eliminate the requirement, or reduce the size, of the end-of-pipe controls.

In some cases, when the drainage area is small, or a treatment train approach is desired, stormwater oil-grit separators and filters are implemented. The majority of end-of-pipe controls in the City of Ottawa involve the design and construction of SWM facilities.

5 HYDROLOGICAL AND HYDRAULIC MODELLING APPLICATIONS

Hydrological and hydraulic models progress from the conceptual level of design to evaluate stormwater management on a subwatershed basis to more detailed models to evaluate dual drainage on a micro scale. At the micro scale, the major and minor system is considered, and some aspects of water quality and erosion control are evaluated using continuous modelling.

5.1. Conceptual Modelling

At the initial stages of development, hydrological modelling is employed to evaluate SWM alternatives on a subwatershed basis, considering water quantity, water quality and erosion control targets. Using single storm event modelling software, runoff simulations are performed for variety of design storms to evaluate overall development runoff with respect to the subwatershed, recipient and stormwater management alternatives.

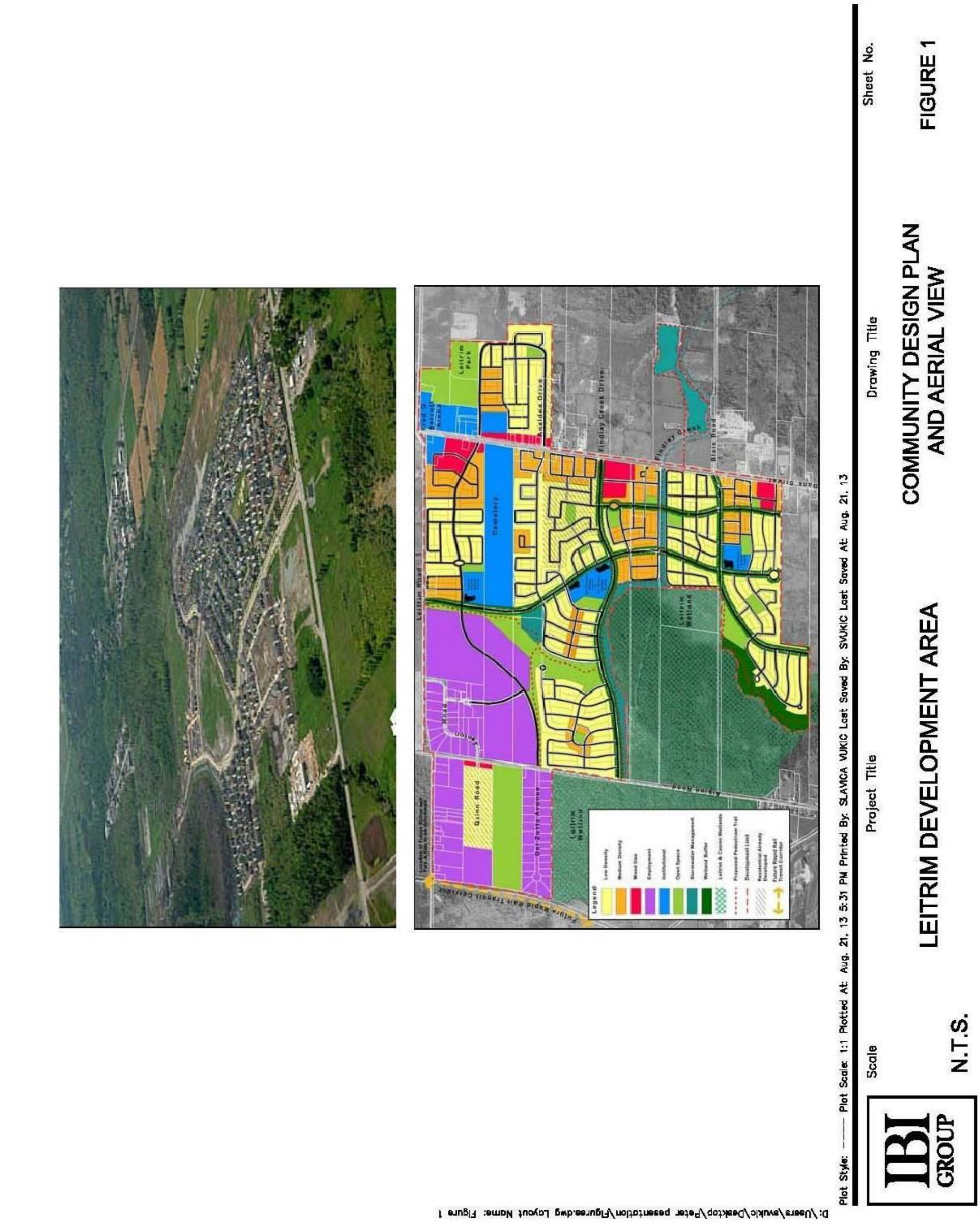
For example, IBI Group has undertaken preliminary engineering and conceptual modelling for the Leitrim Community, a development area of 500 ha (1200 acres), 4000 unit mixed-use community. Figure 1 presents the community design plan and aerial view of the development to date [4]. The project has included the preparation of Master Servicing Studies to support the inclusion of the Leitrim Community in the City of Ottawa urban area; the preparation of stormwater management studies dealing with wetland protection and fish habitat conservation; the design and construction of municipal services, local and collector roads; and the design of interim stormwater management measures to facilitate the phased construction of the multi-million dollar development. Figure 2 presents the hydrological model schematic for the development area [5].

Since the inception of the Leitrim Community, stormwater management has been a key component to the development design. IBI Group has been involved in all aspects of stormwater management design from the initial Environmental Study Report and Pre-Design in 1995 which satisfied the Environmental Assessment process and explored different alternatives for providing stormwater management to the community. The goal of the stormwater management system for the Leitrim Community is to function as a link between the residential development and the area’s natural wetlands and creeks. The Environmental Study Report was updated in 2005 with the consolidation of three facilities and relocation outside the urban boundary. IBI Group has used sophisticated hydrologic models and fully dynamic modelling and for the design of the Findlay Creek Village Stormwater Management Facility; and analysis for the Serviceability Report, Findlay Creek floodplain modelling and Wetland Control Structure. Figure 3 presents the artist rendering of the design and the aerial view of the Findlay Creek Village Stormwater Management Facility [6].

5.2 Detail Dual Drainage

The hydrological model developed during the conceptual stage is typically further refined to reflect the minor and major system components (dual drainage) of the development. Simulations are performed for a variety of design storms to confirm the overall development runoff with respect to the subwatershed and recipient.

A dual drainage system is considered standard urban drainage design to accommodate stormwater urban runoff from new developments. Dual drainage accommodates both major and minor stormwater runoff. During frequent storms the effective runoff collected by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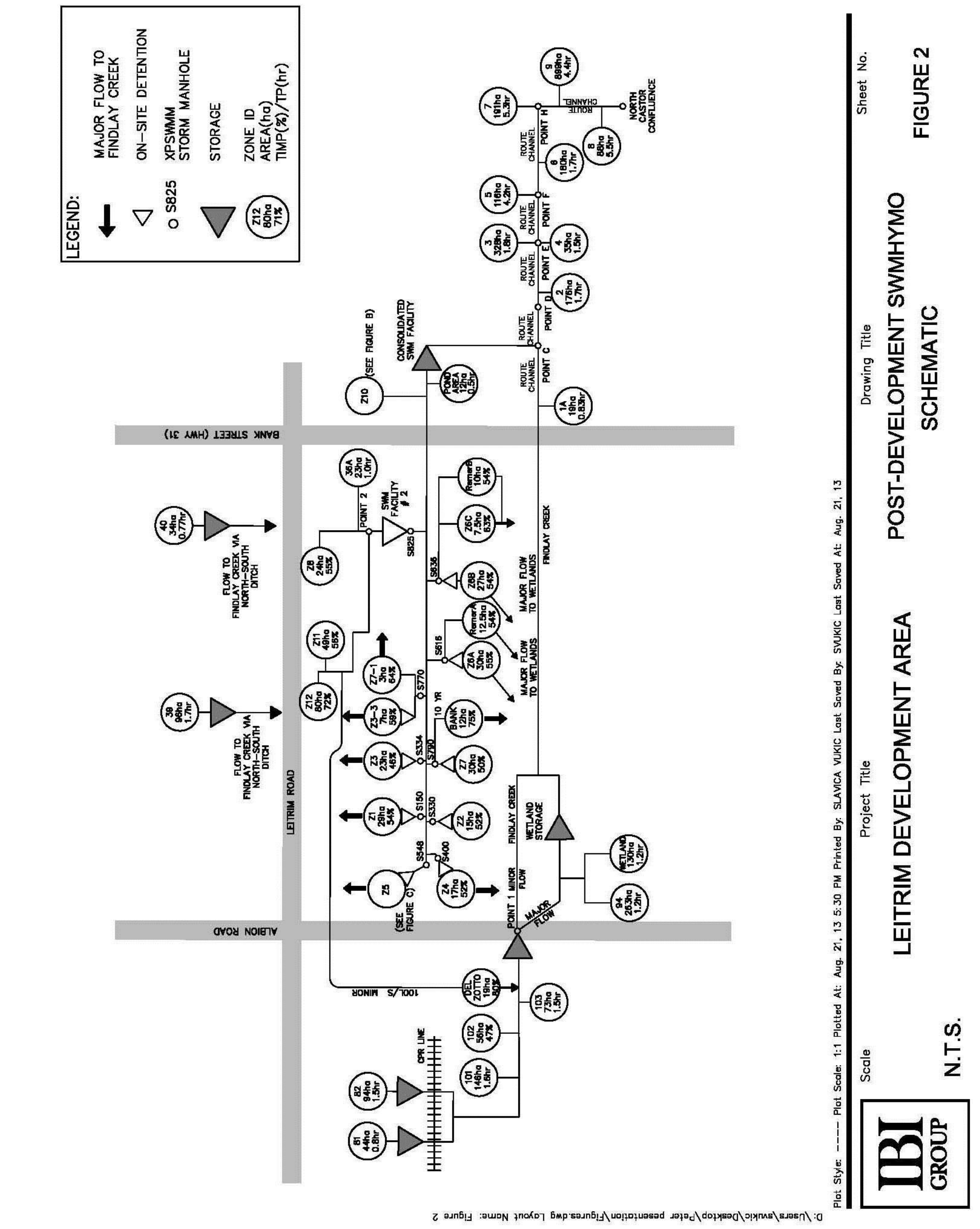
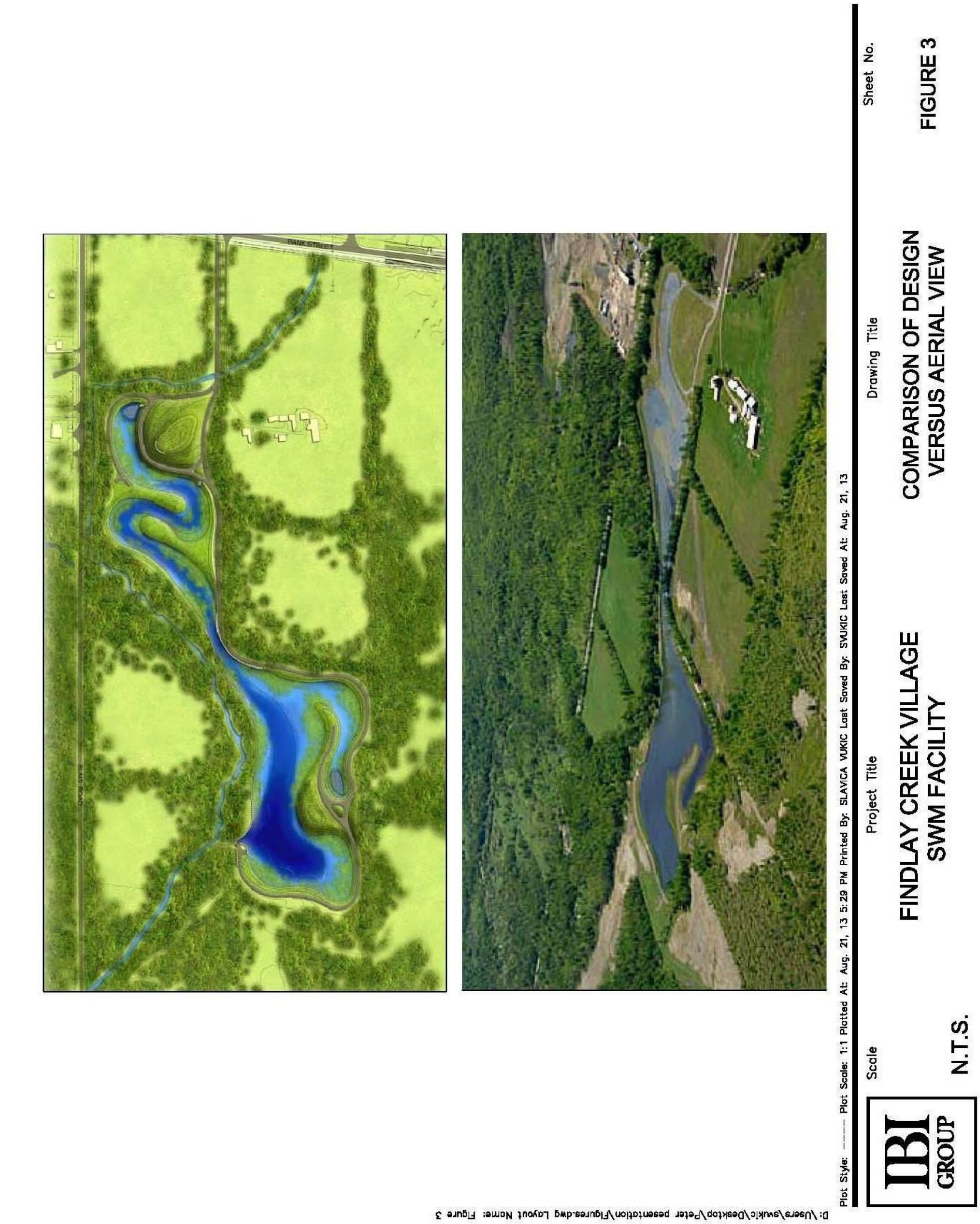
Scale
N.T.S.
Project Title
LEITRIM DEVELOPMENT AREA
Drawing Title
COMMUNITY DESIGN PLAN
AND AERIAL VIEW
Sheet No.
FIGURE 1
IBI
GROUP
Leitrim Road
Cemetery
Leitrim Park
Leitrim Wetland


LEGEND:
MAJOR FLOW TO FINDLAY CREEK
ON-SITE DETENTION
S825 XPSWMM STORM MANHOLE
STORAGE
ZONE ID AREA(ha) TIMP(%)/TP(hr)
Z12 80ha 71%
LEITRIM ROAD
BANK STREET (HWY 31)
ALBION ROAD
FINDLAY CREEK
FINDLAY CREEK WETLAND STORAGE
CONSOLIDATED SWM FACILITY
NORTH CASTOR CONFLUENCE
Scale
N.T.S.
Project Title
LEITRIM DEVELOPMENT AREA
Drawing Title
POST-DEVELOPMENT SWMHYMO SCHEMATIC
Sheet No.
FIGURE 2
IBI GROUP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D:\Users\svukic\Desktop\Peter pesentation\Figures.dwg Layout Name: Figure 3
Scale
N.T.S.
Project Title
FINDLAY CREEK VILLAGE
SWM FACILITY
Drawing Title
COMPARISON OF DESIGN
VERSUS AERIAL VIEW
Sheet No.
FIGURE 3

catchment areas is directly released to the network of storm sewers, called the minor system. During less frequent storms, the balance of the flow is accommodated by a system of swales and street segments called the major system. The main advantage of this drainage arrangement is its ability to adjust the rate of total inflow into the minor system.

A typical example of a dual drainage (major system) model schematic and rendering of the simulated result are illustrated on Figure 4 and Figure 5 [7]. In its design and analysis of urban dual drainage systems, IBI Group looks at land use and grading plans and considers local design constraints and regulatory requirements.

IBI Group typically conducts fully dynamic surcharge analysis to evaluate the performance and design of the minor system (trunk storm sewers) and reservoirs (such as SWM facilities) and associated flow splitter, inlet and outlet structures, taking into account downstream boundary conditions. A typical example of fully dynamic model schematic is illustrated on Figure 6 [8].

5.3 Water Quality and Erosion Control

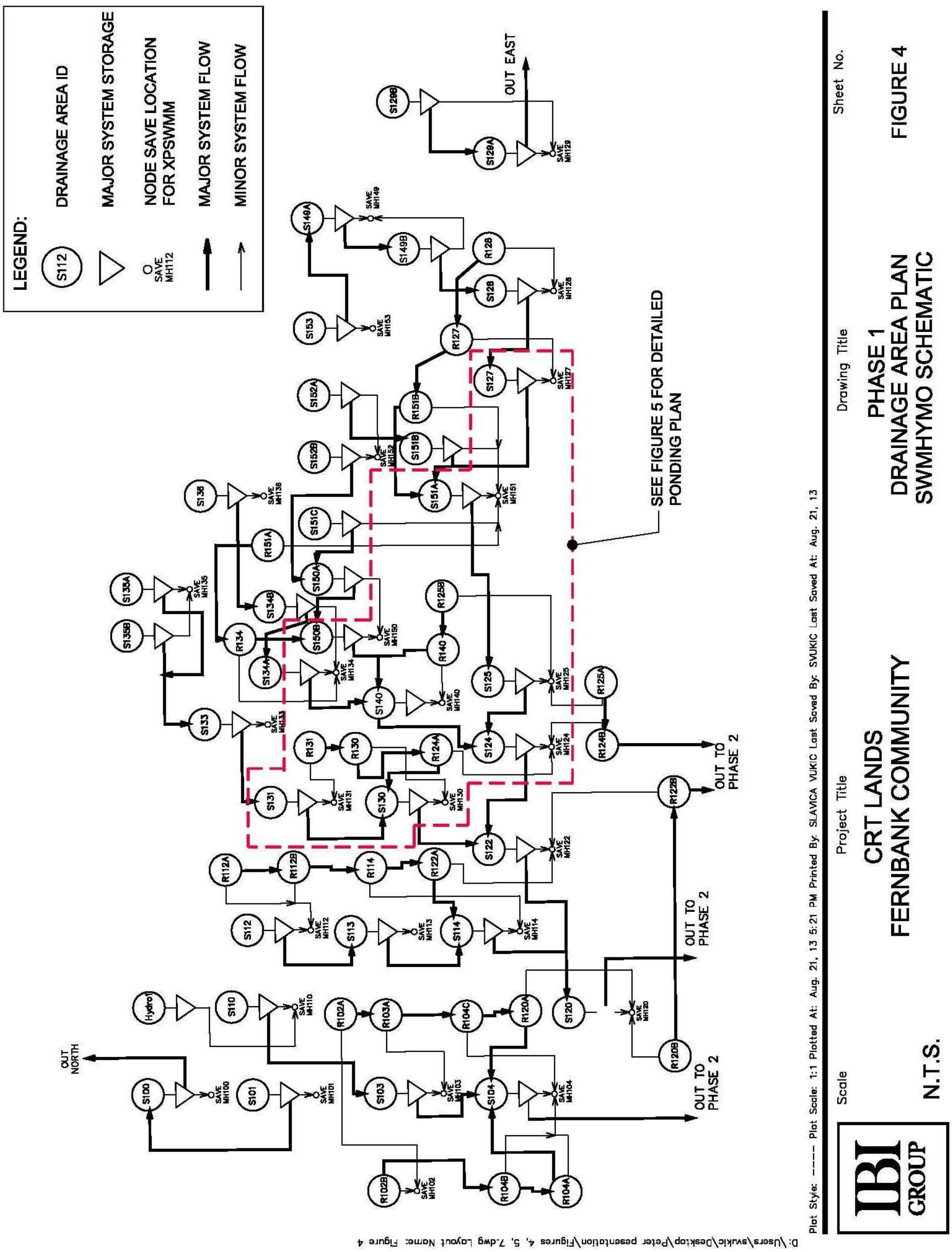
Changes to the hydrologic cycle as a result of urban development are noticeable not only in quantity of stormwater runoff, but also in quality. The deposition of pollutants on impervious surfaces and subsequent precipitation and wash-off of such accumulated deposits affects the quality of stormwater runoff. An increase in sediment load is a key factor affecting water quality and contamination of downstream watercourses.

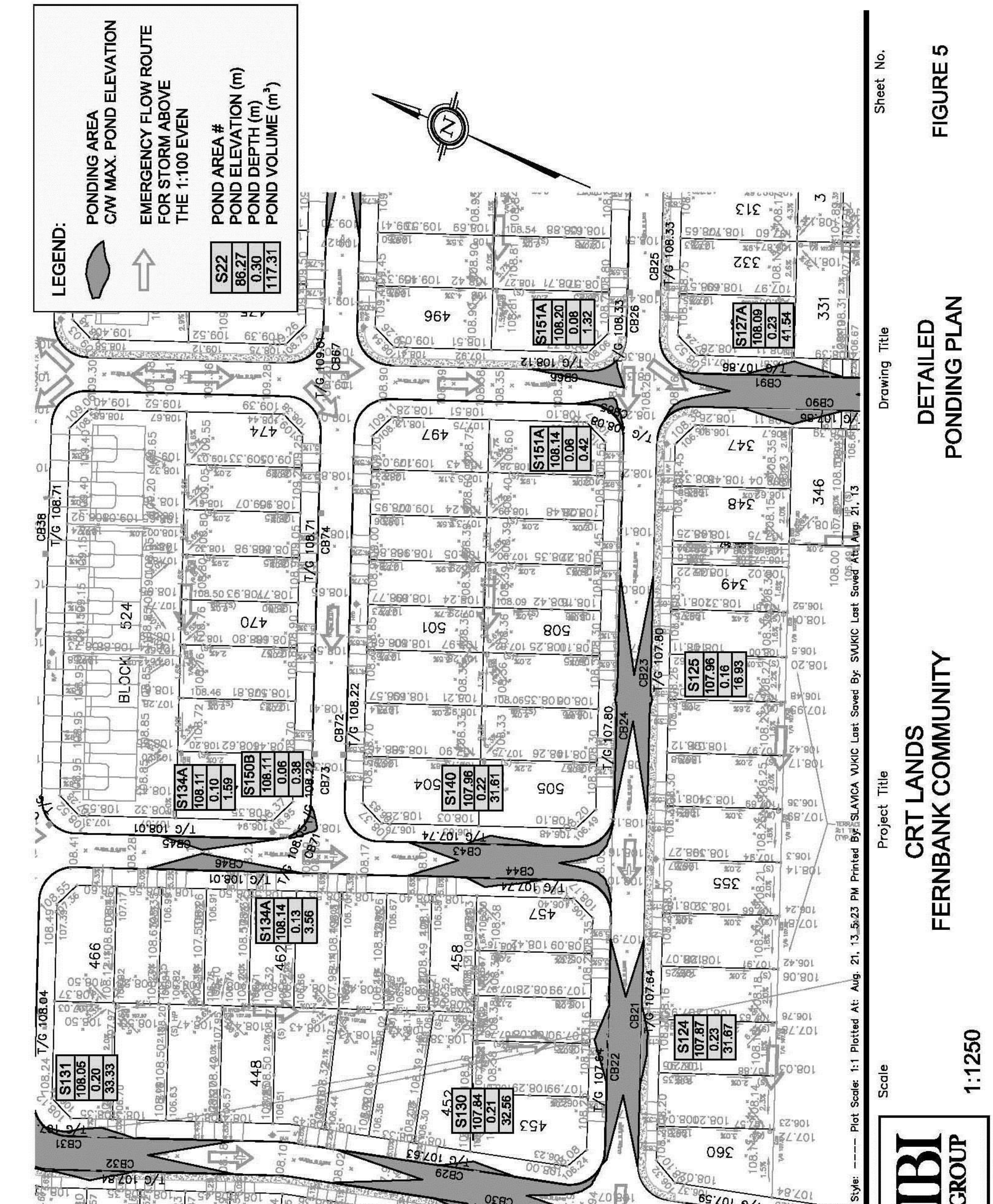
IBI Group also evaluates the effects of urbanization from an erosion perspective. In general, urban development impacts natural channels by accelerating rates of erosion or deposition. IBI Group conducts erosion analysis to identify potential changes in in-stream erosion of receiving watercourses due to urbanization, and to develop mitigation measures, if required. Erosive impacts are simulated with both single storm event and continuous modelling techniques. Typical evaluation consists of shear stress analysis on a continuous basis.

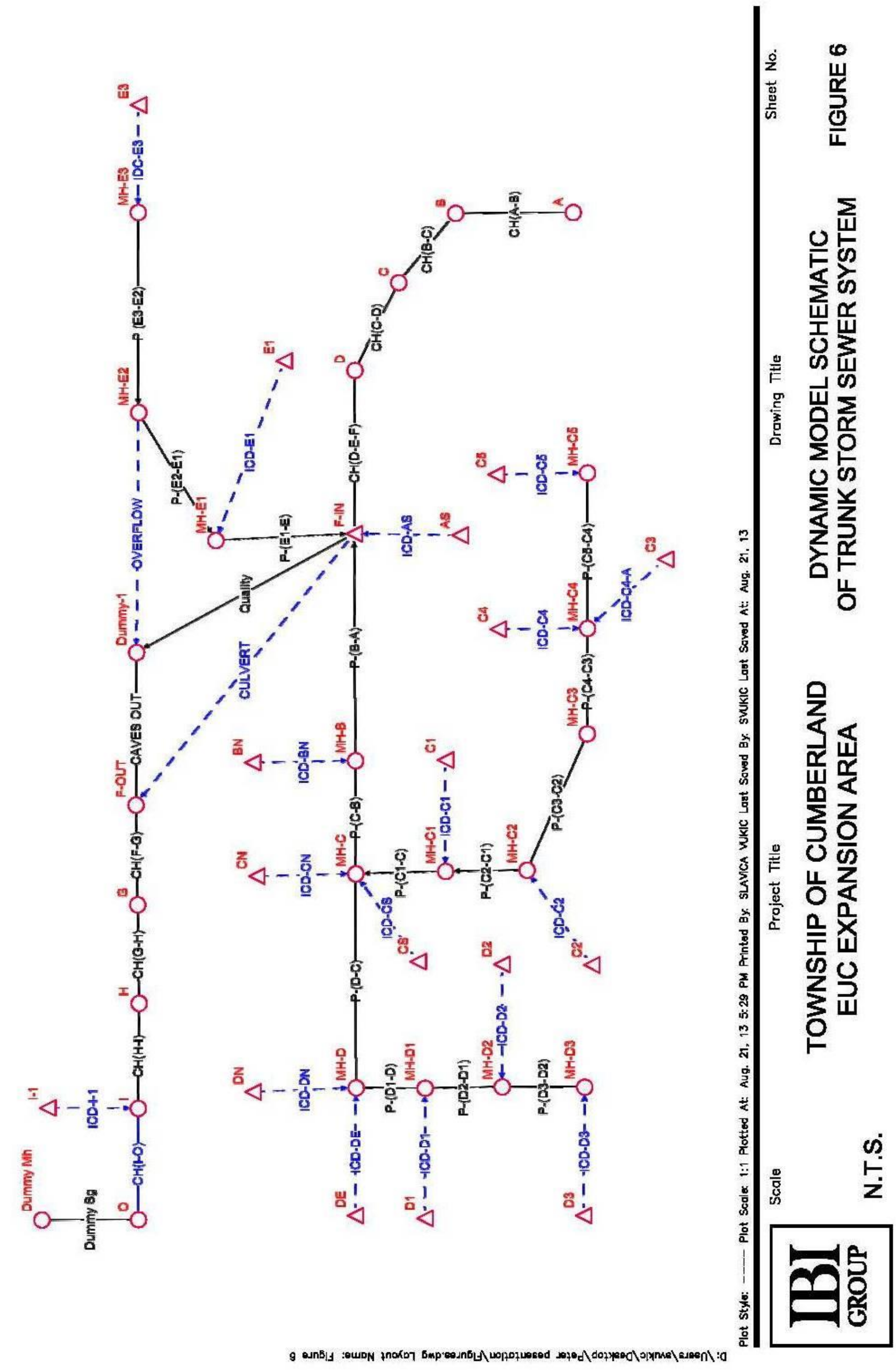
IBI Group evaluates a system’s functionality for pollutant removal using a continuous modelling technique. The model has the ability to generate a flow and pollutant series (such as suspended solids) in a continuous hydrologic regime, under both existing and post-development conditions. The continuous model incorporates several algorithms for pollutant supply (build up), delivery mechanisms (wash off to storm sewers) and routing (through stormwater management or treatment facilities). Precipitation data is based on several decades of observation and statistical analysis. IBI Group selects typical wet, average and dry years for evaluation. Figure 7 presents a typical example of inflow and outflow pollutographs from a SWM facility [9].

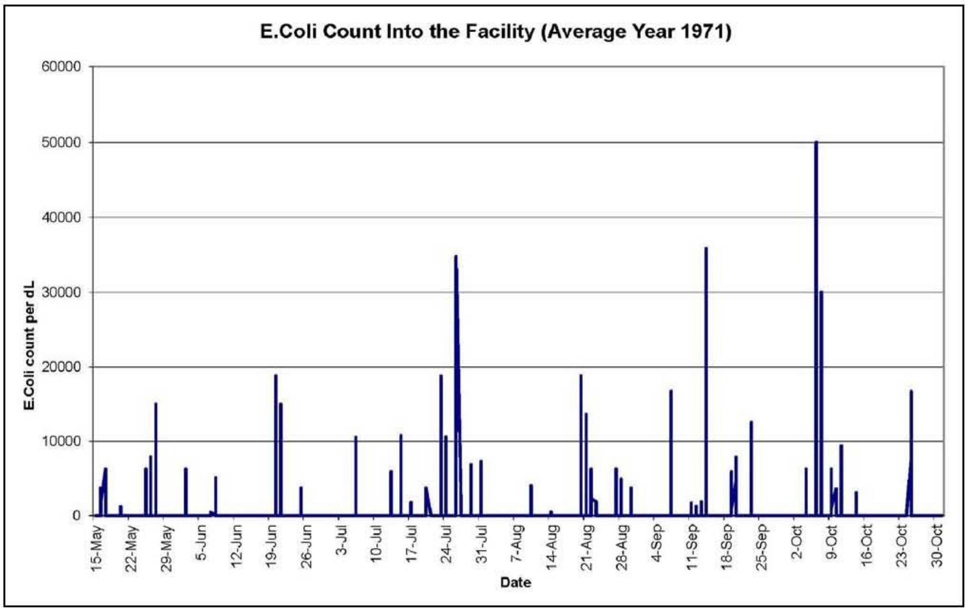
6 PRACTICAL EXAMPLES OF END-OF-PIPE SWM FACILITIES

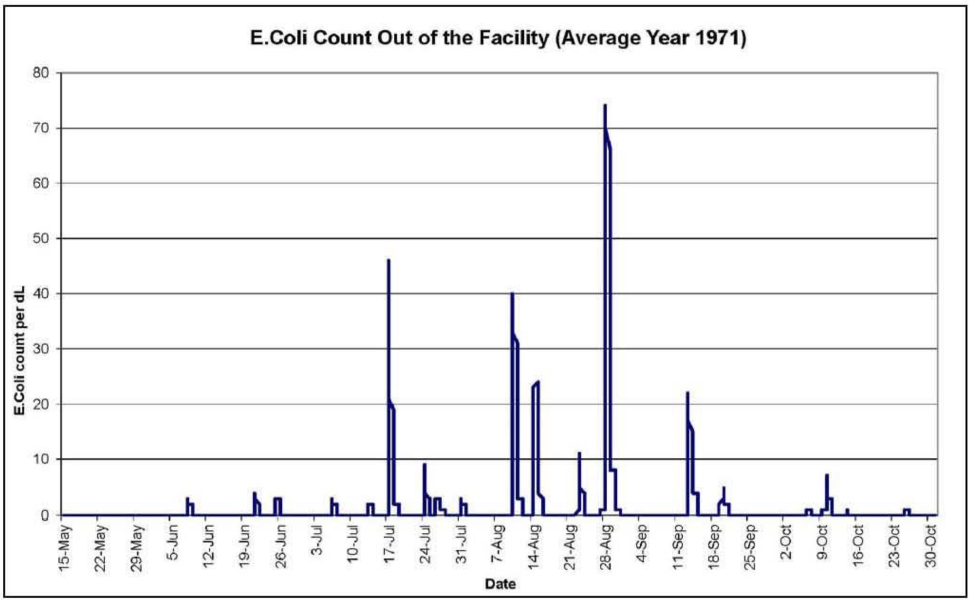
IBI Group is a leader in SWM facility design, having produced state-of-the-art facilities in Ontario since the early 1990s. Throughout the design process, IBI Group takes into account local regulatory requirements, environmental factors, and the function of the proposed development, all while ensuring a hydraulically-efficient design. Generally, SWM facilities consist of three major components, outlined in the below sections. The general layout of an end-of-pipe stormwater facility is illustrated in Figure 8 [3].


LEGEND:
S112
DRAINAGE AREA ID
MAJOR SYSTEM STORAGE
SAVE MH112
NODE SAVE LOCATION FOR XPSWMM
MAJOR SYSTEM FLOW
MINOR SYSTEM FLOW
OUT EAST
OUT NORTH
OUT TO PHASE 2
SEE FIGURE 5 FOR DETAILED PONDING PLAN
Scale
N.T.S.
Project Title
CRT LANDS
FERNBANK COMMUNITY
Drawing Title
PHASE 1
DRAINAGE AREA PLAN
SWMHYMO SCHEMATIC
Sheet No.
FIGURE 4
IBI GROUP


LEGEND:
PONDING AREA
C/W MAX. POND ELEVATION
EMERGENCY FLOW ROUTE
FOR STORM ABOVE
THE 1:100 EVEN
POND AREA #
POND ELEVATION (m)
POND DEPTH (m)
POND VOLUME (m³)
S22
86.27
0.30
117.31
Scale
1:1250
Project Title
CRT LANDS
FERNBANK COMMUNITY
Drawing Title
DETAILED
PONDING PLAN
Sheet No.
FIGURE 5


TOWNSHIP OF CUMBERLAND
EUC EXPANSION AREA
DYNAMIC MODEL SCHEMATIC
OF TRUNK STORM SEWER SYSTEM
FIGURE 6
N.T.S.
IBI GROUP





D:\Users\svukic\Desktop\Peter pesentation\Figures 4, 5, 7.dwg Layout Name: Figure 7

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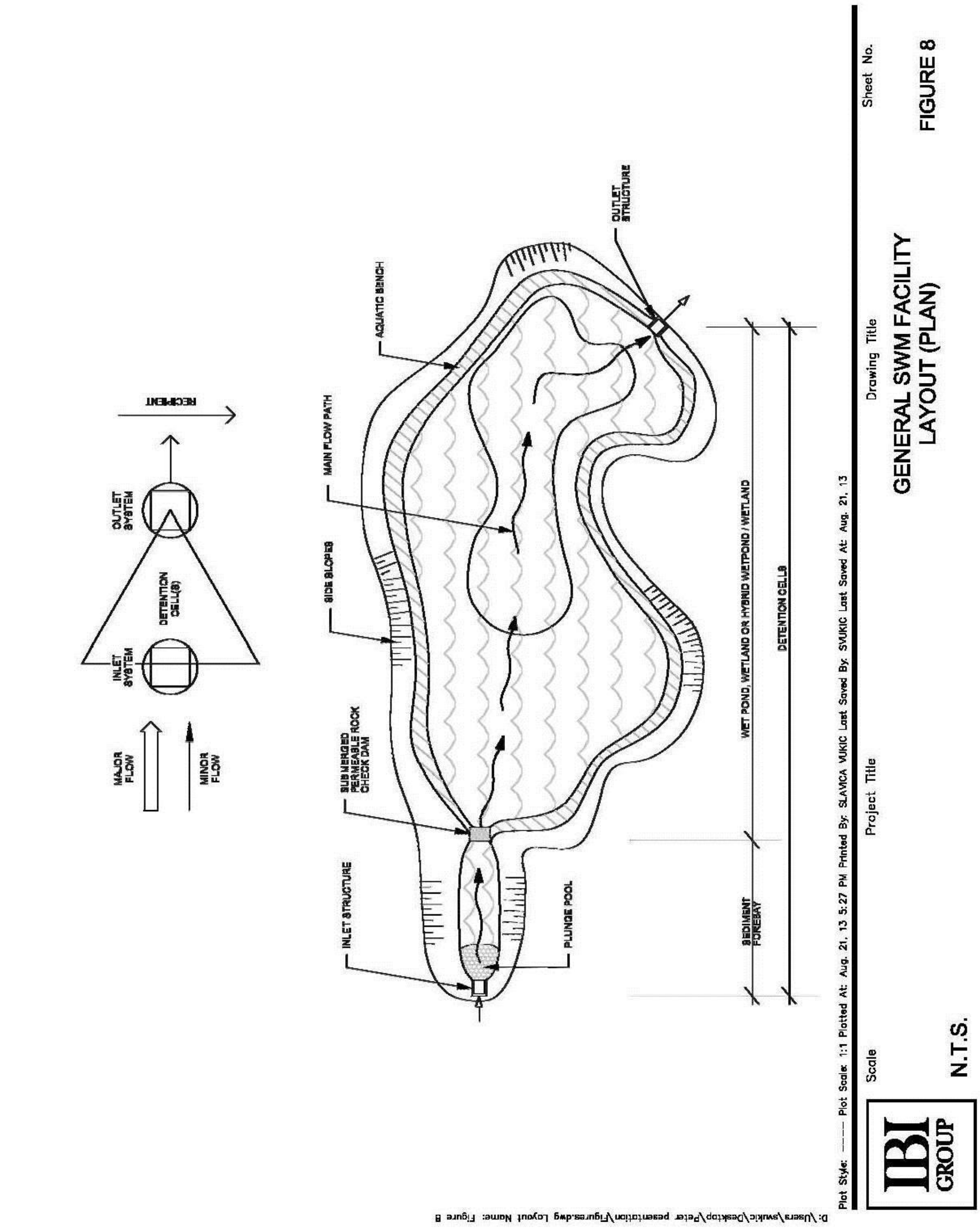
IBI GROUP

N.T.S.

TYPICAL EXAMPLE OF INFLOW AND OUTFLOW POLLUTOGRAPHS

FIGURE 7


MAJOR FLOW
MINOR FLOW
INLET SYSTEM
DETENTION CELL(S)
OUTLET SYSTEM
RECIPIENT
INLET STRUCTURE
PLUNGE POOL
SUBMERGED PERMEABLE ROCK CHECK DAM
SIDE SLOPES
MAIN FLOW PATH
AQUATIC BENCH
OUTLET STRUCTURE
SEDIMENT FOREBAY
WET POND, WETLAND OR HYBRID WETPOND / WETLAND
DETENTION CELLS
Plot Style: —— Plot Scale: 1:1 Plotted At: Aug. 21, 13 5:27 PM Printed By: SLAVICA VUKIC Last Saved By: SVUKIC Last Saved At: Aug. 21, 13
IBI GROUP
Scale
N.T.S.
Project Title
Drawing Title
GENERAL SWM FACILITY LAYOUT (PLAN)
Sheet No.
FIGURE 8

6.1. Inlet Systems

Inlet systems convey stormwater runoff to the detention cells. More sophisticated minor inlet systems are provided with flow splitters. Their function is separation and re-direction of a portion of the runoff to meet multiple design objectives of SWM facilities.

6.2 Detentions Cells

Detentions cells are provided with a permanent storage and extended detention storage. Based on the overall layout of the permanent storage and its biological functions, the following are different types of detention cells:

- sediment forebay;
- wet pond;
- wetland; and,
- hybrid wet pond/wetland.

As illustrated on Figure 8, runoff is first discharged from the inlet structure to the sediment forebay, the purpose of which is to provide sediment removal of the runoff. From the sediment forebay, the runoff is discharged to the downstream cell (wet pond, wetland or hybrid wet pond/wetland) to complete treatment.

6.3 Outlet Systems

The main function of an outlet system is to release and convey the attenuated water from the detention cell to a recipient. An outlet system consists of outlet structures, outlet sewers/channels and headwalls. The primary function of the outlet structure is the release of the attenuated water via the outlet sewers/channels, providing conveyance to the recipient.

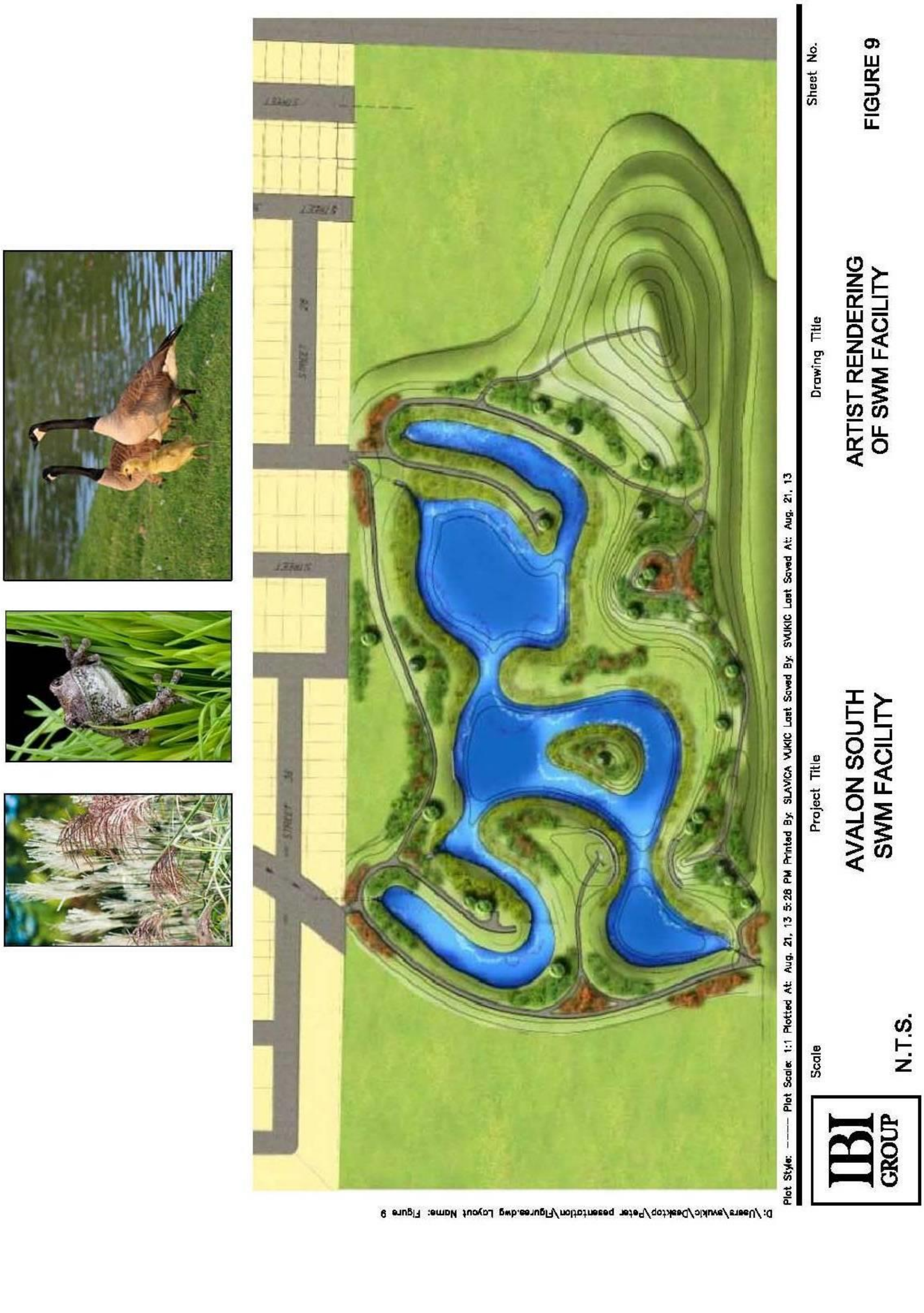
6.4 Example: Avalon South Stormwater Management Facility

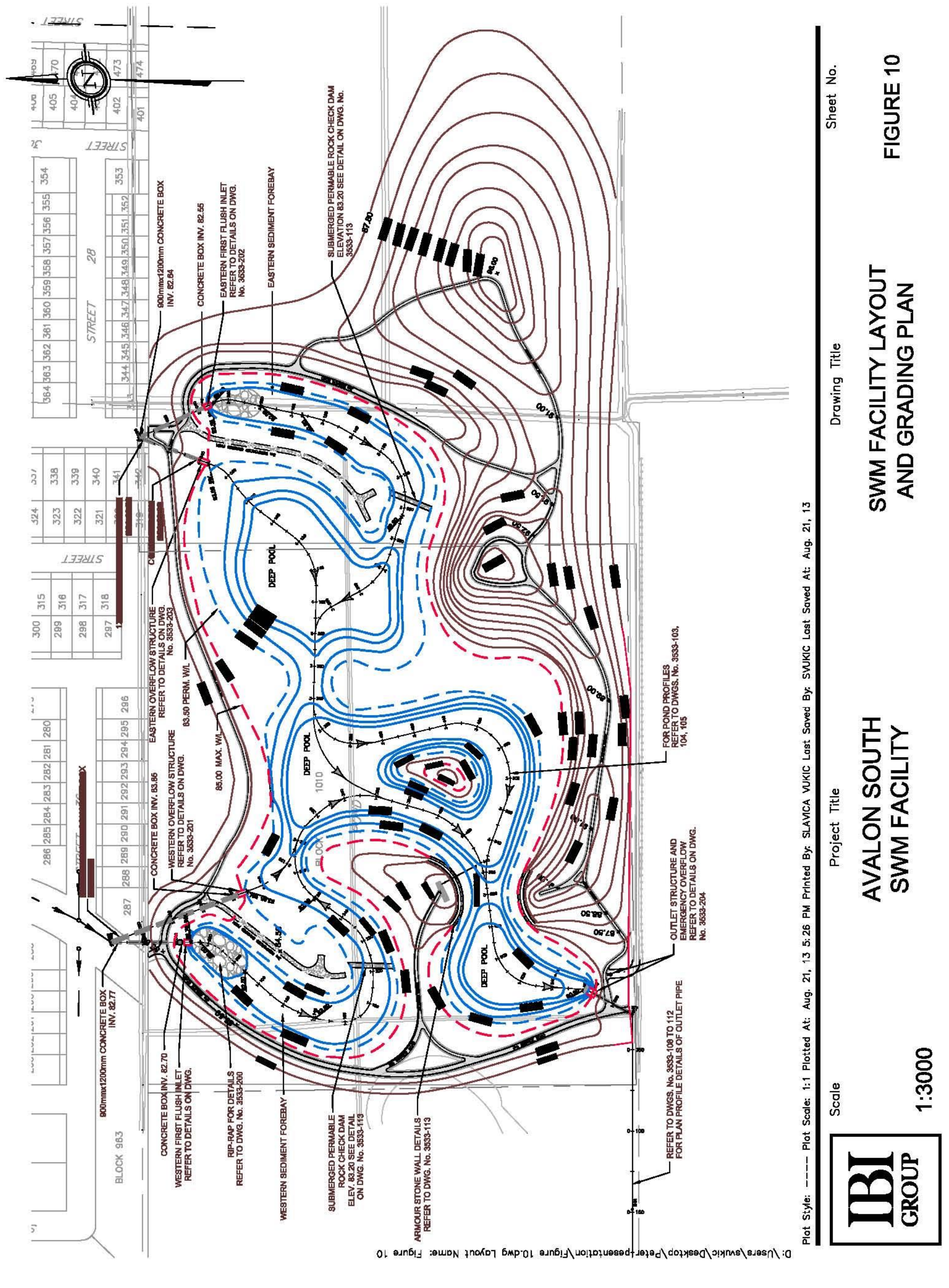
The Avalon South SWM Facility, a hybrid wetland/wet pond, was designed to service approximately 160 ha (400 acres) of urban residential development and provide water quality and quantity control. Located outside the urban boundary, the facility is an extension of the community and provides a natural area for wildlife habitat and recreational pathways.

The total volume of the SWM facility is 83,000 m^3 and the permanent volume accounts for approximately 64,000 m^3. The facility is equipped with two inlet systems with flow splitters and two sediment forebays. The facility has deep pools separated by shallower regions with peninsulas that serve as lookouts over the facility. The facility outlets via a storm sewer pipe approximately 1 km (0.6 miles) in length to a discharge point at McKinnons Creek. The SWM facility construction was completed in 2010 at a cost of $5 million. The artist rendering of the SWM facility is indicated on Figure 9 and the design grading plan is illustrated on Figure 10 [10].

7 DISCUSSION AND CONCLUSIONS

Over the last two decades stormwater management in Ontario, Canada has evolved to a comprehensive strategy protecting the natural attributes of the watershed. Leading regulatory authorities in Province of Ontario are Conservation Authorities (CA’s) and the Ontario


Sheet No.
FIGURE 9
Drawing Title
ARTIST RENDERING
OF SWM FACILITY
Project Title
AVALON SOUTH
SWM FACILITY
Scale
N.T.S.
IBI
GROUP


Project Title
AVALON SOUTH
SWM FACILITY
Drawing Title
SWM FACILITY LAYOUT
AND GRADING PLAN
Sheet No.
FIGURE 10
Scale
1:3000
IBI GROUP
DEEP POOL
WESTERN SEDIMENT FOREBAY
EASTERN SEDIMENT FOREBAY
83.50 PERM. W/L
85.00 MAX. W/L

Ministry of the Environment (MOE). In some cases, the Federal Government becomes involved in a particular subject area due to crown properties.

The urban development is generally investigated on a subwatershed basis and through Master Servicing Studies. Potential development scenarios are envisioned by municipalities and harmonized with environmental attributes in the watershed and the required stormwater management concepts are incorporated in particular Municipal Official Plan Amendments (OPA).

In the Province of Ontario, guidance for the planning and design of stormwater management systems was developed by the MOE and published in Stormwater Management Planning and Design Manual (March 2003) [2]. The main aspects of the regulatory requirements are related to water balance, water quality, erosion/geomorphology and water quantity. To achieve the required regulatory requirements, the recommended stormwater management strategy is to provide an integrated treatment train approach by providing control at the lot level, during conveyance and at the end of the pipe.

The City of Ottawa is one of the first municipalities in Ontario which adopted comprehensive stormwater management principles. Presently, the City owns approximately 300 SWM facilities. Since 1990 until the end of 2012, the population growth within the City of Ottawa has increased from 678,000 to 1,000,000. The urban development area land consumption within the City limits has increased to over 2000 ha (4942 acres) within the same time period. The adoption of the stormwater management principles in urban development has played a major role in the protection of the natural environment and contributed to overall development sustainability.

In the City of Ottawa, IBI Group has been a leader in providing water resources services, hydrological modelling and design of state-of-the-art SWM facilities since the early 1990s. Practical examples have been presented which focus on applications related to hydrological and hydraulic modelling as well as the design of a relatively large end-of-pipe SWM facility, Avalon South, constructed in 2010.

IBI Group has been a major contributor to the adoption and development of stormwater management principles and designs in the City of Ottawa. At the present time, IBI Group has been completing the Stormwater Management Facility Design Guidelines and Standards [3]. Its primary intent is to provide assistance for local engineers designing SWM facilities. The audience for the Guidelines also includes public agencies such as the City of Ottawa, conservation authorities, the Ontario Ministry of the Environment and other agencies concerned with land use, development, and the management of urban runoff. Some parts of the Guidelines can be used for educational purposes.

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[2] Ontario Ministry of the Environment: *Stomwater Management Planning and Design Manual.* 2003.

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[10] IBI Group: Design Brief Avalon South (Neighbourhood 4) Stormwater Facility. April 2006.

SATELLITE TECHNOLOGIES IN WATER MANAGEMENT

Š.Stanko[1] and I.Škultétyová[2] and M.Holubec[3]

Abstract

Satellite technologies have an wide impact on human activity. Not only weather observing, navigation exploitation is the main purpose of satellites. Much more type of exploitation is possible. Very important role concerns the global social, economy and weather security. The satellite technologies discovered us the undiscovered history of the Earth, show us the environment on global level. The water generally present the basic raw material of the life and the satellites allow us to understanding it better, to protect it to view our future with various scenarios of human behaviour concerning the water management. The paper focuses on the satellite exploitation from various views on water, on the aspects for monitoring, measure the engineering network a evaluating the data concerning the global warming and climate change.

Keywords

GIS, Satellite, Sewage, Water management, Water Supply

1 INTRODUCTION

The focusing on the water management exploitation started more than twenty years ago. Now we are able to measure the global water data through the Earth concerning the water level, water temperature, sea water flow etc. This information gives us the useful information for not only human protection but environment entire.

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[3] Ing. Michal Holubec, Slovak University of Technology in Bratislava, Faculty of Civil Engineering, Radlinského11, 81368 Bratislava, +421-2-59274282, michal.holubec@stuba-sk

The water management in the history exploited only the traditional technologies based on the ground observation. To achieve the global water monitoring over the world in the same time was impossible. The reason was too many observation stations, which localization depended on the land place possibility. Except the technical equipment possibilities and the cost investment and operation there was no opportunity to obtain the real data in the “same” time.
The satellite technologies, which are extremely improved in the last decades allowed uses the new approach in the world water observation. The system of satellite are not useful only for the GPS navigation, but various satellite systems allow the measure water capacity, the life dependence on the world water, on the seasonal changes and everything is possible to evaluate by the computer equipment immediately.
The exploitation of the satellite systems such as GPS, GLONAS or the new built up European system GALILEO allowed us to positioning measurement and the special satellites such as SEWAVES, AQUA, AURA, SEASAT, TERRA, GRACE etc. allowed us to measure various parameters concerning the water in the world with the aim to clever exploitation of the water, of the climate change prediction and reducing the negative human activity.
The climate change has the influence on global warming which affects the weather changes, extremes in the weather and have the future negative affect on the thermohaline world water cycle. The measure of the snow covering of the Greenland Island by two systems: GRACE (measure of annual ice decrease - weight 183GT) and ICESAT - laser (annual ice decrease weight 160 GT) give us the relatively exact information about annual decrease of the ice and allow to estimates the influence on climate change. The measure of the sea level by the satellite GRACE showed us, that in consequence on Earth gravity in dependency on sea deep, the sea level have not always reach the level zero, but there are the extremes plus 130 m and minus 80 meter. It means that sea level in the world context is not the flat. Satellite are exploitable in the rainfall measurement or water capacity e.g. in the Brazil rainforest which can help the scientist to obtain the exact information of world water cycle. Satellite technologies become very important in global and local water management.
The water management in the history exploited only the traditional technologies based on the observation on ground level. In the better case form the plane. To achieve the global water monitoring over the world in the same time was impossible. The reason was too many observation stations, which localization depended on the land place possibility. Except the technical equipment possibilities and the cost investment and operation there was no opportunity to obtain the real data in the “same” time.
The satellite technologies, which are extremely improved in the last decades allowed uses the new approach in the world water observation. The system of satellite are not useful only for the GPS navigation, but various satellite systems allow the measure water capacity, the life dependence on the world water, on the seasonal changes and everything is possible to evaluate by the computer equipment immediately.

2 SATELLITE EXPLOITATION

Applications of satellite technology are exploitable for:
- Field of water resources - generally
- Measurement and control discharges of water, of water consumption, the flow rate in rivers

Monitoring of:
- agriculture in connection with the water ensuring ≈ agricultural fertility control water ≈ water consumption

- water quality in the lakes, rivers, sea and oceans
- weather conditions, temperature change, observing the ice mass.

Systems are useful only on condition of mutual interaction of ground technological equipment, or by Earth remote sensing - using electromagnetic radiation.

2.1. WARNINGS STATES

The one of the useful rule of satellite is the monitoring and the system of earlier alerting of water generally. The many types of alerts are possible for monitoring. The very important in last time become the tsunami warning system (TWS) which is very important in regional and international importance. On a larger scale tsunami warning systems are currently of high interest.(FIG, 2006) For example the GeoForschungsZentrum Potsdam (GFZ) will co-develop a part of the IOTWS (Indian Ocean Tsunami Warning System) near Indonesia. (Fig.1.) This development is a German-Indonesian cooperation called GITEWS (German Indonesian Tsunami Early Warning System) granted by the German government (BMBF 2004) [2] .

The rule of monitoring is receive immediate alerts when safety thresholds are reached to control flooding or react to changes in water quality, reduce costs by knowing when expensive equipment may be flooded.

Process of data communications through satellite includes:
• Download complete data from data loggers and remote metres
• Improve data integrity by receiving data often and from a single source
• Reduce costs by eliminating the need to send technicians to remote locations

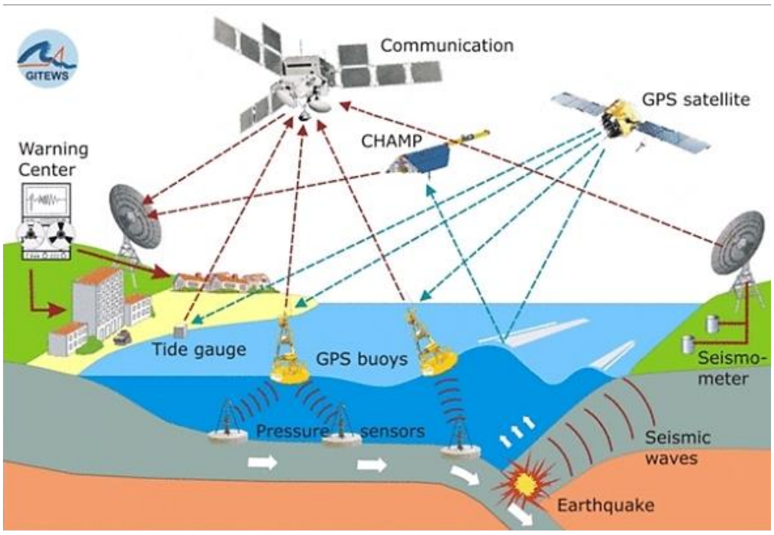

Fig. 1. Tsunami warning system – TWS, [12]

The system will integrate terrestrial observation techniques like seismometers and tide gauge measurements by GPS as well as marine measurements on GPS buoys and with ocean bottom pressure sensors and the processing centre in Indonesia. The base is the already available global earthquake monitoring system of GFZ and its also available real-time communication technique. Overall the system consists of four chain links: the data acquisition, the data processing, the validation and the warning component. The final implemented system will have an open and modular character to ensure the possibility to be further enlarged without problems.

2.1. Water vapour measurement

Because there is the theory that water vapour is the most important factor influencing the greenhouse effect but doesn’t feature on the UN’s list of greenhouse gases responsible for anthropogenic global warming, the vapour measurement and evaluation concerning the climate change became important.

Over the world are installed more than hundred GPS/MET low –Earth orbiting satellites –cca 775km orbital altitude, GPS 20,231 km which provides more than 4 thousand global soundings of temperature and water vapour each hour.

The Radio Occultation (RO) technique exploits the correspondent phase and amplitude deformations affecting the GPS signal due to its propagation through the terrestrial atmosphere, during the rising or setting of the GPS satellite above the limb. This technique will provide accurate measurements of the atmospheric refractive indexes from which it is possible to derive atmospheric vertical profiles of temperature, pressure and humidity, as well as profiles of electron content in the ionosphere. The application of the Radio Occultation technique to sound the Earth's atmosphere needs the presence of transmitting sources like the satellites of the GPS (Global Positioning System) constellation.

The Radio Occultation is a technique for sounding the structure of planetary atmospheres developed over the last 35 years at Jet Propulsion Laboratory and at the Stanford University. (Agenzia Spaziale Italiana) [1] To sound the planetary atmosphere, the technique is based on deriving the refractivity index from radio signals perturbations transmitted by space probes while a satellite is moving around the planet. The application of the Radio Occultation technique to sound the Earth's atmosphere is strongly linked to the GPS constellation development. The GPS Radio Occultation observations were used for the first time to study the Earth's atmosphere in 1995 during the Global Positioning System/Meteorology (GPS/MET) mission. This mission provided an active sounding of the atmosphere under all-weather conditions and with relatively high vertical resolution.

A GPS Radio Occultation occurs when a transmitting GPS satellite, setting or rising behind the Earth's limb, is viewed by a LEO satellite. (Fig.2.)

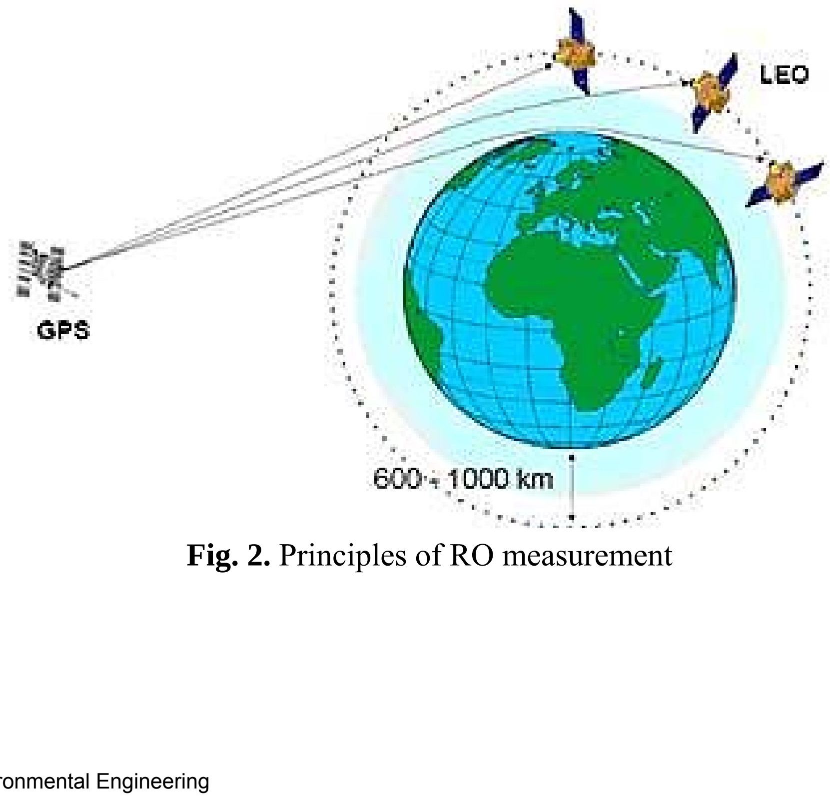

Fig. 2. Principles of RO measurement

2.1. Sewage flow measurement/monitoring

If we want to evaluate the sewage behaviour, we need to know the processes, which occur in the sewage in the various conditions, mean the wet-weather, dry weather and flood time. The experiences of the sewage monitoring showed us, that this monitoring is really very complicated. We can exploit the ground data transfer or the mobile data transfer, but always there are the problems with the connectivity. The new approach of satellite exploitation provides us the possibility to exploit this technology in the large city areas.

The CSO (Combined Sewer Overflow) monitoring gives us the information about the number of overflow, the amount of the water, which load the recipient. The advantage is in the large area monitoring and mobile operator independent.

If we need to simulate the process of surface runoff, flow simulation in sewer network by mathematical modelling, the exact data about the network are necessary. The accuracy is reached with the satellite and GPRS technology combination and exploitation. The technology is very successfully used in sewage and water supply measurement. (Fig. 5)

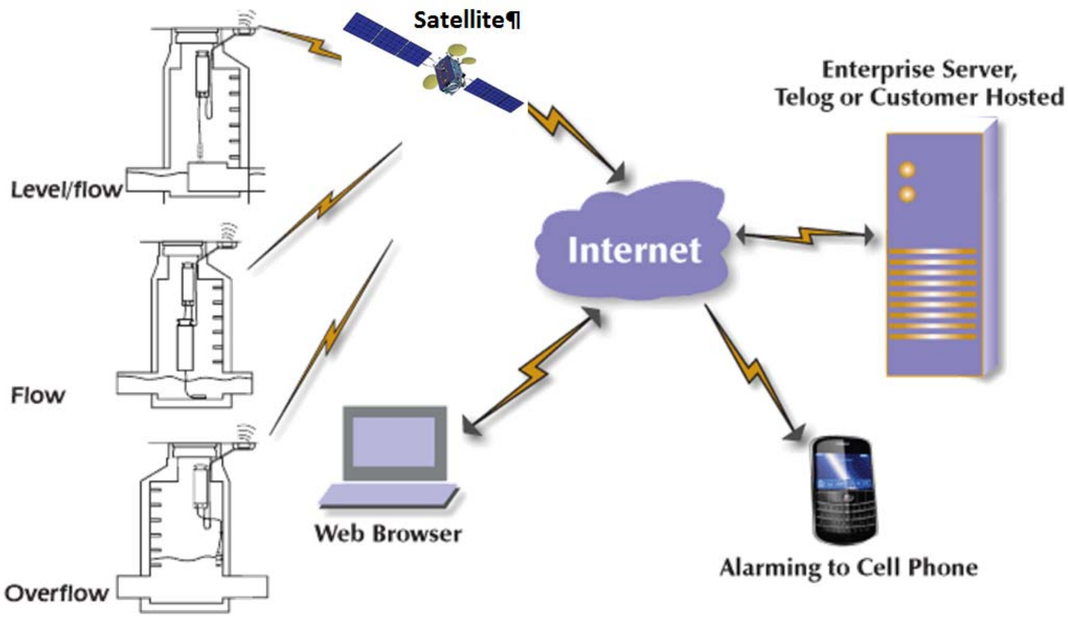

Fig. 3. Sewage monitoring

2.1. Slovakia – Smartnet system

SmartNet is the first private commercial network of permanent GNSS reference stations with nationwide coverage. It consists of 24 stations (Fig. 4) (11/2010) equipped with precise GNSS receivers from Leica that support GPS signals and GLONASS frequencies on L1 and L2 (including L2C) and are ready to further expand the support of three-frequency signals, such as Galileo and GPS L5. (SmartNet Slovakia, 2012) [6]

All stations are sending continuously measured data through the Internet to a common server with installed software-Leica Spider, which further ensures their processing and the creation and delivery of RTK and DGPS products for users, SmartNet. These products -

differential correction - are accessible to users on the ground via data services, mobile operator (GPRS or UMTS).

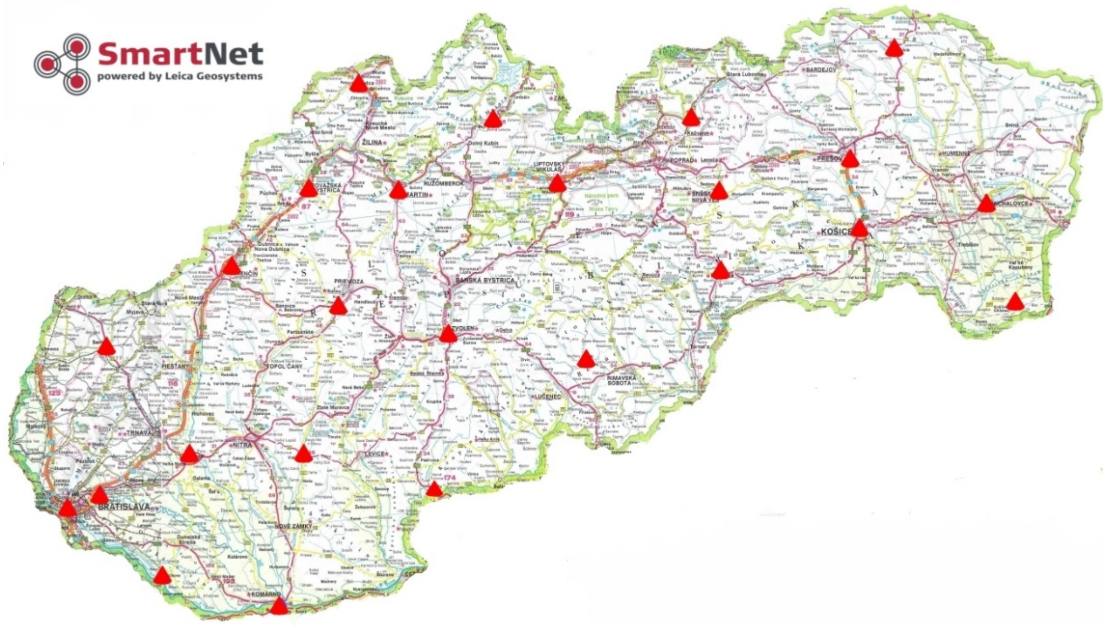

Fig. 4. Slovakia – coverage of 24 SmartNet station

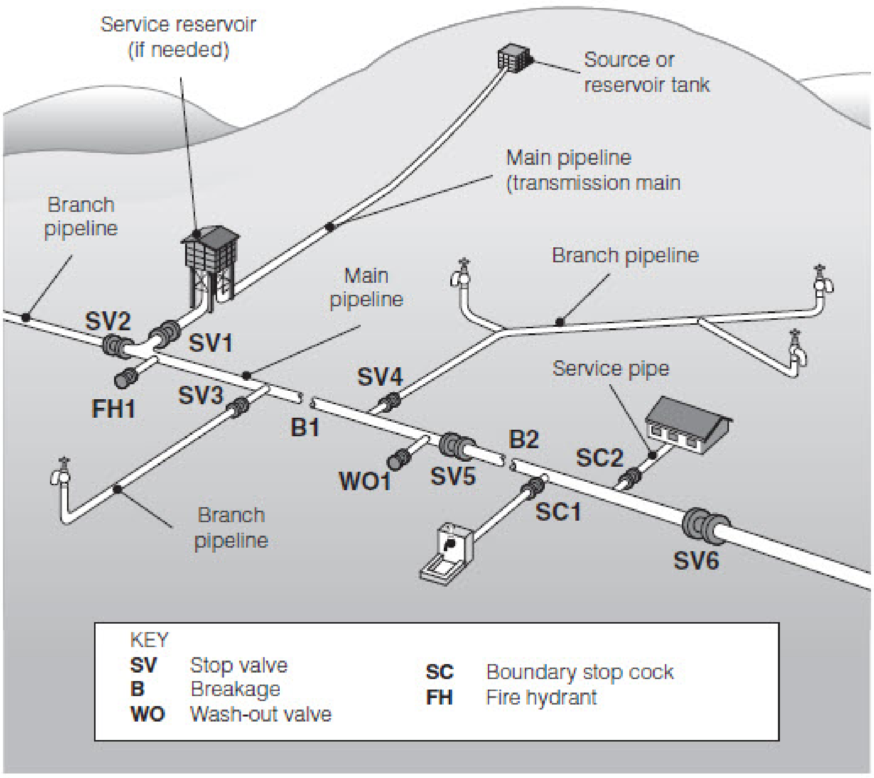

Fig. 5. The SmartNet data are exploitable in GIS systems

SmartNet System description (Fig. 6):

Rover and reference station network receive signals from the same satellites

Network server base on the nearby reference stations rover position generate the virtual station so that its position was as close rover position in the field. Network rover calculates and sends virtual corrections, which are optimized for the rover position at the beginning of the measurement.

Virtual corrections are sent to the rover.

Rover accepts correction as if they were emitted from a single reference station nearby and applies them to its measurements.

The system accuracy depends on the receiving station and reaches 2 cm.

Figure (Fig. 7), (Fig. 8) shows the situation and longitudinal profile, which was recalculated by the software SeWaCAD, with the goal to obtain the hydraulic information about the sewer system. The system contains five CSO structure. The problem with the elaboration was the incorrect – not exact data suitable for mathematical model. Because there was short time for geodetic exact measurement, we used the system GPS for the data measurement with exploitation the SmartNet system. The data were completed together with the combination of the very old project, with some parts of geodetic measurement and satellite data. There was used the Leica equipment for the data gaining.

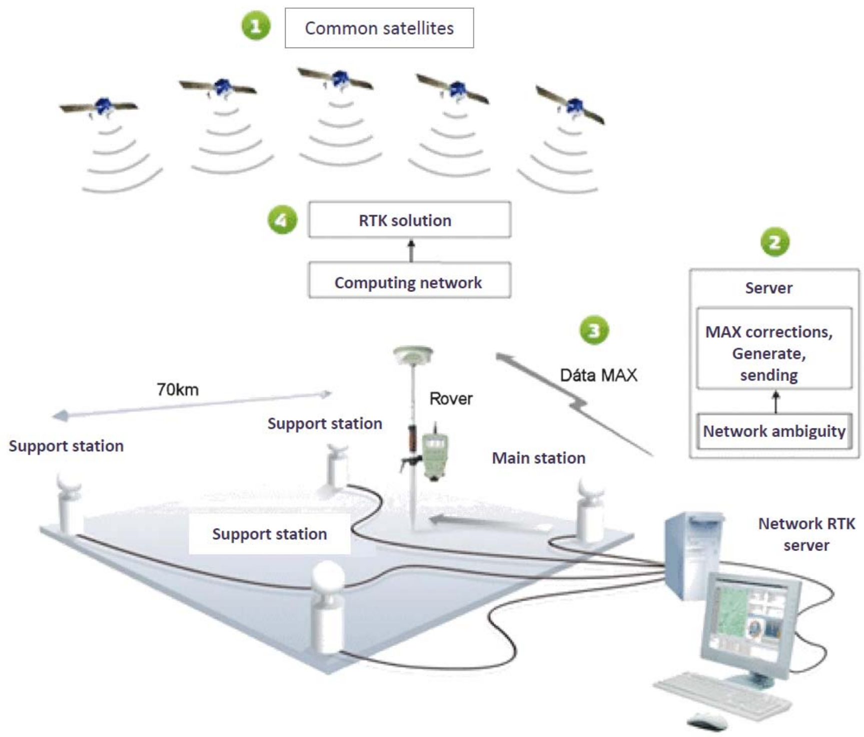

Fig. 6. The relationship between the network server and RTK Rover with VRS concept

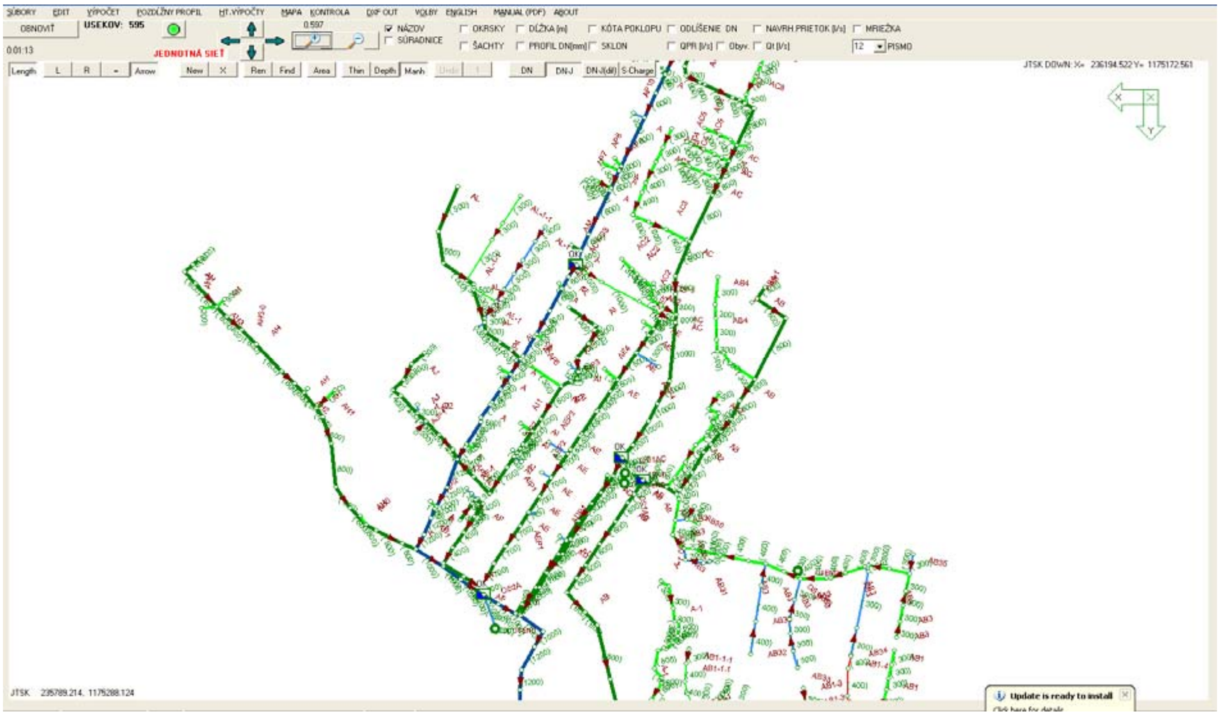

Fig. 7. Situation of the city SVIDNIK-Slovakia – passport of existing sewer network

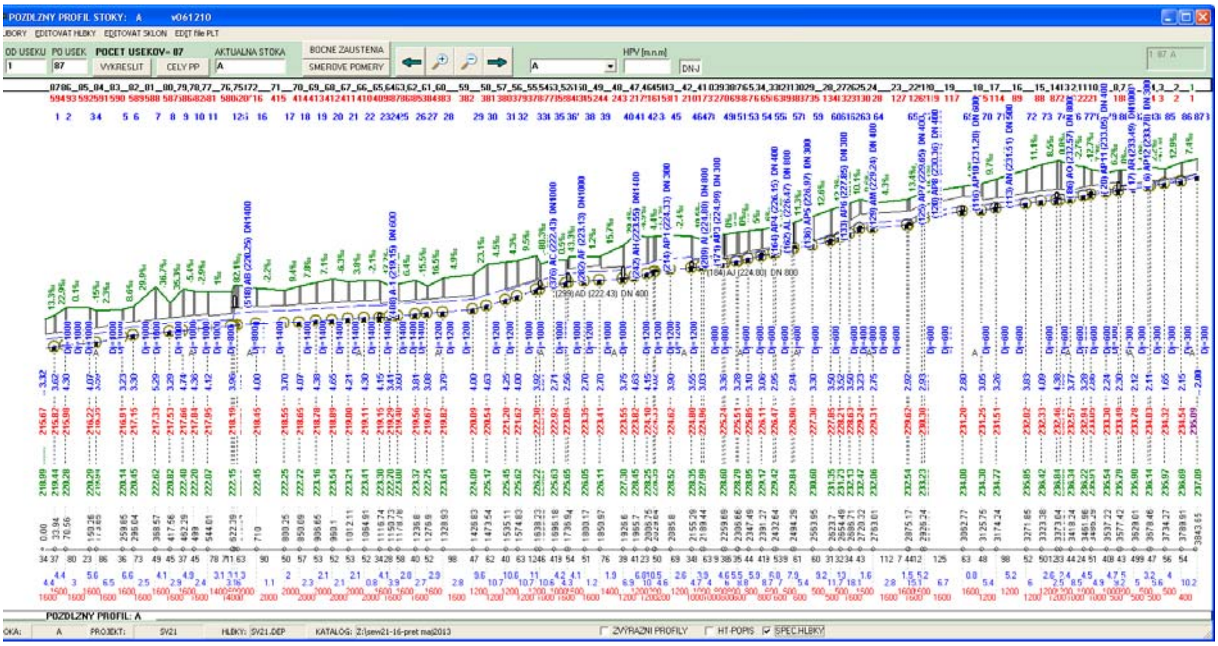

Fig. 8. Longitudinal sewer profile of the SVIDNIK city in Slovakia – passport of existing sewer network

2.1. Quantification of damages - measure the flooded area

The last years convinced us, that the floods caused by several nature impact can imposed the huge damage of the human activity. To estimate the economy, social and commercial impact is possible by the satellite images comparison in the time. (Fig. 9).

The study of Asahi City in Japan showed that the combination of satellite images and their evaluation through the numerical model can bring the estimation of flooded area and estimated the damages. Base on this there are the possibility how to protect the dangerous areas against the flood influence.

In this study, tsunami-inundated areas in Asahi City, Chiba Prefecture (Fig. 10), were identified on the basis of the map compiled by disaster relief volunteers, the interpretation of satellite images, and a numerical simulation of tsunami propagation. (K.Kitamura et all,2012) [8] The relationship between the fraction of totally collapsed buildings in Asahi City and the hydrodynamic features of tsunami inundation flow, obtained from the numerical simulation, was determined.

The GIS dataset for tsunami-inundated areas was constructed by gathering information from different resources and conducting interviews with the residents of the affected areas. It was observed that the tsunami run-up reached a height of approximately 5 m in Asahi City, Chiba Prefecture (Fig. 10, Fig. 11). The seawalls were not very effective; however, tide prevention forests might be effective in containing the tsunami waves in some areas.

Fig. 9. Satellite images of the coast of Japan - 2011 before and after the tsunami.

The name consists from the words - Gravity Recovery and Climate Experiment. The gravitational field strength of a planet depends on size and mass, and the Earth is not uniform in either respect. Because of its rotation Earth's radius is 21km greater at the equator than at the poles and water (which covers 71.1% of Earth's surface) is much less dense than the rock that covers the remaining 28.9%. These two factors, combined with the centripetal force effect of Earth's rotation itself mean that the strength of Earth's gravitational field varies across its surface.

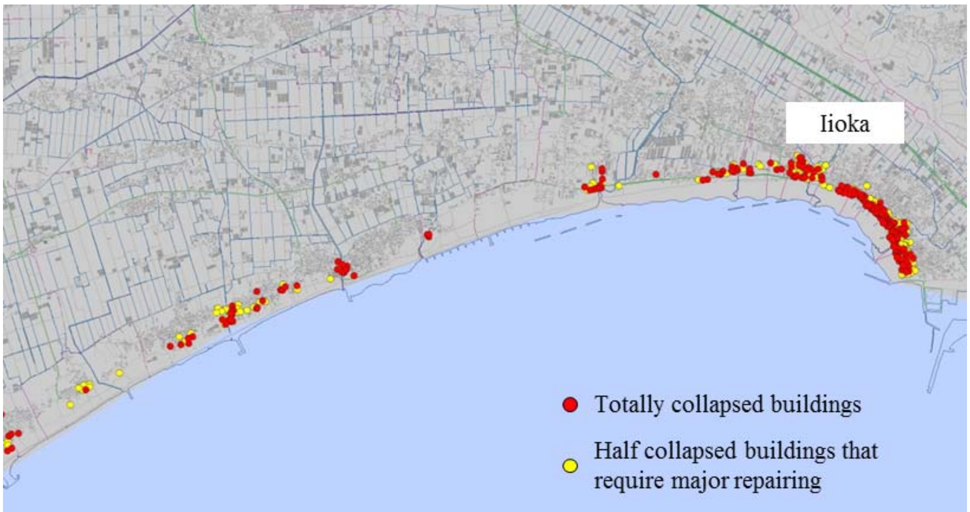

Fig. 10. Distribution of damaged buildings after tsunami in Asahi City, Chiba Prefecture

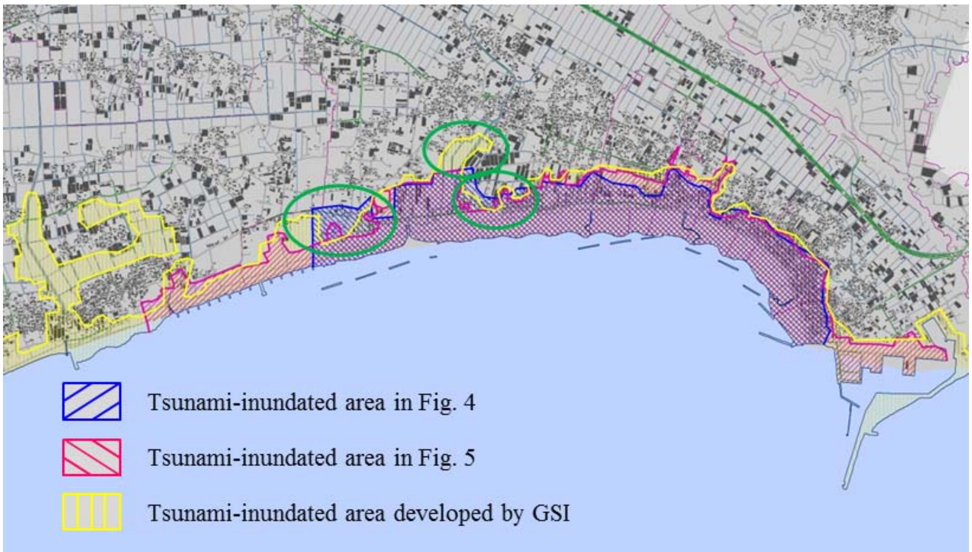

Fig. 11. Comparison among the tsunami-inundated areas developed by Geospatial Information Authority - GSI

The GRACE system can provide us the shape of Earth geoid. To calculate the ice mass on the Greenland we can use the data from satellite GRACE with the comparison of satellite ICESAT1. ICESAT1 exploit the installed laser, which measure the distance from the surface with the accuracy 2cm.

Comparing the results from the GRACE system and ICESAT1, mean comparison of the gravity exploitation measure and the laser measurement we can calculate the information about the annual changes of ice mass in Greenland surface. The calculation from the GRACE system give us the mass 183Gt of the ice loss and the system ICESAT1 give us the amount of 160Gt of annual ice mass loss. The difference is about 23Gt, (Joodaki,G. 2012) [7] which support the theory about water warming concerning the climate change.

Comparison of 2005 ICESat data with 1998-89 airborne laser surveys shows losses during the interim of 80±25 Gt (Thomas et al., 2006) [11], and this is probably an underestimate because of sparse coverage of regions where other investigations show large losses. (J.A.Church, 2010) [4] Ice mass decreasing have an impact on sea level increasing and finally on sea-land flooding.

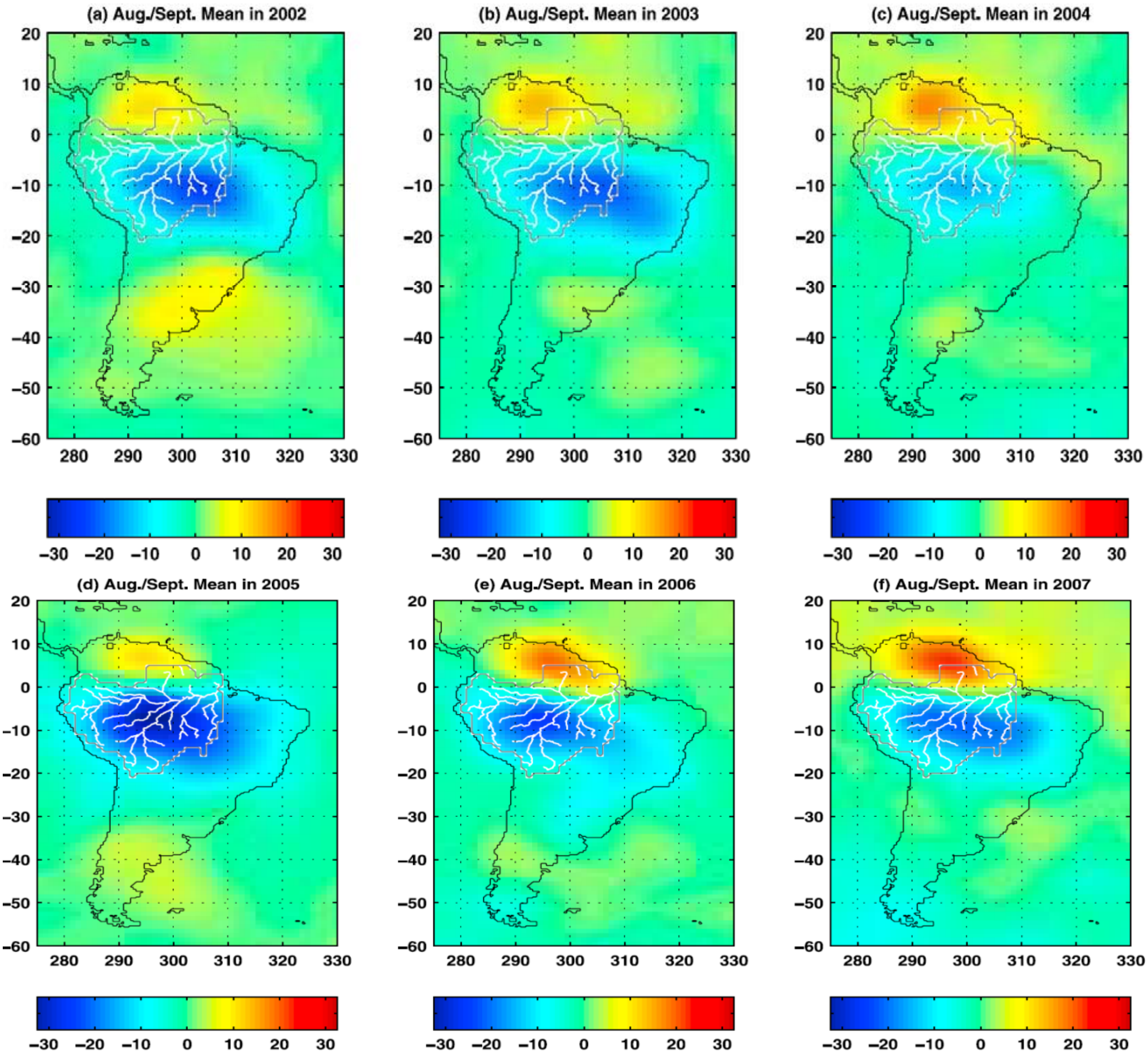

Fig. 12. Changes of water mass in Brazil rainforest – from GRACE satellite GRACE-averaged August and September water storage changes (in centimeters of water) in South America in (a) 2002, (b) 2003, (c) 2004, (d) 2005, (e) 2006, and (f) 2007. A 2-step filtering scheme (P4M6 and 500-km Gaussian smoothing) is applied, as described in (Chen, 2009) [3]

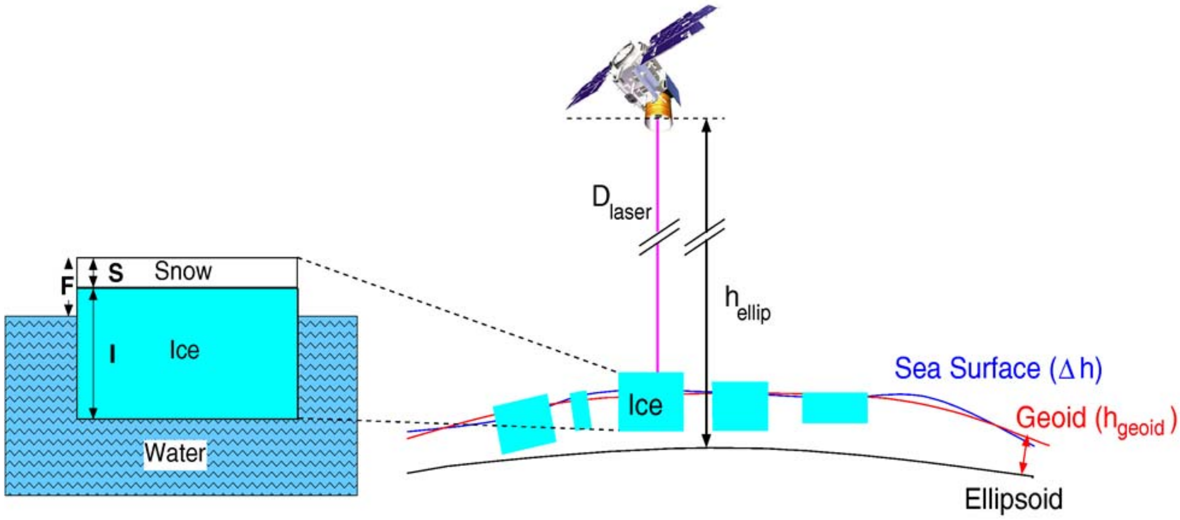

Fig. 13. Principle of the level/high measurement of the snow/ice by the SEASAT1 – laser technology

Calculation principle of Fig. 13:

$$F = h_{ellip} - D_{laser} - h_{geoid} - \Delta h \tag{1}$$

Where

Δh - Ocean dynamic topography

3 RESULTS AND SIGNIFICANCES

The exploitation of the satellite systems such as GPS, GLONAS or the new built up European system GALILEO allowed us to positioning measurement and the special satellites such as SEWAVES, AQUA, AURA, SEASAT, TERRA, GRACE etc. allowed us to measure various parameters concerning the water in the world with the aim to clever exploitation of the water, of the climate change prediction and reducing the negative human activity.

The climate change has the influence on global warming which affects the weather changes, extremes in the weather and have the future negative affect on the thermohaline world water cycle. The measure of the snow covering of the Greenland Island by two systems: GRACE (measure of annual ice decrease - weight 183Gt) and ICESAT - laser (annual ice decrease weight 160 Gt) give us the relatively exact information about annual decrease of the ice and allow estimating the influence on climate change. The measure of the sea level by the satellite GRACE showed us, that in consequence on Earth gravity in dependency on sea deep, the sea level have not always reach the level zero, but there are the extremes plus 130 m and minus 80 meter. It means that sea level in the world context is not the flat. Satellite are exploitable in the rainfall measurement or water capacity e.g. in the Brazil rainforest which can help the scientist to obtain the exact information of world water cycle. Satellite technologies become very important in global and local water management.

4 CONCLUSION

The satellite exploitation provides us the very wide range of information not only about the Earth about the ordinary communication general knows but also the exact procedures for the water management in the wide range. The access to the satellite data can provide us the data which are exploitable for the global water management monitoring, for the prediction of various changes over the world and chance to provide the level of insurance for the people against the flood, weather and various states for the water such as agricultural exploitation etc. The exact data for measurement the points in the unreachable areas is the advantage too. As example we can imagine the rate of sea level measurement, which is possible after few hours over the world comparing the old boat system for the measurement which takes a hundred years for the exact information. To learn how to exploit various types of various satellites could improve the global environment and make the insurance for the life of all people over the world.

Acknowledgments

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SELECTION OF THE OPTIMAL SEWERAGE SYSTEM SOLUTION FOR THE RURAL ENVIRONMENT

Marija Šperac[1], Željko Šreng[2]

Abstract

Given the specificity of rural areas such as the small number of residents in the larger area, specific activities with different water consumption, and the geographical characteristics of the terrain, it is important to choose the type of sewage system that will meet the requirements. With application of the optimization method synthesis, objective of the system is defined and variant solutions for new sewer system are identified and evaluated. For each variant technical evaluation of solutions and the estimated cost of construction is made. As technically most suitable and economically most cost-effective, the variant has been selected which includes the construction of distribution sewer for sanitary-fecal wastewater of the area with the connection of the pressure sewer to the existing sewer system.

Keywords

sewer system, rural areas, the optimal synthesis, technical solution, the cost of construction.

1.INTRODUCTION

When selecting the type of drainage or sewer system, among other things, take into account all the parameters that essentially affect the appropriateness of technical solutions and which are in the process of the analysis already known in advance. This primarily involves the existing construction and land use plans, topography areas, hydrological features of receiver, position of a drainage area in relation to the receiver, hydrological conditions in terms of rainfall and runoff, etc.In all cases the choice of the system should follow the technical - economic analysis, taking into consideration all the present state of the complex of sanitary facilities and local conditions, as well as aspects of rationality and the possibility of harmonizing solutions for removal and cleaning of sanitary sewage, industrial wastewater and stormwater.Selection of the drainage system should be based on future conditions and

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requirements, especially on the conditions for protection of the environment, but also taking into account the current financial capacity to implement the system. In doing so, the sanitary aspect, ie the protection of human health should never be questioned. Quantities of wastewater of an observed area are in direct connection with the consumption of water, and are generally analyzed through the use of water.

Water consumption and the amount of waste water are defined by the following basic sizes:

1. specific water consumption per user at the end of the planning period,
2. number of users at the end of the planning period,
3. coverage area and density of users and their schedules, and
4. coefficient uneven of consumption. [1]

Specific norms of waste water depends on the method of water supply, population structure, climate, etc. However, when determining the specific norms of wastewater must take into account a part of the water consumed for cooking, drinking, watering the garden, etc. Selection type of drainage depends on many factors, primarily the specific local circumstances and conditions, ecological and sanitary requirements, technological requirements and economic indicators. According to the basic indicators, pollution from waste water from household installations is much more dangerous than pollution of rainwater. The amount of pollution in the annual leakage of rainwater, approximately corresponding from 10 to 12 daily household water pollution (as seen in relation to the urban area of 1 ha).On the other hand, industrial waste water may represent a particular problem, especially in the case if they contain toxic substances that could damage the existing wastewater facilities or the function of the common wastewater treatment plant. By the analysis of the relevant indicators for the selection of the drainage system it is assumed that such industries their wastewater reduce to the standard normal household waste water before entering into the public sewer system. Dividing sewage system has advantages over the corresponding mixed system in terms of waste water treatment. Flows to the waste water treatment plant are unvaried both from the amount of variation, and from uniform characteristics of wastewater. Large variations dry / rainy season absent, which are present in mixed systems that require (and despite the application of rainy relief) appropriate additional facilities to synchronize the purification process in the rainy period (additional settling tanks, retention basins, etc.)

2. THE OPTIMIZATION SYNTHESIS

For the construction of a new drainage system, the optimization synthesis is used for the system optimization. It is shown in Figure 1.This method was applied to the design of the concept of sewage system for rural settlements Duzluk. Three possible alternative solutions have been proposed and discussed, and for each variant estimates of construction costs and the technical evaluation of solutions have been made. Settlement Duzluk and facilities around the recreational lake Orahovica don't have organized collection system and wastewater treatment. Drainage is through individual septic tanks or direct discharge into waterways and canals. The current state of sewage and waste water treatment is not satisfactory in sanitary and hygienic way. Settlement Duzluk has 201 inhabitants. Settlement Duzluk is partly within the boundaries of the Park of Nature Papuk. Specifically, the application of dividing drainage ways was provided to resort Duzluk, where the construction of sanitary and sewage system of the village with a connection to the sewerage system of Orahovica was provided. [2]

This construction solution is fully justified only for sanitary sewage system of Duzluk settlements, because Duzluk is in the category of small villages, characterized by predominantly rare residential building in a hilly area intersected by streams. Therefore, we can expect that the storm waters aren't heavily polluted.
Today, the drainage of existing roads is already solved easier way (gutters, ditches, etc.), while the runoff from the remaining part of the basin flow directly to watercourses. For the construction of storm sewers for Duzluk settlements, except for the area around the recreational lake, at the moment there is no need or justification, and the same can be expected only after a prolonged period of time.
If we need the partial collection and drainage of rainwater in the center of the village, solution can be found in building of a distribution network of closed storm channels and their attachment to the open water.

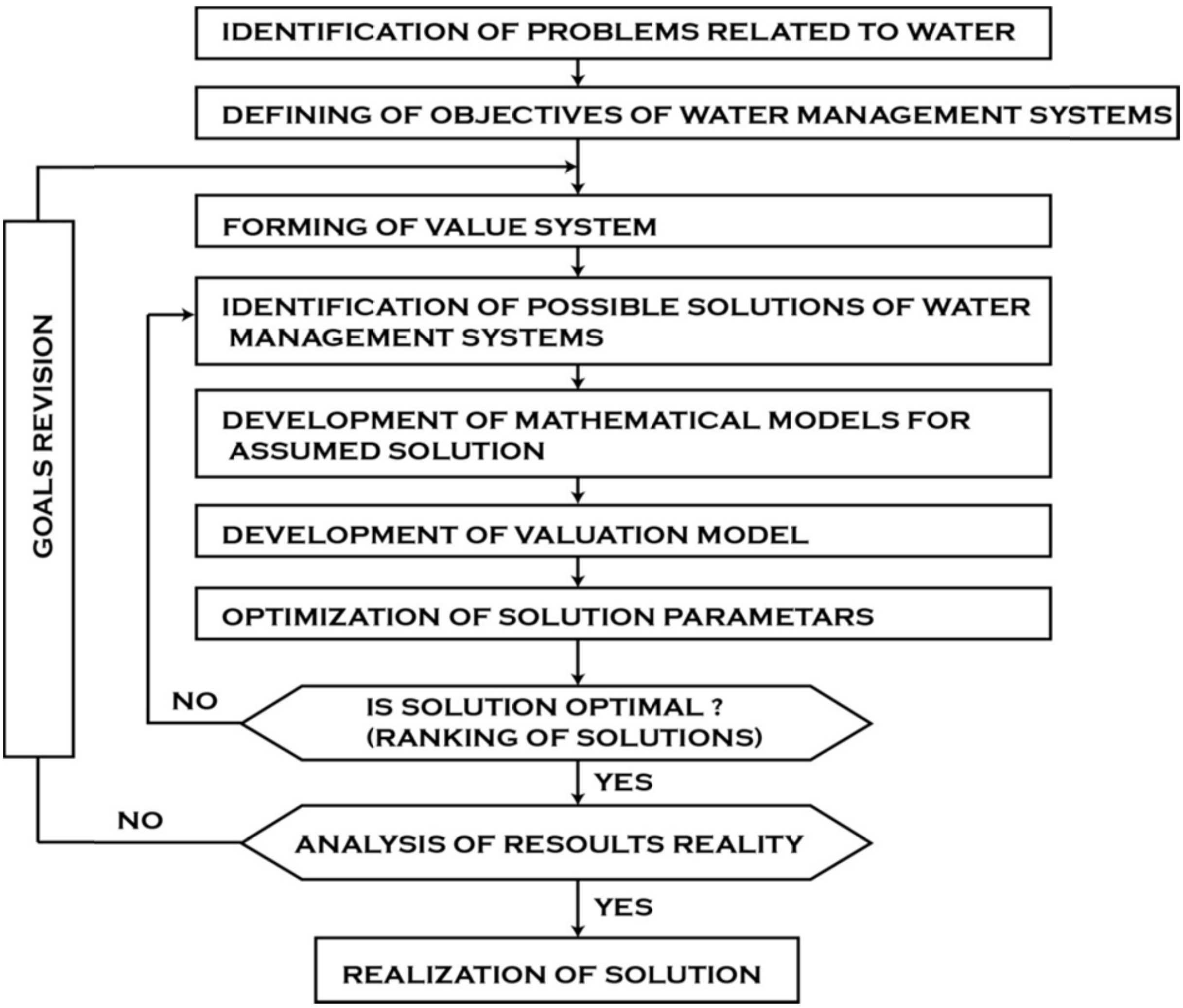

Fig.1 Block diagram of optimization syntesis

In the design of the sewage system for Duzluk settlements, three possible alternative solutions have been proposed and discussed, and for each variant estimates of construction costs and the technical evaluation of solutions have been made. [3]

Two basic variants are in the spirit of the above mentioned and elaborated Conceptual design of drainage system and wastewater treatment for the areas "PAPUK" d.o.o.Orahovica, while the third option is based on the sewerage system Duzluk, as an independent system with its own device for wastewater treatment and discharge into the river Vučica.
The following alternative solutions were considered:

Solution 1˚ - SANITARY- FAECAL SEWAGE SYSTEM OF DUZLUK COMMUNITY WITH THE PRESSURE CONNECTION TO THE ORAHOVICA SEWERAGE SYSTEM

Solution 2˚ - SANITARY- FAECAL SEWAGE SYSTEM OF DUZLUK COMMUNITY WITH THE GRAVITATIONAL CONNECTION TO ORAHOVICA SEWERAGE SYSTEM

Solution 3˚- SANITARY SEWAGE SYSTEM OF DUZLUK COMMUNITY WITH THE SEPARATE DEVICE FOR WASTE WATER TREATMENT

3. DESCRIPTION OF ALTERNATIVE SOLUTIONS

3.1. Solution 1˚ - pressure connection to Orahovica sewage system

This solution suggests that sanitary- faecal waste water of the Duzluk community will be collected with divided closed sewer system that consists of gravity collector, pumping stations and pressure piping pertaining to the system.
Collected waste water would be transported by pressure to the Orahovica sewerage system and further away to the future joint waste water purification device. Until that waste water would be discharged into the Vučica River as a temporary solution (second category receiver).
Location of the future waste water purification device would be behind the pumping station Fatovo.
The main parts of the sewerage network of the Duzluk community are the main gravity collector of approximate length of 1.700 meters, which is the recepient of other collectors and secondary sewer community network with the total length of approximately 3.500 meters.

To ensure the flow of waste water in gravity sewer system, i.e. to ensure minimun slopes and following the criteria of minimum depth of piping entrenchment. The topography of the terrain would also be taken into account when it comes to the construction of the four pumping stations with the corresponding pressure piplines. The total length of the pressure pipelines is approximately 1.200 meters.

Collected sanitary- faecal waste waters of the Duzluk community would be transported to the Orahovica sewerage network with the main pressure station and interlocal pressure pipelines of approximate length of 1.350 meters and in direction of the future waste water purification device.

The main technical elements of the system are:
- main collector of the Duzluk community 1.700 m
- secondary collectors of the Duzluk community 3.500 m
- pumping stations in sewerage system of the Duzluk community 4
- corresponding pressure pipelines 1.200 m
- the main pressure pumping station
- interlocal pressure pipelines Duzluk – Orahovica 1.350 m

3.2. Solution 2˚ - gravity connection on Orahovica sewage system

This solution suggests that sanitary- faecal waste water of the Duzluk community will be collected with divided closed sewer system that consists of gravity collector, pumping stations and pressure piping pertaining to the system.
Collected waste water would be transported by gravity to the Orahovica sewerage system and further away to the future joint waste water purification device. Until that waste water would be discharged into the Vučica River as a temporary solution (second category receiver).
Location of the future waste water purification device would be behind the pumping station Fatovo.
The main parts of the sewerage network of the Duzluk community are the main gravity collector of approximate length of 1.700 meters, which is the recepient of other collectors and secondary sewer community network with the total length of approximately 3.300 meters.
To ensure the flow of waste water in gravity sewer system, i.e. to ensure minimum slopes and following the criteria of minimum depth of piping entrenchment. The topography of the terrain would also be taken into account when it comes to the construction of the four pumping stations with the corresponding pressure piplines. The total length of the pressure pipelines is approximately 1.200 meters.
Collected sanitary- faecal waste waters of the Duzluk community would be transported to the Orahovica sewerage network with interlocal gravity pipelines of approximate length of 1.350 meters and in direction of the future waste water purification device.

The main technical elements of the system are:
- main collector of the Duzluk community 1.700 m
- secondary collectors of the Duzluk community 3.300 m
- pumping stations in sewerage system of the Duzluk community 4
- corresponding pressure pipelines 1.200 m
- interlocal pressure pipelines Duzluk – Orahovica 1.350 m

3.3. Solution 3˚ - individual purification device

This solution suggests that sanitary- faecal waste water of the Duzluk community will be collected with divided closed sewer system that consists of gravity collector, pumping stations and pressure piping pertaining to the system.
Collected waste water will be transported to the individual waste water purification device with capacity of 600 EP (equivalent population). After treatment waste water would be discharged into the Vučica River (second category receiver). [4]

The main parts of the sewerage network of the Duzluk community are the main gravity collector of approximate length of 1.900 meters, which is the recepient of other collectors and secondary sewer community network with the total length of approximately 3.300 meters.
To ensure the flow of waste water in gravity sewer system, i.e. to ensure minimun slopes and following the criteria of minimum depth of piping entrenchment. The topography of the terrain would also be taken into account when it comes to the construction of the four pumping stations with the corresponding pressure piplines. The total length of the pressure pipelines is approximately 1.600 meters.
Collected sanitary- faecal waste waters of the Duzluk community would be transported to the individual waste water puification device which would be located north of the Orahovica lake and after treatment discharged into the Vučica River.

The main technical elements of the system are:
- main collector of the Duzluk community 1.900 m
- secondary collectors of the Duzluk community 3.300 m
- pumping stations in sewerage system of the Duzluk community 4
- corresponding pressure pipelines 1.600 m
- waste water purification device (600 EP)
- pressure pipelines from purification device to the Vučica River 120 m

4. ESTIMATE OF CONSTRUCTION COSTS BY SOLUTIONS

4.1. Solution 1

Tab.1 Estimate of construction cost for solution 1

Ordinal number	**item**	**length (m)**	**unit cost (kn)**	**PRICE (kn)**
1.	main collector	1.700	1.200,00	2.040.000,00
2.	secondary collectors	3.500	800,00	2.800.000,00
3.	four pumping stations		150.000,00	600.000,00
4.	pressure pipelines	1.200	150,00	180.000,00
5.	the main pressure pumping station		150.000,00	150.000,00
6.	interlocal pressure pipelines	1.350	150,00	202.500,00
TOTAL	**SOLUTION 1:**	**7.750**		**5.972.500,00**

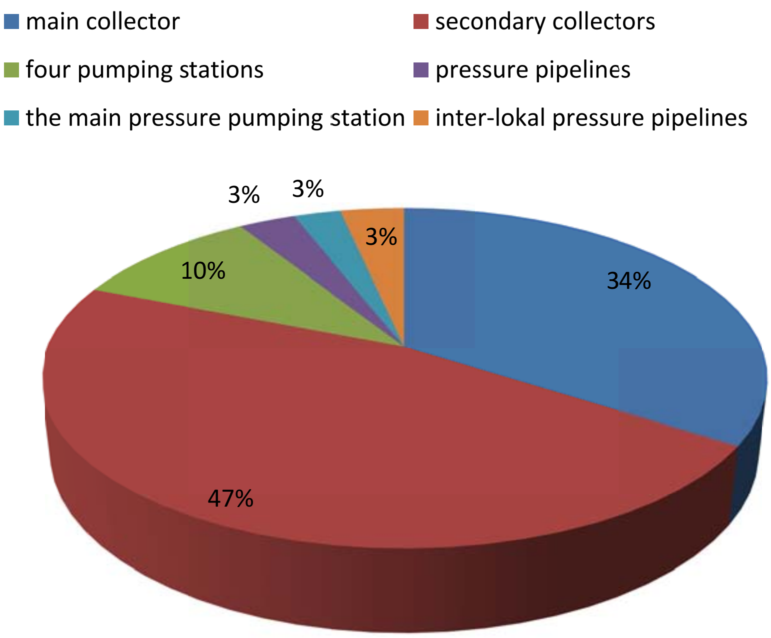

Fig.2 Estimate of construction cost for solution 1

4.2. Solution 2

Tab.2 Estimate of construction cost for solution 2

Ordinal number	item	length (m)	unit cost (kn)	PRICE (kn)
1.	main collector	1.700	1.200,00	2.040.000,00
2.	secondary collectors	3.300	800,00	2.640.000,00
3.	four pumping stations		150.000,00	600.000,00
4.	pressure pipelines	1.200	150,00	180.000,00
5.	interlocal gravity pipelines	1.350	1.200,00	1.620.000,00
TOTAL	**SOLUTION 2:**	**7.550**		**7.080.000,00**

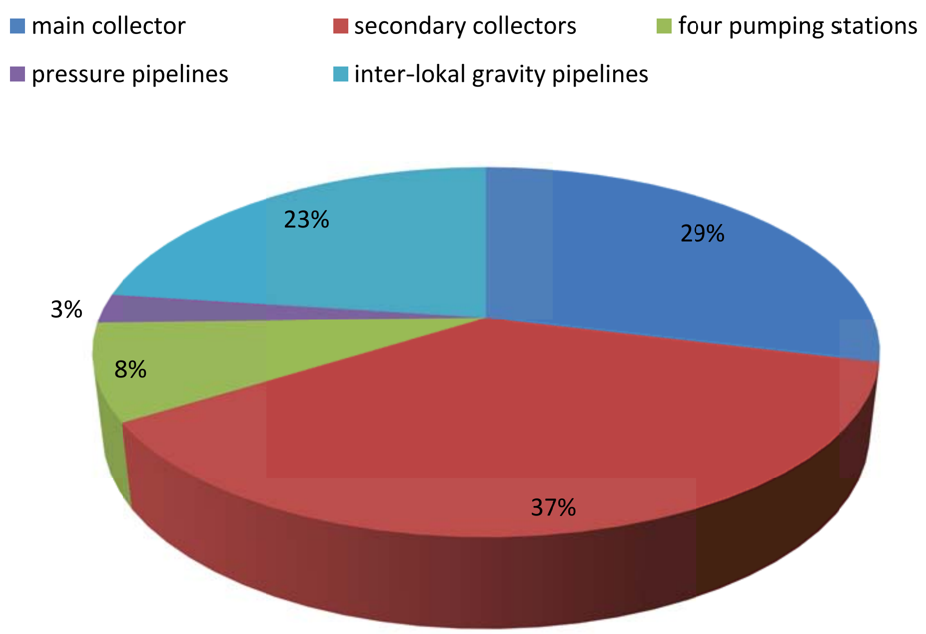

Fig.3 Estimate of construction cost for solution 2

4.3. Solution 3

Tab.3 Estimate of construction cost for solution 3

Ordinal number	item	length (m)	unit cost (kn)	PRICE (kn)
1.	main collector	1.900	1.200,00	2.280.000,00
2.	secondary collectors	3.300	800,00	2.640.000,00
3.	four pumping stations		150.000,00	600.000,00
4.	pressure pipelines	1.600	150,00	240.000,00
5.	purification device (600 EP)		1.440.000,00	1.440.000,00
TOTAL	**SOLUTION 3:**	**6.800**		**7.200.000,00**

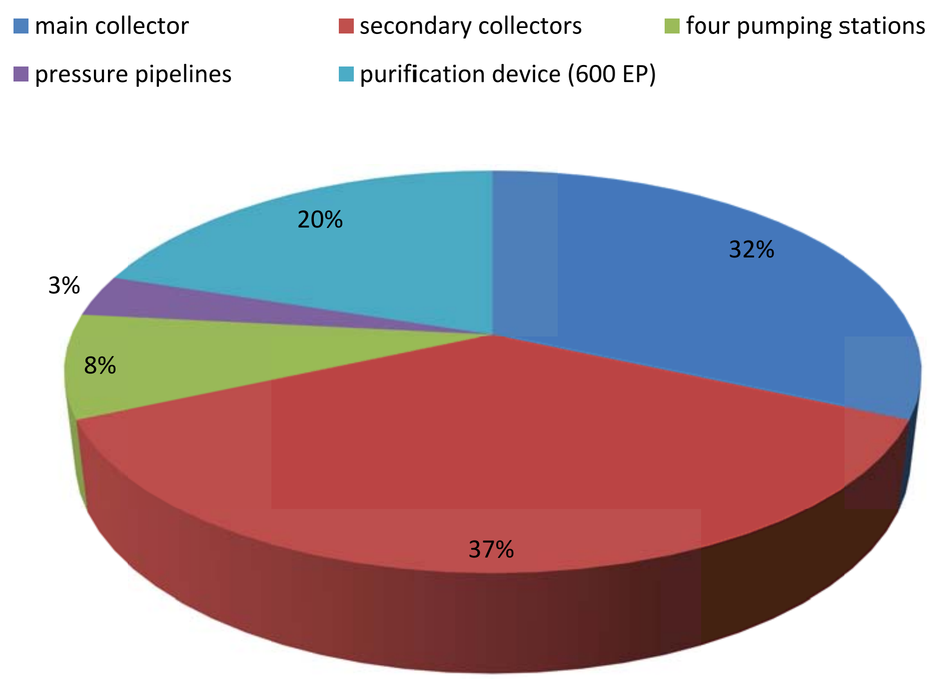

Fig.4 Estimate of construction cost for solution 3

5. TECHNICAL EVALUATION AND SELECTION OF SOLUTIONS

In preliminary design of the sewerage system of sanitary- faecal waste water in the Duzluk community, three possible alternative solutions were proposed and dicussed. Estimate for the amount of investment and technical evaluation has been made for each solution.

Two basic variants are in the spirit of the above mentioned, and elaborated Conceptual design of drainage system and wastewater treatment for the areas "PAPUK" d.o.o.Orahovica, while the third option is based on the sewerage system Duzluk, as an independent system with its own device for wastewater treatment and discharge into the river Vučica.

When it comes to the choice of the solution of the sanitary-faecal sewerage system of the Duzluk community, the following parameters, amongst others, were taken into account:

- appropriateness of the tehnical solution, in relation to topographical conditions, hydrographic characteristic, spatial planning and position of drainage area concerning the receiver
- functionality of technical solutions of sewerage system
- amount of the investments of making alternative solutions
- maintenance of sewerage system and operating costs
- possibility of building the sewerage system in phases

After analysis of developed solutions of sanitary- faecal sewerage of Duzluk communites, as technically the most acceptable and economically most cost- efficient, solution number 1 was selected, i.e. construction of divided sewerage system of sanitary- faecal waste waters of Duzluk communites with pressure connection to the sewerage system Orahovica.

6. CONCLUSION

For finding the optimal solutions for the new sewer system, the method of the optimization synthesis is used. Three variants of solutions are proposed. For given criterias for the optimization, and existing or potential limitations of the system the optimal variant is chosen. [5]
Building of the sewerage system is a complex and expensive process. Therefore, it is necessary to plan well and design and to find the optimal solution to satisfy the needs of users for a longer period of time and provide a quality management system.

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HYDRAULIC ANALYSIS OF THE MAXIMUM FLOW RATE OF THE REGIONAL WATER SUPPLY SYSTEM "STUDENCHICA"

Goce Taseski[1], Petko Pelivanoski[2]

Abstract

The regional water supply system "Studenchica" was launched in 1980. Its purpose is to water supply the cities Kichevo, Prilep, Makedonski Brod, Krushevo and a number of villages in the region where the system is spanning. Also this system provides water for TE Oslomej in Kichevo. In this analysis two mathematical-hydraulic models are made: the first hydraulic mathematical model examines the current maximum permeable of the main supply pipeline with determined ratios of coarseness of the pipeline, and the second hydraulic mathematical model analyzes the maximum permeable of the main supply pipeline for period after 30 years with forecasted ratio of coarseness of the pipeline.

Keywords

water supply system, ratio of coarseness, hydraulic analysis, maximum permeable of the pipeline.

1. Introduction

The regional water supply system "Studenchica" (figure 1) covers the water from the source Studenchica which by gravitational route is brought to the cities Kichevo, Prilep, Makedonski

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Brod and the settlements in their immediate vicinity while only for the town Krushevo the water is supplied from a pumping station. Also gravity brings water for cooling the generators from the thermal power plant Oslomej. In situations where the capacity of the source Studenchica is greater than the population's needs, the surplus water is distributed to the artificial accumulation in Prilep which water is then used for irrigation.

Figure 1. Schematic review of the regional water supply system "Studenchica"

The intake piping to all settlements that are connected by the water supply system "Studenchica" are derived from steel tubes of varying diameter and appropriate thickness of the pipe walls depending on the available pressure.

The water supply system is integrated with adequate hydro mechanical equipment which serves for management and maintenance of the system. In the split construction in village Drugovo, electromagnetic water meters are placed that measure the total intake amount of water and the water that goes towards Kichevo, Prilep and TE Oslomej. On the intake piping numbers of air and drain valves are performed as well as sector covers. A chlorine station was built within the water supply system near the final chamber of Barbaras.

For management with the regional water supply system "Studenchica" a Public Enterprise for the regional water supply system Studenchica was established in 1990 (JP Studenchica). The powers of the public enterprise are: rational management of the water supply system, to perform regular maintenance and upgrading of the system and to perform proper allocation of the quantities of water towards the connected customers in accordance with the water management permission.

Seeing that the regional water supply system "Studenchica" is in use nearly 32 years and already exceeds the prescribed exploited period with the General project there imposes the need for analysis of the maximum permeable of the pipes by sections.
According to earlier in this paper complete hydraulic analyses of the current state of the system are made as well as hydraulic analyses for the future thirty-year period of exploitation of the system.

2. Geometric characteristics of the intake pipeline

The geometric characteristics of the pipeline were provided by the existing technical documentation and verification on the spot with the professional services that manage the system and are shown in table 1.

Table1. Geometric characteristics of the pipeline from the Regional water supply system Studenchica

No.	Section from	Section to	Length L [m]	OD [mm]	δ [mm]	ID [mm]
1	Zafat	RG Drugovo	12,401	1,016.0	12	992.0
2	RG Drugovo	Barbaras	1,631	914.4	12	890.4
			17,353	914.4	13	889.0
			5,500	914.4	12	890.4
			2,400	863.6	12	839.6
			900	863.6	11	841.6
			4,326	863.6	8	847.6
3	Barbaras	IB Prilep	3,225	812.8	9	794.8
			2,250	812.8	11	740.8
			10,112	762.0	11	740.0
			7,119	711.2	10	691.2
			4,044	711.2	8	695.2
4	RG Drugovo	REK Oslomej	13.5	1,016.0	12	992.0
			2,491.2	457.2	8	449.2
			11,683.4	355.6	6	349.6
5	RG Debreshte	Pump station Norovo	11,562.5	355.6	5.6	344.5
6	RG Drugovo	Chlorine station Kichevo	652.0	508.0	8	492.0
TOTAL LENGTH			**98,398.3**	/	/	/

The table shows that the total length of the steel piping from RWSS Studenchica is 98,398.3 m, the smallest exterior diameter is the section of RG Debreshte - pump station Norovo is 355.6 mm, while the largest exterior diameter is the section of Zafat - RG Drugovo is 1,016.0 mm.

Except the outer diameters of the pipelines, the table presents the thickness of the pipe walls and the inner diameter of the pipe, which is used in the hydraulic analysis of the system.

It should be mentioned that in December 2010 elaborate is made based on the measured thickness of the pipe walls where it can be seen that until the date of measurement there are no significant changes in the thickness of the pipe.

3. Calibration of the mathematical model - determining the ratio of the coarseness of the pipeline

All mathematical models that derive results used later for making some decisions should previously be calibrated. Calibration is a procedure whereby the input parameters in the mathematical model are adjusted in the range of adjustable limits to the actual parameters of the analyzed system.

The calibration process consists of collecting data from the real system, their calibration with the model and assess of the accuracy of calibration. The collection of data from the real system involves collecting field data, such as upland elevations of sources, connecting points, all objects on the system that contain hydro mechanical equipment, measurement of flow and pressure in characteristic points of the system and more. All these measurements should be carried out at sufficient number and different amounts of water flowing into the system in order to obtain reliable data on the system condition.

It is best the calibration to take place in two stages. The first stage is the level before the study which phase will reveal whether the received data are reasonable at this stage that would eventually discover uncontrolled leakage of the system, whether the valves are open or closed, controls the amount of leaks in split ______________________refuse chutes, illogical downs of the hydrodynamic line - large losses etc. Once the first phase will give satisfactory results i.e. the obtained pressures and flows by sections are adjustable-expected sizes, in the second phase it will determine the coefficients of hydraulic resistance.

Then with the defined ratios of hydraulic resistance the pressures in controlled junction points are controlled and compared with the measured data, which determines the percentage of correctness of the model, i.e. the smaller the deviations we have the more real mathematical model of the system.

Aforementioned procedure for calibration of mathematical model to the real system was made during the mathematical hydraulic model of RWSS Studenchica. The values of the measured pressures and flows on 09.11.2011 on certain areas in VS Studenchica were provided by JP Studenchica. According the data it can be noted that two measurements of the pressure and flow are made at the section from Zafat to Drugovo, five measurements on the section Drugovo-Barbaras and four measurements on the section Barbaras-Prilep.

With the help of calibration of the mathematical model and the measurement data of the system, values of hydraulic resistance coefficient "C" are obtained according to Hazen-Williams or a coefficient of hydraulic resistance "λ" is obtained. In addition, table 2 gives the calculated values of the coefficient of hydraulic resistance "C" or the appropriate coefficient of hydraulic resistance "λ" obtained by calibration of the mathematical model.

Table 2. Coefficient of friction by sections resulting from the calibration of hydraulic mathematical model

No..	Section		Coefficient of friction	
	from	to	C	λ
1	Zafat	VV Dobrenoec	125	0.017
2	VV Dobrenoec	RG Drugovo	130	0.016
3	RG Drugovo	VV Drugovo	130	70.250
4	VV Drugovo	VV Plasnica	135	0.016
5	VV Plasnica	VV Мак. Brod	109	0.023
6	VV Мак. Brod	VV Barbaras	148	0.013
7	VV Barbaras	R.Sh. Debreshte	80	0.041
8	R.Sh. Debreshte	Sarandinovo	190	0.008
9	Sarandinovo	Chlorine station	107	0.025

10	Chlorine station	I.B. Prilep	1000	0.001

As an indication of the obtained values it should be noted that the large value of the coefficient of hydraulic resistance λ = 70.250 is due to the fact that this section had a cover that during the measurement of pressure was not fully open, provided that the local losses are quite large in the model of the coefficient of hydraulic resistance these local losses are calculated and therefore its value is quite high.

The coefficient of hydraulic resistance C by Hazen-Williams according to recommendations based on experiences of the application of steel pipes in the system under pressure should move in the range of 140-150 for new pipes to 110 for pipes at the end of exploitation. While the obtained coefficient of hydraulic resistance with hydraulic mathematical model ranges between 80-1.000. Such obtained values of the coefficient of hydraulic resistance indicates anomalies in the measurement of pressure and flow in certain sections of RWSS Studenchica, namely it can be seen that only on certain sections the coefficient of hydraulic resistance is at the recommended boundaries by literature, and on the remaining sections the obtained coefficient of friction is illogical and therefore at a later stage of calibrating the model, the sections where the coefficient of hydraulic resistance is illogical are not considered.

Due to the aforementioned illogical values for the coefficient of hydraulic resistance which are received only on certain parts of RWSS Studenchica, the calibration of the model is made only on three sections (Zafat – Drugovo, Drugovo- Barbaras, Barbaras - Prilep) where for calibration only two points of the section are taken. This procedure is carried out in order to get a matched coefficient of hydraulic resistance (average) on the sections of RWSS Studenchica and the calculated values are shown in table 3.

From table 3 it can be seen that the coefficient of hydraulic resistance in the three sections is within the limits recommended by the literature but it has a different value that ranges between C = 130-120 (λ = 0.016 - 0.020). If one knows the fact that the pipes from VS Studenchica are of the same material, from the same manufacturer and the same age, then the value of the coefficient of hydraulic resistance in all sections should be the same, which in this case isn't, and this is probably due to erroneous data from the measurement of pressure and/or flow in the system.

Table 3. Coefficient of friction on sections obtained by calibrating two points of the section.

No.	Section		Coefficient of friction	
	from	to	C	λ
1	Zafat	RG Drugovo	130	0.016
2	RG Drugovo	VV Barbaras	125	0.018
3	VV Barbaras	IB Prilep	120	0.020

For additional authentication of the parameters obtained from the measurements of pressure and flow on day 07.06.2012, additional measurements on the section Barbaras-Prilep were made and the investor was suggested where attention should be given when performing measurements in order to ignore all previously mentioned anomalies. From the obtained results of the measurements it can be concluded that with the flow of Barbaras from 373 (L/s) i.e. by the chlorine station in Prilep 312 (L/s) the pressure by IB Prilep is 5.5 bar. Then with these measurements additional calibration is performed of the hydraulic model where it was obtained that the coefficient of hydraulic model is C = 125 (λ = 0.019).

Because of the aforementioned hydraulic analysis on RWSS Studenchica, the hydraulic analyses on RWSS Studencica are made for adopted two different coefficients of hydraulic resistance, according to Hazen-Williams they are C = 125 and C = 120, these adopted values are in accordance with the recommendations of the literature for a system that is thirty years old and their size corresponds to the average, obtained with the previous calibration of the hydraulic mathematical model. By the adopted lower value of the coefficient of hydraulic resistance (C = 120) for the period after thirty years-in 2040, the model is on the side of safety, as the system is currently thirty years old and is already entering a period of phase which is close to the bottom for exploitation while in future the system "ages faster" and with it the coefficient of hydraulic resistance by Hazen-Williams will get less value.

4. Junction demand - water needs in settlements

In the mathematical hydraulic model, maximum daily water needs for each township are simulated as junction expenditures in the existing connection points of the main supply pipeline from VS Studenchica. The location of the connection points of the pipeline is obtained by JP Studenchica. This simulation is made from an aspect that with the hydraulic model, a hydraulic analysis of the main supply pipeline will be performed. Also near Gorni Dobrenoec at 4,500 m from the source Studenchica, a junction expenditure is predicted for the small hydro power plant Dobrenoec where as junction expenditure are entered 25 l/s amounts of water as the minimum amount of water to meet the biological minimum, up to 300 l/s as a maximum – installed flow at the hydro power plant.

In the hydraulic model, the junction expenditures as water needs for the population are included for two distinctive periods of exploitation of the system as follows:

- Current state of the system where the water needs for the population in all settlements covered by the VS Studenchica are the greatest, i.e, there are major water losses in the water networks of the settlements and
- In the period after 30 years of exploitation of the system, i.e for the final period of the analysis of water needs in the settlements where in the water networks within the towns the water losses will be minimized.

5. Hydraulic model of the regional water supply system "Studenchica"

The hydraulic model of RWSS Studenchica is made in the software package "EPANET" that serves for hydraulic analysis of the system under pressure as this system. In the hydraulic analysis, two specific periods of exploitation of the system are analyzed as follows:

- hydraulic analysis of the main supply pipeline for current situation (2010) with the greatest water needs for the population and where the coefficient of friction by Hazen-Williams is C = 125
- hydraulic analysis of the main supply pipeline for a period after thirty years (2040), when the total losses in the water supply networks in all settlements covered by the system are minimized with the coefficient of friction by Hazen-Williams is C = 120

It should be said that the hydraulic simulation is done for different capacity of the source and different distribution of water to the settlements.

5.1. Hydraulic analysis of the main supply pipeline for present state – 2012

In the hydraulic analysis of the main supply pipeline for current situation (2012) as it was previously mentioned, the hydraulic resistance coefficient is C = 125, and in the junction expenditures the current water needs condition of the system for 2012 are entered. With this imported inputs, hydraulic simulation is made for the different capacity of the source Studenchica, ranging from the smallest recorded capacity of the source of 490 (L/s) to capacity of the source which covers all water needs for the population, the biological minimum, REK Oslomej and the installed flow of MHE Dobrenoec and a certain amount of water that filled the accumulation in Prilep.
The amount of water that will be distributed to the accumulation in Prilep depends on the maximum flow rateof the system.

The hydraulic simulation of the system is designed so that as water needs are entered water needs of the settlements which needs are in function of the capacity of the source Studenchica. When the capacity of the source Studenchica does not cover the water needs for REK Oslomej, for the population and for the biological minimum, which is discharged into the drain valve Dobrenoec–by the predicted MHE Dobrenoec, then in the junction expenditures as water needs are entered reduced amounts of water with defined percentage of restriction, which limit of percentage decreases as the capacity of the source increases and gets value zero for the condition when the source Studenchica covers the water needs.

While for the biological minimum in period when water needs are not covered for the population drops 25 (L / s), in the period when the capacity of the source is greater than the water needs and releases 50 (L/s), and for REK Oslomej in the entire period from the analysis are delivered 160 (L/s).

Table 4. Effective pressures in characteristic junction points depending on the capacity of the source Studenchica

Q [l/s]	Dobrenoec	Barbaras	Prilep	Kichevo	Oslomej	Pump st. Norovo
	P [m]	P [m]	P [m]	P [m]	P [m]	P [m]
490	**207**	**97**	**103**	**239**	**95**	**49**
600	**206**	**92**	**99**	**237**	**87**	**46**
700	**205**	**86**	**95**	**234**	**79**	**43**
800	**204**	**80**	**89**	**231**	**71**	**40**
900	**203**	**72**	**82**	**228**	**63**	**37**
1,000	**202**	**63**	**74**	**224**	**54**	**33**
1,100	**201**	**52**	**63**	**221**	**47**	**29**
1,200	**199**	**38**	**44**	**217**	**43**	**26**
1,300	**198**	**29**	**32**	**214**	**40**	**25**
1,400	**196**	**27**	**32**	**213**	**39**	**25**
1,500	**195**	**26**	**32**	**211**	**37**	**25**
1,600	**193**	**13**	**16**	**207**	**33**	**23**
1,644	**192**	**5**	**5**	**205**	**32**	**22**

For different capacity of the source Studenchica table 4 presents the elevations of the hydrodynamic lines and the effective pressures obtained from the hydraulic model in characteristic junction points: MHE Dobrenoec, Barbaras, I.B. Prilep, P.K. Kichevo, Oslomej and the pump station Norovo. At the other junction points of RWSS Studenchica the pressure is greater than the one presented in the table and is not important in determining the flow rateof the system. While figure 2 shows a graph of the effective pressure by the final chamber in Barbaras - the highest point of the system and on figure 3 by the overflowed pool in Prilep.

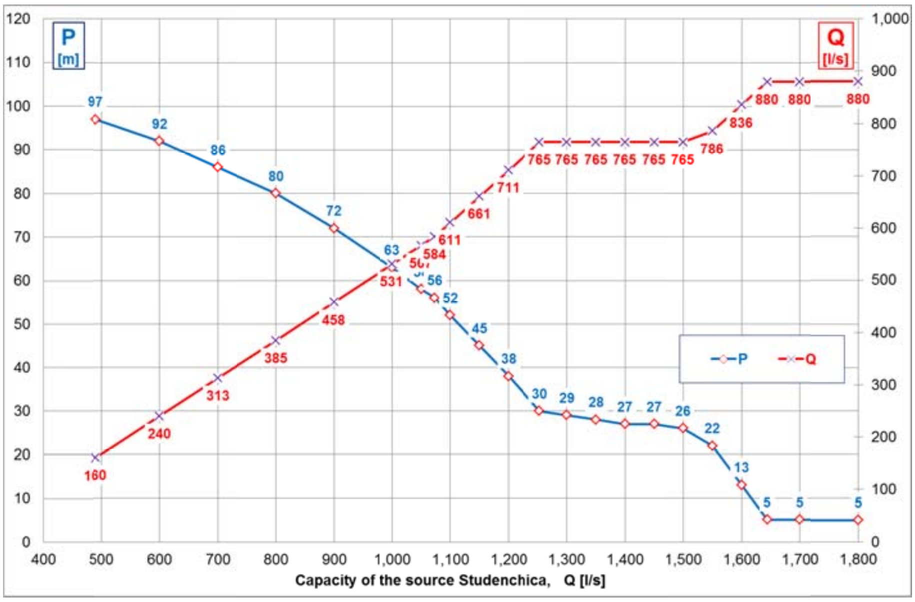

Figure 2. Effective pressures by PK Barbaras and quantity of water that are distributed to Barbaras depending on the capacity of the source Studenchica.

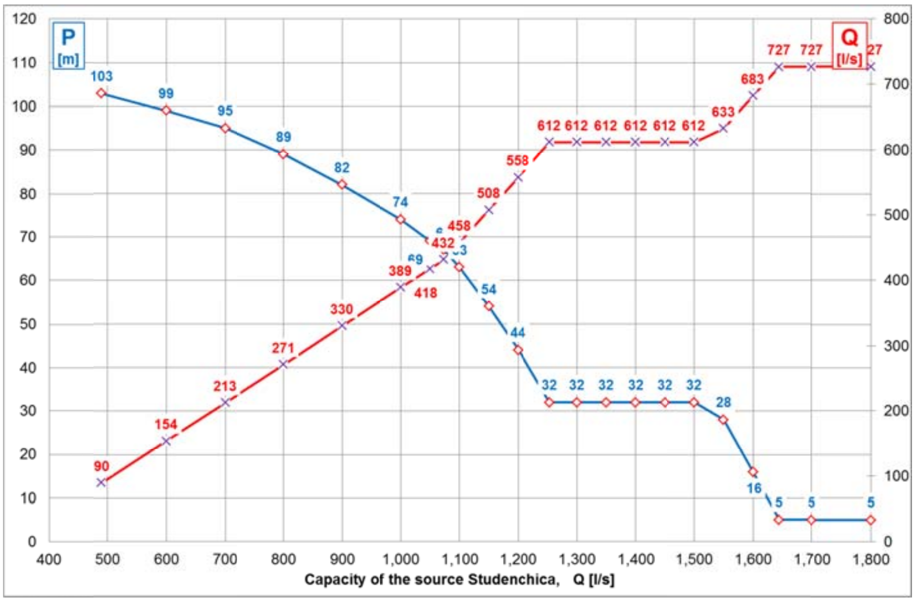

Figure 3. Effective pressures by Prilep and quantity of water that is distributed to Prilep according the capacity of the source Studenchica.

The maximum flow rateof the system is determined by the requirement that the minimum pressure by P.K. Barbaras and by I.B. Prilep allows leakage in the interrupted chamber by gravitation and it is 5 m - minimum allowable pressure in the intake piping. Therefore by the hydraulic analysis, the same can be noted in the previous table and graphs, the maximum permeable hydraulic capability of RWSS Studenchica for adopted coefficient of hydraulic resistance by Hazen-Williams C = 125 is Qmax = 1,644 [L/s]. while the maximum flow rateof the system by sections is shown in table 5.

Table 5. Maximum flow rate of systems by sections

No.	Section from	Section to	Qmax [l/s]
1	Source Studenchica	MHE Dobrenoes	1,644
2	M.H.E. Dobrenoec	RG Drugovo	1,344
3	RG Drugovo	P.K. Barbaras	880
4	RG Drugovo	Kichevo	258
5	RG Drugovo	REK Oslomej	205
6	P.K. Barbaras	RG Debreshte	815
7	RG Debreshte	I.B. Prilep	727
8	RG Debreshte	Pump station Norovo	88

5.2. Hydraulic analysis of the main supply pipeline for a period after thirty years - 2040

Hydraulic analysis of the main supply pipeline for the period after thirty years of exploitation is done on the same way as the current situation with the difference that the coefficient of hydraulic resistance by Hazen-Williams is projected to be C = 120 and the water needs for the population as a result of reduction of the total loss of water in the settlements are smaller than the current situation.

Thus for different capacity of the source Studenchica in the following table 6 are shown elevations of the hydrodynamic lines and the effective pressures obtained from the hydraulic model in characteristic junction points: MHE Dobrenoec, Barbaras, I.B. Prilep, P.K. Kichevo Oslomej and the pump station Norovo. At other junction points from RWSS Studenchica the pressure is greater than the one in the table and isn't important in determining the flow rateof the system.

Table 6. Effective pressures in characteristic junction points depending on the capacity of the source Studenchica

Q [l/s]	Effective pressures, P [m]					
	Dobrenoec	Barbaras	Prilep	Kichevo	Oslomej	Pump st. Norovo
490	**206**	**97**	**104**	**238**	**71**	**47**
600	**206**	**92**	**101**	**236**	**60**	**45**
700	**205**	**86**	**97**	**233**	**50**	**41**
850	**203**	**74**	**86**	**228**	**39**	**37**
900	**203**	**69**	**80**	**227**	**37**	**36**
1,100	**200**	**59**	**69**	**223**	**33**	**35**
1,200	**199**	**57**	**69**	**221**	**31**	**35**
1,300	**197**	**49**	**59**	**218**	**29**	**33**
1,400	**195**	**34**	**39**	**214**	**25**	**31**

1,500	**193**	**17**	**15**	**210**	**20**	**28**
1,535	**193**	**10**	**6**	**208**	**19**	**27**

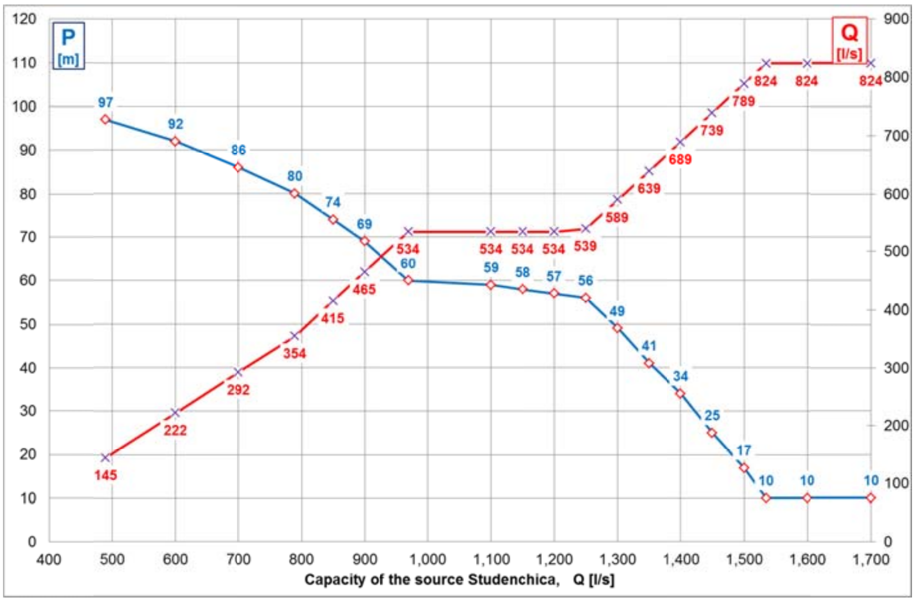

Figure 4. Effective pressures by Barbaras and quantity of water that is distributed to Barbaras depending on the capacity of the source Studenchica.

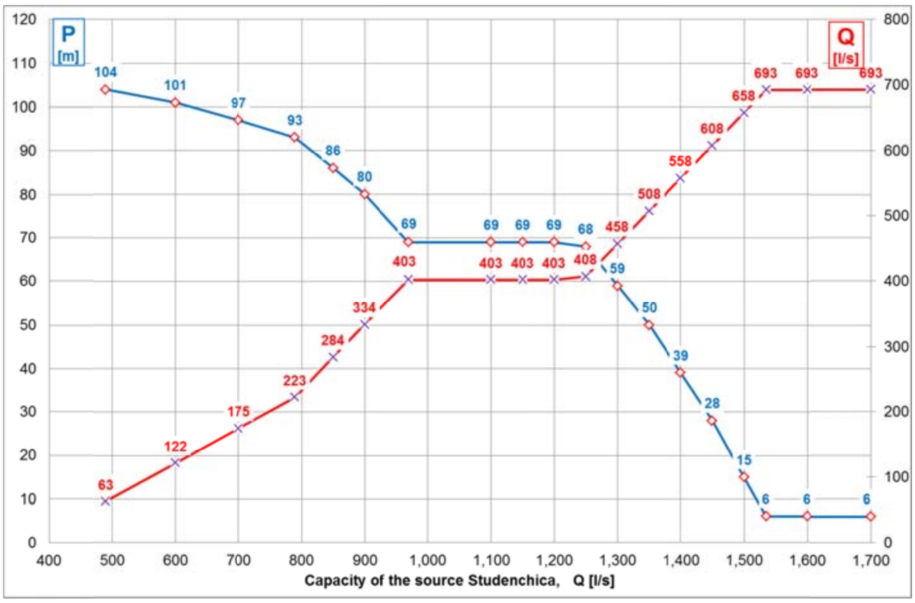

Figure 5. Effective pressures by Prilep and quantity of water that is distributed to Prilep according the capacity of the source Studenchica.

According the previous hydraulic analyses it can be seen that the maximum flow rateof the system after forty years of exploitation (as of 2010) and for predicted coefficient of hydraulic resistance by Hazen-Williams C = 120 is Qmax = 1,535 [L/s]. while the maximum flow rateof the system by sections is shown in table 7.

Table 7. Maximum flow rate of the system by sections

No.	Section from	Section To	Qmax [l/s]
1	Source Studenchica	MHE Dobrenoec	1,535
2	MHE Dobrenoec	РГ Drugovo	1,235
3	RG Drugovo	P.K. Barbaras	824
4	RG Drugovo	Kichevo	205
5	RG Drugovo	REK Oslomej	205
6	P.K. Barbaras	RG Debreshte	767
7	RG Debreshte	I.B. Prilep	693
8	RG Debreshte	Pump station Norovo	74

The lesser flow rate of the system is due to the lower value of the coefficient of hydraulic resistance provided by Hazen-Williams (C = 120).

6. Conclusions

According the made hydraulic analysis for maximum current flow rateof the system it can be concluded that it is in excellent current "form" because the results do not vary with the predicted sizes of the General project – it can be said that until today the system is maintained in an excellent condition.

If we continue with such commitment in maintaining the system even though it has already exceeded the predicted exploitation period the same will be used in the next thirty years.

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DETERMINING OF RELEVANT HYDRAULIC LOAD ON WASTEWATER WWTPS

D. Vouk [1], D. Malus [2], D. Nakić [3]

Abstract

Hydraulic load is a relevant parameter with regard to dimensioning of certain objects on each wastewater treatment plant (WWTP) - mechanical pretreatment, primary sedimentation tank, secondary sedimentation tank, SBR reactor, membranes, effluent disposal system, etc. In the paper the focus is put on the potential errors that might occur while determining the relevant hydraulic loading of the WWTP. Special emphasis will be given to determining of the relevant hydraulic load in the case of pumping the wastewater to the WWTP from several different directions. A hypothetical sewage system has been created in which the collected wastewater is pumped by three pumping stations at different locations and transported to the WWTP through pressure pipelines. The results emphasize the importance of using the mathematical models when determining the relevant hydraulic load on the WWTP.

Keywords

hydraulic load, sanitary wastewater, WWTP, mathematical modeling

1 INTRODUCTION

In correct planning and designing of the WWTP it is extremely important to define as accurately as possible the input values on which individual solutions and their technical

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properties are based. Along with wastewater quality, the most important parameter among input values is the relevant hydraulic load.

The hydraulic load directly affects defining of the required capacity of the entire WWTP, as well as technical characteristics of separate plant components which are dimensioned in relation to the relevant hydraulic load. This refers to all components of mechanical pretreatment (coarse and fine screen, aerated sand and fat trap, etc.) and of the first phase of treatment (primary sedimentation tanks, micro-screens, etc), as well as to some components of the second and third stage of treatment (secondary sedimentation tanks, SBR reactors, membranes, anaerobic selectors, lagoons, constructed wetlands, etc.), and finally to effluent disposal system (sewers, pumping stations, etc.).

Over-sizing of individual components leads to unnecessary enlarging of structures and equipment capacities and, consequently, operation and maintenance costs. For instance, hydraulic retention time is large beyond plan, both for the sand trap and for primary and secondary sedimentation tanks. At the same time, the equipment installed is oversized, consuming more power resulting in higher operation and maintenance costs. Also, in later replacement of electromechanical equipment, equipment of the same size and capacity must be installed (again larger than necessary) in relation to fixed dimensions of the structure. In membrane treatment technology, the most expensive elements are membranes, whose required area is dependent solely on the relevant hydraulic load. Too low hydraulic load (less than real) results in under-sizing of individual parts of the WWTP. In this case, a possible consequence is reduced efficiency of treatment of the whole plant and impossibility to achieve the required effluent quality. Additional problems occur in the WWTP operation and maintenance. As a rule, such things do not happen in reality, except in case of grave error in calculation.

The hydraulic load of the WWTP is one of the key parameters influencing formation of the unit price of treatment (€/m^3) which is integrated in the total price of water paid by end users.

In case of over-sizing of the WWTP based on incorrect calculation of the relevant hydraulic load, unnecessarily high costs will be reflected on end users and investors. This is important with respect to the fact that construction of a great number of the WWTP is financed from EU funds and government investments.

The purpose of this paper is to indicate potential errors that might happen in defining of the relevant hydraulic load on the WWTP. Particular emphasis will be put on the absence of detailed hydraulic analyses of wastewater flow within the entire outflow and inflow in the WWTP. The point of this paper is put on the problems of defining of the relevant hydraulic load in case of wastewater inflow into WWTP from several different directions. A hypothetical example of the entire sewerage system has been created which corresponds to a real situation, and in which wastewater is brought to the WWTP through three pressure pipelines from three different directions.

2 ERRORS IN DEFINING OF RELEVANT HYDRAULIC LOAD

2.1. General

Relevant hydraulic load of the WWTP which treats the sanitary wastewater is a magnitude which is defined in dependence on a number of parameters, out of which some are of stochastic nature, which makes the entire problem even more difficult.

Reviewing a large number of studies and designs of sanitary wastewater system and WWTPs, the authors of this paper have noticed the occurrence of numerous errors in defining of relevant hydraulic loads. Frequently repeated types of errors include, as follows:

- wrong estimate of the number of connected users during the entire project period,
- wrong estimate of specific inflow of wastewater from the population during the entire project period.
- wrong estimate of relevant quantities of wastewater from industry,
- wrong estimate of magnitude of external water,
- wrong estimate of daily and hourly irregularities of wastewater inflow,
- erroneously defined capacities of pumping stations in pressure inflow of wastewater to the WWTP,
- absence of detailed hydraulic analysis of wastewater flow within the entire sewerage system and inflow to the WWTP.

Hereafter, the importance of detailed hydraulic analyses of wastewater flow within the entire sewerage system will be discussed in more detail.

2.2 Hydraulic analyses of flow within the sewerage system

In carrying out of hydraulic analyses the frequent common practice is to use simple conventional methods, with assumed stationary flow conditions. Thus, relevant hydraulic loads for dimensioning of separate parts of the system, including the WWTP, are defined by the simple method of adding up the inflows from each upstream section of the sewerage network, without taking into consideration a number of influential factors.

Conventional methods cannot describe certain hydraulic situations, and frequently require accepting of results of too high safety factors which finally result in accepting of hydraulic loads higher than real and hence over-sizing of the entire system, including the WWTP. For example, at pressure inflow of wastewater from several different directions, the conventional method implies simple adding up of the capacities of all pumping stations, without taking into account the probability of their simultaneous operation.

The use of mathematical models greatly facilitates conducting of hydraulic analyses of sewerage systems, with presentation of real flow conditions and the additional possibility of testing of a large number of complex scenarios for the purpose of maximum optimization. The complexity of problems also refers to testing of various load dynamics of the system through an arbitrary time period, respecting of system sluggishness, calculation of backwater and control of sewer filling with wastewater, testing of sections where pressure flow occurs and time duration of pressure conditions, comparison of results with differently defined dynamics of pumping stations operation, etc. Taking into consideration the previously mentioned factors, it is possible to define real hydraulic loads within the system, and optimize hydraulic load of the WWTP.

3 ANALYSIS OF THE PROBLEM AND RESULTS

Two hypothetical examples of integrated sewerage system have been created, which correspond well to the real situation. In both examples, wastewater is brought to WWTP by three pressure pipelines from three different directions.

3.1. Description of the first system

The assumed location is the coastal zone with emphasis on developed tourist activity. In the circumstances, maximum load on the system occurs at the peak of the tourist season (last week of July, and first week in August), while for a longer period during the year the system operates with considerably lower capacity (up to eight times lower, compared to maximum load).

The entire system consists of three subsystems (Fig. 1). Each subsystem is characterized by the pumping station (PS) located at its downstream end, which transports wastewater by the corresponding pressure pipeline to WWTP.

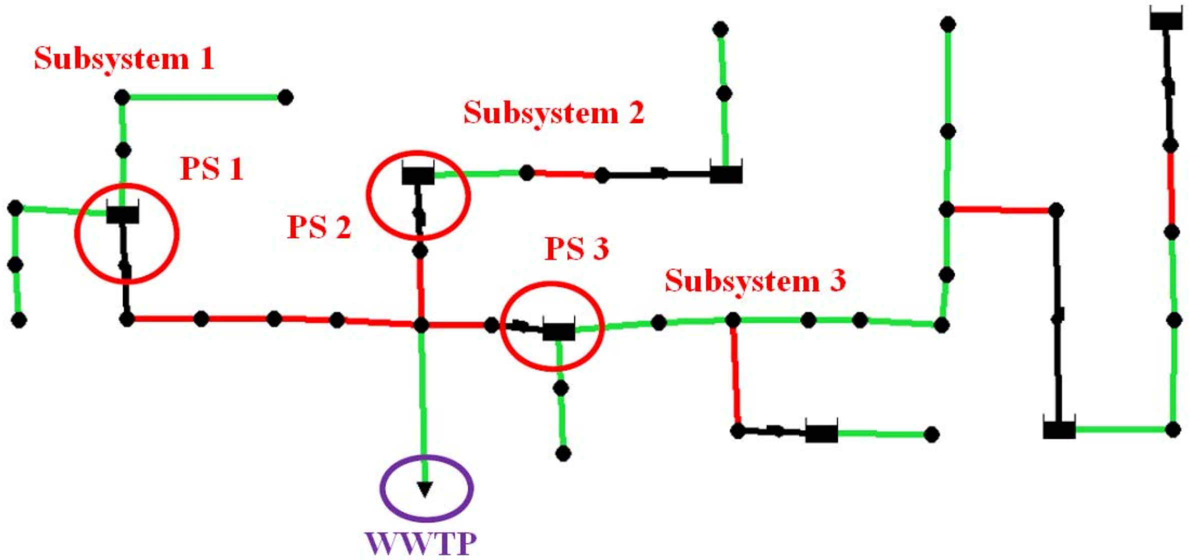

Fig. 1 Diagram of the hypothetical sewerage system (gravity collectors are marked green, and pressure pipelines red)

Tab. 1 Characteristics of the system

System part	Length of gravity collectors (m)	Length of pressure pipeline (m)	Diameter of pressure pipeline (to the WWTP) (DN - mm)	Maximum hydraulic load (l/s)
Subsystem 1	2475	1600	DN 200	30,0
Subsystem 2	3475	350	DN 250	55,0
Subsystem 3	5850	750	DN 250	41,0

3.1.1 Subsystem 1

Subsystem 1 is situated in the western part of the area, with developed tourist industry and a number of hotels and apartments. Wastewater generated within Subsystem 1 are conveyed by gravity to the pumping station by two collector directions, each DN 300, one 1025 m long with average slope of $17^{o}/_{oo}$, and the other 1450 m long with average slope of $11^{o}/_{oo}$. The system is characterized by large slopes, because it is situated in the steep coastal zone. From PS 1 wastewater is transported by the pressure pipeline DN 200, 1600 m long, to the WWTP situated in the central position.

Wastewater inflows originate from the hotels and apartments, and in the hour of maximum inflow they reach 30.0 l/s. This value also includes a part of external water of 50% $Q_{daily,average}$. For the maximum hourly inflow, the pumps manometer head in PS 1 is 40.0 m.

3.1.2 Subsystem 2

Subsystem 2 is situated in the northern part of the area, with developed tourist industry characterized by a large motoring camp and holiday resort. Wastewater generated in the camp and resort is brought to PS 2 by gravity sewerage with additional pumping station situated upstream. The total length of gravity collectors to PS 2 is 3470 m, with average slopes 4-5$^{o}/_{oo}$. From PS 2 wastewater is transported by a separate pressure pipeline DN 250, 350 m long, to the WWTP.
The inflows of wastewater from the motoring camp and the resort, in the hour of maximum inflow, amount to 55.0 l/s. These values include also a part of external water of 50% $Q_{daily,average}$. For the maximum hourly inflow the manometer head in PS 2 is 36.0 m.

3.1.3 Subsystem 3

Subsystem 3 is situated in the eastern part of the area and comprises tourist accommodation units in the motoring camp and local population in three minor settlements. Wastewater is transported to PS 3 by a gravity collector DN 300. Subsystem 3 is characterized by three pumping stations in the upstream part of the system. The total length of the main gravity collector to PS 3 is 5150 m, with average slope of 17,5$^{o}/_{oo}$. From the southern side wastewater is brought to PS 3 by a smaller gravity collector DN 300 of total length of 700 m and average slope of 25$^{o}/_{oo}$. From PS 3, wastewater is transported to WWTP by the pressure pipeline DN 250, of total length of 750 m.
Wastewater inflow comes exclusively from the population, motoring camp and private apartments, and in the maximum inflow hour amount to 41.0 l/s. Thus, the total value of the maximum hourly inflow is 41.0 l/s. This value includes a part of external water of 50% $Q_{daily,average}$. For the maximum hourly inflow the manometer head in PS 3 is 33.0 m.

3.1.4 Hydraulic analyses

The most frequent practice is to add up the capacities of all operating pumps, which means assuming simultaneous operation of all pumping stations transporting wastewater to the WWTP. In the analyzed hypothetical case, relevant hydraulic load of the WWTP for simultaneous operation of all pumping stations is 132 l/s, which may be seen from results of the hydraulic calculation carried out on the mathematical model (Fig. 2). The mathematical model used was EPASWMM version 5.0.021. In defining of pumps in the mathematical model real Q/H curves were used.
By time regulation of pump operation, various scenarios were tested which do not include simultaneous switching on of the pumps, not even in the moment of maximum hydraulic load of the system. The results of the analyses have shown that it is not possible to achieve satisfactory hydraulic-operational flow conditions if at a moment only one pump is operating.
Namely, in this case, at the moment of maximum load of the system considerable backwater would occur, with flooding of the surrounding terrain. However, additional analyses have shown that it is possible, regulating the pumps operation with two minor pumps operating simultaneously (PS 1 and PS 3) and the third one (PS 2) operating separately, to achieve favorable hydraulic-operational flow conditions. In this situation, maximum hydraulic load of WWTP is 83 l/s (according to results obtained on the mathematical model).

Also, in the analyzed scenario another operating regime of minor upstream pumps was applied in order to reduce maximum load on PS 2 and PS 3, i.e. peak inflows from upstream pumping stations were delayed. With such regulation of pump operation, backwater before WWTP occurs, but it has been proved on the model that the pressure line is considerably below the terrain line, and there is no risk of flooding (Figs. 4 through 6). As the terrain slopes, as well as slopes of the gravity collectors in the hypothetical sewerage system are considerable, retention potential of pipelines upstream from pumping stations has not been fully utilized. Applying of the above measures of regulation of operation of pumps before WWTP reduces relevant hydraulic load for dimensioning of the plant by approximately 37 percent.

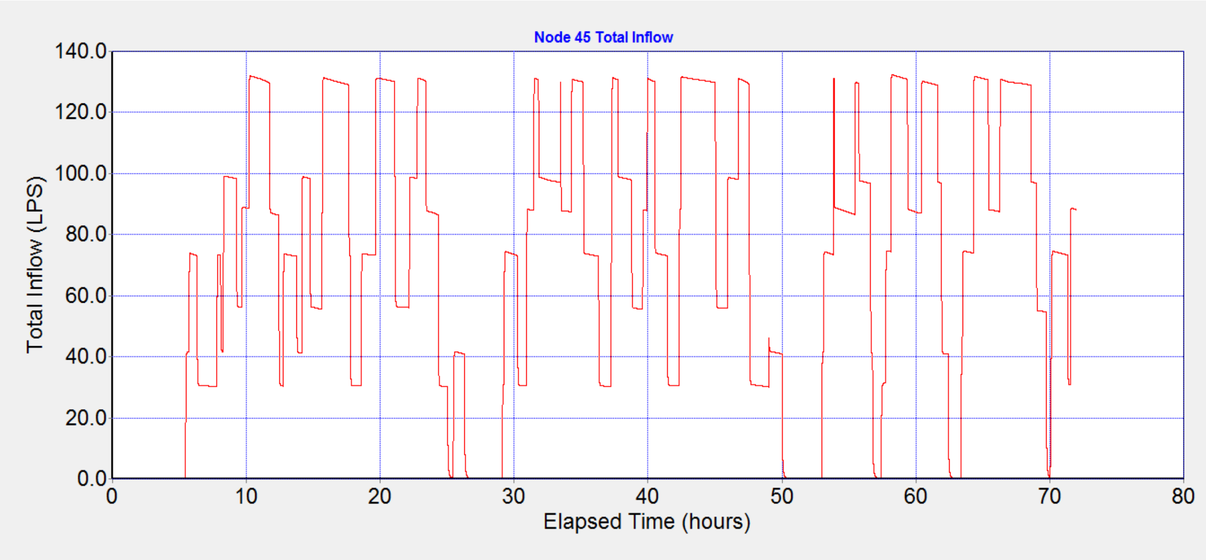

Fig. 1. Total inflow to WWTP when PS 1, PS 2 and PS 3 are operating simultaneously

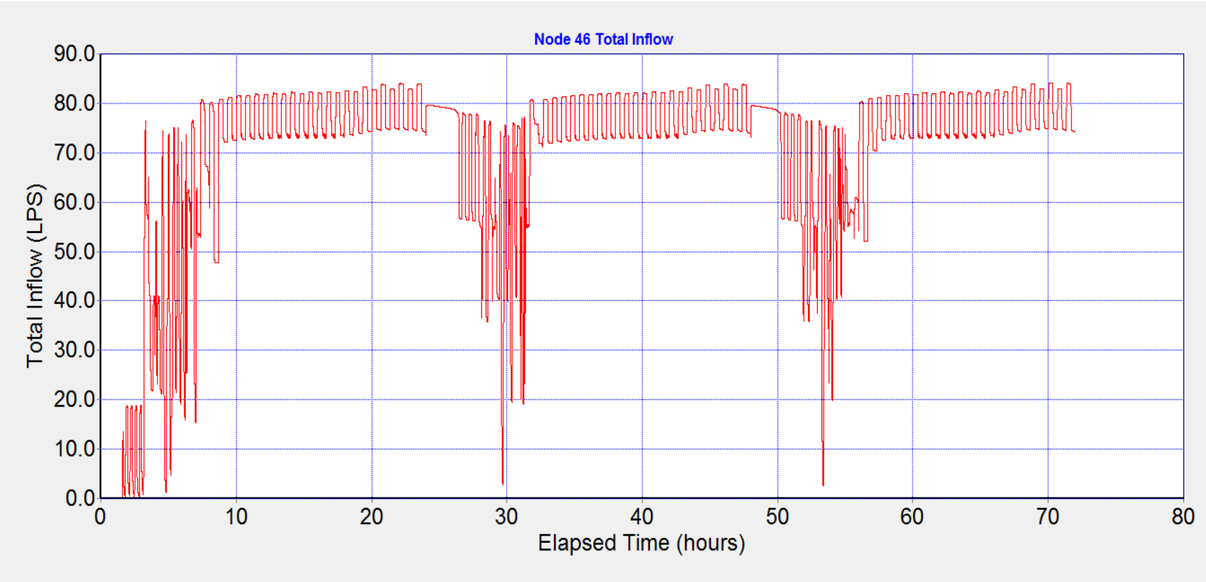

Fig. 2. Total inflow to the WWTP with optimization of pump operation when PS 1, PS 2 and PS 3 are not operating simultaneously

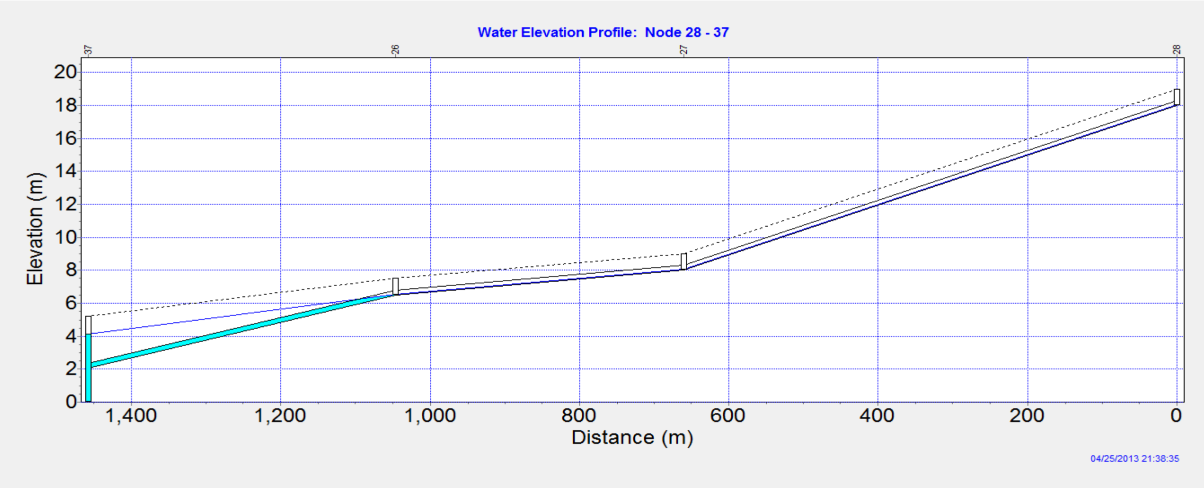

Fig. 4. Longitudinal profile of northern section upstream from pumping station PS 1 at the moment of maximum load

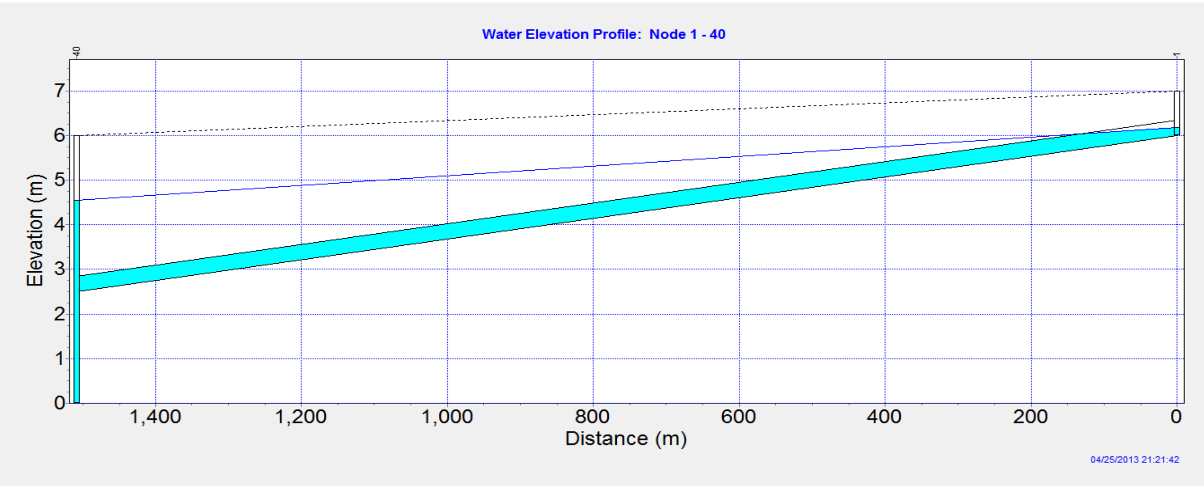

Fig. 5. Longitudinal profile of the section upstream from pumping station PS 2 at the moment of maximum load

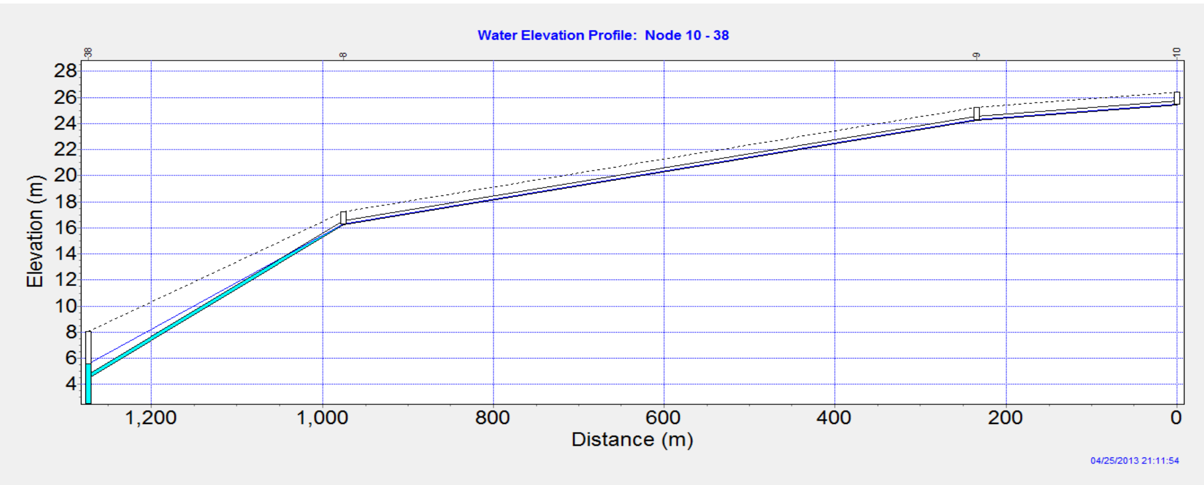

Fig. 6. Longitudinal profile of the western section upstream from pumping station PS 3 in the most unfavorable moment

3.2 Description of the second system

Another sewerage system has been modeled on the identical diagram (Fig. 1), consisting of three subsystems. Each subsystem is characterized by a pumping station situated on its downstream end, transporting wastewater through a corresponding pressure pipeline to the WWTP. Each subsystem is characterized by connecting of a minor community of permanent users. Subsystem 1 is presented as a community with 500 connected users, subsystem 2 with 1000 users, and subsystem 3 with 750 users.

Tab. 2 Characteristics of the system

System part	Length of gravity collectors (m)	Length of pressure pipeline (m)	Diameter of pressure pipeline (to the WWTP) (DN - mm)	Maximum hydraulic load (l/s)
Subsystem 1	500	1600	DN 90	2.74
Subsystem 2	1000	350	DN 90	4.50
Subsystem 3	750	750	DN 90	3.60

3.2.1 Subsystem 1

Subsystem 1 is situated in the western part of the area. Wastewater generated within Subsystem 1 is transported by gravity to the PS through two collectors as described in Chapter 3.1.1. From PS 1, wastewater is transported by the pressure pipeline DN 90, 1600 m long, to the WWTP in the central position. The relevant hourly inflow in PS is 2.74 l/s. This value includes a part of external water of 50 percent of $Q_{daily,average}$ or minimum 1.0 l/s.

3.2.2 Subsystem 2

Subsystem 2 is situated in the northern part of the area. Wastewater generated within Subsystem 2 is brought by gravity to PS 2, with interpolation of an additional pumping station situated upstream. The characteristics of the gravity pipeline network are described in Chapter 3.1.2. From PS 2, wastewater is transported by a separate pressure pipeline DN 90, of total length of 350 m, to the inflow manhole before the fine screen in the WWTP. The relevant hourly inflow of wastewater in PS 2 is 4.5 l/s. The above value includes the part of external water of 50 percent of $Q_{daily,average}$ or minimum 1.0 l/s.

3.2.3 Subsystem 3

Subsystem 3 is situated in the eastern part of the area. Wastewater is brought to the pumping station PS 3 by the gravity collector DN 300. Basic technical characteristics of Subsystem 3 are described in Chapter 3.1.3. Wastewater is transported from PS 3 to the WWTP by the pressure pipeline DN 90 of total length of 750 m. The relevant hourly inflow of wastewater to PS 3 is 3.6 l/s. The above value includes the part of external water of 50 percent of $Q_{daily,average}$ or minimum 1,0 l/s.

3.2.4 Hydraulic analyses

Just like in the previous example, relevant hydraulic load of the WWTP was calculated first assuming simultaneous operation of all three pumping stations. In relation to magnitudes of

wastewater inflows and relevant manometer heads, the capacities of all three pumping stations were determined as 7.0 l/s. In defining of pumps on the mathematical model real Q/H curves were used. With simultaneous operation of all three pumping stations, relevant hydraulic load of the WWTP is 21.0 l/s.
Further, the possibility of reducing the hydraulic load of the WWTP was examined, with regulation of pump operation in PS 1, PS 2 and PS 3. Different scenarios were tested, without simultaneous switching on of pumps, not even at the moment of maximum load on the system. The results of the analyses have shown that it is possible to achieve satisfactory hydraulic operating flow conditions if at a given moment only one pumping station is in operation. Namely, in that case, at the moment of maximum load there would be a negligible backwater, but there would be no pressure flow in any of the stations (Figs. 8 through 10). The resulting maximum hydraulic load of WWTP is 7.0 l/s, which corresponds to the capacity of one pumping station (Fig. 7). By these measures of regulation of pump operation the relevant hydraulic load for dimensioning of the plant is reduced by approximately 66 percent.

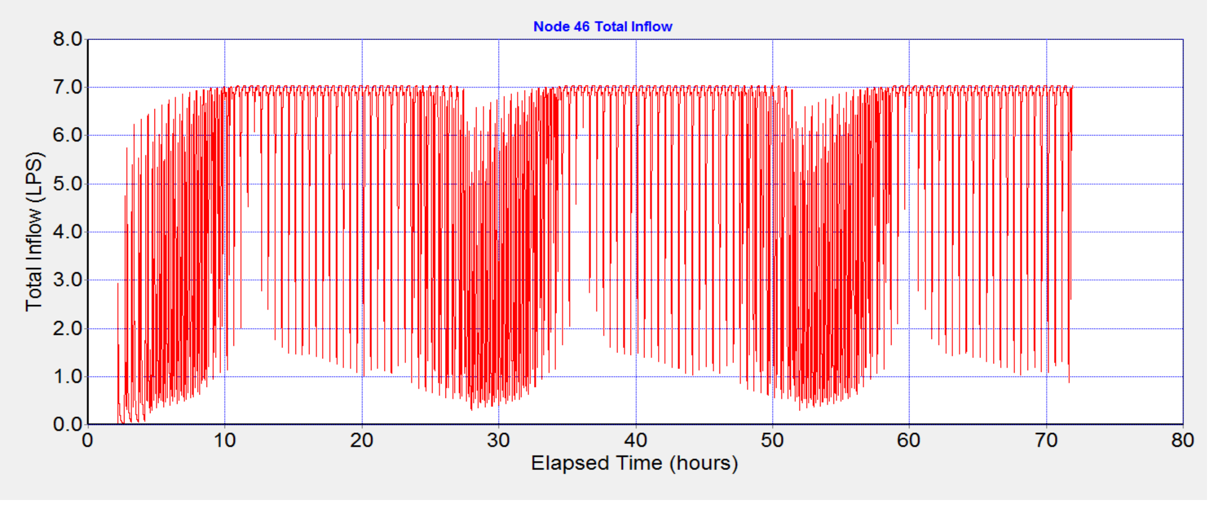

Fig.7. Total inflow to the WWTP with optimization of pump operation when PS 1, PS 2 and PS 3 are not operating simultaneously

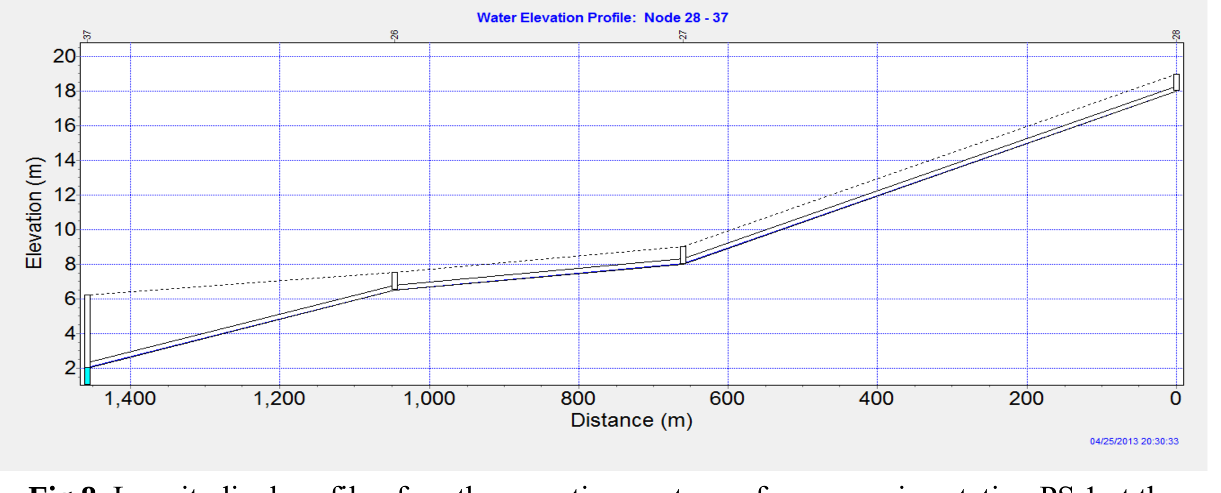

Fig.8. Longitudinal profile of northern section upstream from pumping station PS 1 at the moment of maximum load

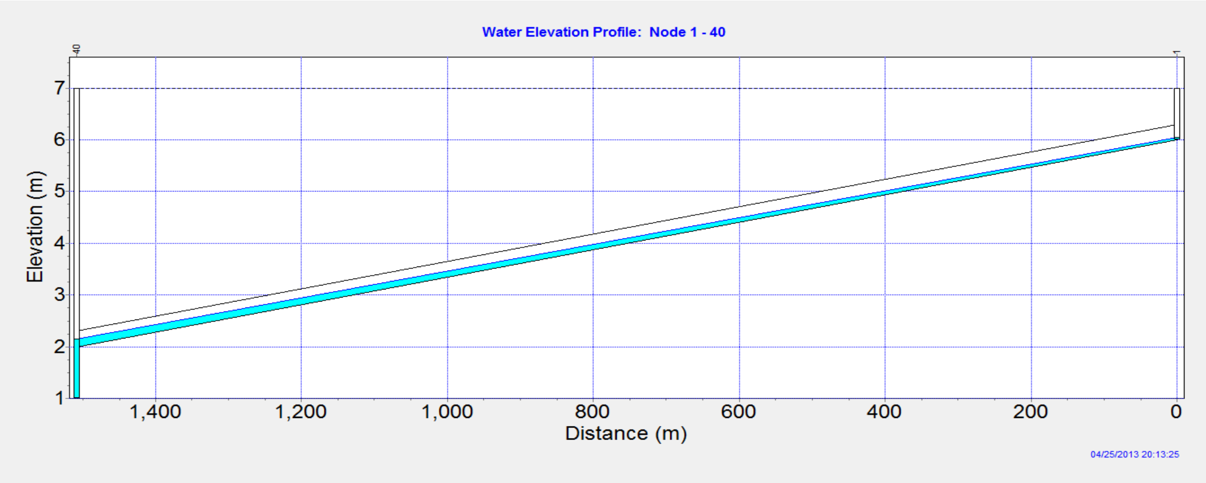

Fig.9. Longitudinal profile of section upstream from pumping station PS 2 at the moment of maximum load

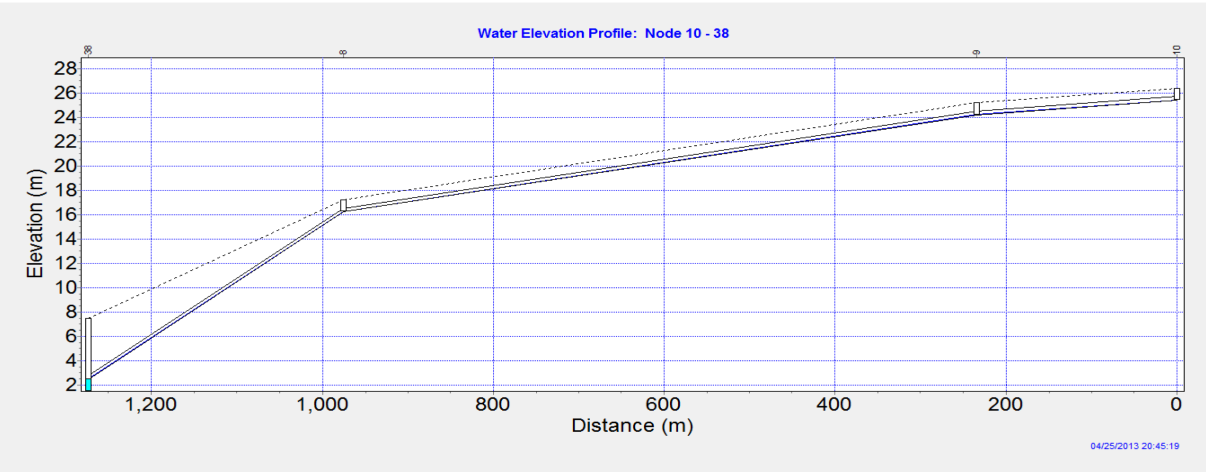

Fig.10. Longitudinal profile of western section upstream from pumping station PS 3 at the most unfavorable moment

4 CONCLUSION

Defining of the relevant hydraulic load of the WWTP is a question that must be assigned particular importance, already from the initial phases of working out of the design documentation. Using of simple procedures of hydraulic calculation may lead to over-sizing of some components of the WWTP, which results in numerous operational and economic drawbacks. Complex hydraulic situations cannot be described by stationary models, and errors resulting from such practice cannot be tolerated due to great damages they cause. The results of this paper emphasize the importance of application of mathematical modeling in defining of relevant hydraulic loads of the WWTP, as well as the convenience offered by automatic regulation of operation. Namely, in present time of availability and easy accessibility of various mathematical models, it is recommended to analyze on the model characteristic hydraulic conditions, including defining of the relevant hydraulic load of the WWTP in any wastewater system.

In pressure inflow of wastewater to the WWTP from several different directions, it is considered rational and reasonable to examine the possibility of regulation of the pumping station in the way to exclude their simultaneous switching on. In this way, it is possible to reduce hydraulic load of the WWTP. The demonstrated example is not the only one which may justify the use of complex mathematical models in order to optimize defining of relevant hydraulic loads for dimensioning of all relevant elements of the sewerage system.

MATHEMATICAL MODELING OF WATER SUPPLY SYSTEMS WITH FOCUS ON THE WATER LOSS ANALYSIS

D. Vouk [1], D. Malus [2], I. Halkijević [3], D. Nakić

Abstract

Mathematical modeling of the water supply system provides quick and effective insight into the real hydraulic operating conditions regarding flow and pressure. Some models offer the possibility of real-time water loss modeling, defining water losses in terms of pressure change. Therefore, instead of considering leakage as a constant value within the certain nodes, the leakage flows were expressed as an emitter flow in terms of pressure. In this way, the opportunity was made to monitor leakage changes in real time in relation to pressure changes within the system. This provides the possibility to get an insight into real values of water loss reduction after implementation of recommended optimization measures. This paper describes the application of mathematical modeling for water loss analysis, by considering leakage as an emitter flow. The proposed methodology is analyzed on an actual example.

Keywords

mathematical modeling, water supply, water losses, calibration, pressure control, emitter coefficient

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1 INTRODUCTION

Analysis of public water supply systems (hydraulic operating conditions, water hammer, leakages, defining of adequate technical solutions, improvement of existing systems, etc.), in particular of the systems of more complex nature (large number of sections linked in rings, nodes, pumping stations, water storage tanks) is almost unthinkable without application of mathematical (numeric) models which can simulate various situations respecting all specific properties of the particular system.

In the framework of worldwide practice [1], application of various mathematical models results in abandoning of conventional methods of water supply systems which can hardly describe certain hydraulic situations. Also, the concept of water supply systems analysis based on momentary data or short episodes is being abandoned, replaced by longer periods resulting in more reliable results and higher quality conclusions.

Leakages are an unavoidable phenomenon in any water supply system. Therefore, defining of good quality measures of improvement of existing water supply systems greatly depends on comprehensive and precise description of the actual condition, which must include realistic presentation and analysis of leakages within the system. Only a thorough analysis of leakages within the system allows making of adequate conclusions, which will, through defining of measures of sustainable system management (extension and optimization), result in rational spending of available financial resources. In his context, realizing and analysis of leakages from a wider aspect is necessary, the analysis of leakages by mathematical models being an unavoidable segment.

In Croatian engineering profession, analyses of leakages are oriented to mathematical modeling only to a negligible extent. Even in cases when models are used, it is noticed that a large number of available possibilities of various models remain unused. Schematic presentations of the system are frequently simplified by reducing the number of sections, junctions, etc., which is, from the aspect of leakage analysis, considered inadequate. Likewise, simulations of characteristic hydraulic conditions are frequently done without calibration of the model on the set of measured data, which would confirm the credibility of the results obtained on the model and define the real value of actual leakages and their distribution.

By all means, in defining of optimum measures to reduce leakages, using of various possibilities offered by mathematical models is considered appropriate and desirable, with previous adequate processing and preparing of real values of input data. This implies more advanced forms of mathematical modeling with the possibility of monitoring of leakages in real time, where leakages on the model are defined as variables dependent on pressure within the system, which corresponds to the actual situation.

This paper will demonstrate an example of hydraulic optimization of the existing water supply system of the town of Velika Gorica, from the initial phase of field investigations and adequate processing of their results, which will further serve for calibration of the actual condition of the system, followed by optimization and defining of future solutions in order to reduce total leakages.

2 MATHEMATICAL MODELING

2.1. Aims and possibilities

The aims of mathematical modeling of water supply systems are manifold. Mathematical modeling of water supply systems facilitates presentation of hydraulic operating conditions, i.e. presentation of real flow and pressure situations within the water supply network.
The use of the model greatly facilitates and accelerates conducting of hydraulic analyses of water supply systems with reduced probability of errors. Models offer a simple possibility of examining of a large number of complex scenarios, with optimization in mind. Problem complexity refers to:

- simulation of dynamic situations in unlimited period,
- examining of different dynamics of water consumption in arbitrary periods (monthly, daily or hourly irregularities of consumption) at the same point allows defining of several different categories of consumers with different consumption irregularities,
- comparison of results with differently defined dynamics of pressure units operation (pumps),
- modeling with constant and variable speed drives (pump frequency regulation),
- pump operation cost estimate (from power estimate for pump operation),
- modeling of influence of various regulation devices – pressure reduction valve, flow control valve, etc.
- modeling of complex (multizonal) systems with larger numbers of storage tanks, with possibility of defining of filling and emptying dynamics for each storage tank,
- modeling of water age,
- conducting of leakage analysis,
- modeling of water losses in real time.

Among other things, models offer high quality graphic presentation of results, facilitating their reviewing and checking.

2.2 Mathematical models

The paper points out, first of all, the advantages of the program package EPANET [2], [3]. The model was developed by USEPA (United States Environmental Protection Agency). EPANET is a computer program for time simulations of hydraulic conditions within the pressure pipe system (water supply network), but also of water quality (among others, the age of water in separate points in the system. EPANET also offers the possibility of modeling of leakages in real time, i.e. modeling of non-stationary water leakages in dependence of pressure variations within the system. In most systems, non-stationary conditions of pressure and, consequently, of leakages, are expressed. By means of certain regularities [4] it is possible to calculate on the model the leakages at individual nodes depending on the momentary pressure. Great advantage of EPANET model is its easy accessibility. Namely, it can be, as a completely free tool, be taken off from the Internet. It may be useful both to designers and to the investors. EPANET as a free tool can be installed and used on all computers in the company. In case of purchase of commercial software, most frequently, except for one license, this requires additional financial resources.

Likewise, it is appropriate to mention other commercial models, like WATER-GEMS of the firm Bentley, WATER-CAD developed by Haestad Methods and later bought by Bentley, and MIKE-URBAN developed by experts of DHI.

Each of the above models has the possibility of direct forming of EPANET file, as well as of introducing of the previously developed model in EPANET. Therefore, it would not be fair from the investor to limit the designers to use any software, but require in the terms of reference delivery of EPANET file in digital version that may be later freely used by the investor.
Although the authors suggest the use of EPANET as the optimum solution, it is fair to point out the advantages of commercial models. Their advantages result from better graphic support, direct integration with GIS tools (although nowadays EPANET in a simple way may be connected to GIS tools, using separate modules). Further, speaking about advantages of commercial models, there is the possibility of simulation of the water hammer and testing of different procedures to reduce its impact. A great advantage is integration of genetic algorithms which allow automatic model calibration in relation to various parameters, and offer various possibilities of analysis and detecting of water losses [5], [4], [6], [7]. However, this requires extreme care in dealing with systems of a more complex nature. So far, the authors' experience indicates that in some cases model calibration using genetic algorithms required considerably more time than manual calibration of the same models on EPANET.
The great advantage of application of commercial models with integrated genetic algorithms is the possibility of better analysis of water losses. By such models it is possible to predict in real time the locations of new ruptures of pipe networks and leakage [8]. Such practice also requires a sound database on the condition of pipe network (pipe material, pipe age, etc.). It is also exceptionally useful to have a sound database on failures in the pipe network. The representatives of municipal utilities are urged to establish the practice of keeping the logbook of interventions, if they have not done it yet (the database must include information on the location of the failure, date and time when the failure was noticed, time required for repair, type of failure, the pipe profile where the failure was noticed, pipe material, description of pipe damage, leak intensity, how the failure was repaired, what were the costs of repair, description of the basic characteristics of the new situation, and other relevant information)
At present, availability of such information may seem unimportant, but in sound analyses of leakages and defining of optimum measures for reduction of losses it is very valuable

2.3 Methodology of mathematical modeling of water losses

Methodology of mathematical modeling in analysis of water losses and in sustainable management may be divided in several basic steps:

- ❑ Creation of the preliminary modes of the actual situation with analysis of relevant hydraulic conditions and possible irregularities in normal operating conditions.
- ❑ Preliminary division of the system into zones with checking of new hydraulic conditions caused by closing of section valves.
- ❑ Calibration of the mathematical model of the actual situation:
  (a) with defining of fixed values of water loss
  (b) with modeling of water losses depending on pressure.
- ❑ Analysis of water losses on calibrated mathematical model.
- ❑ Final division of the system into zones (DMAs) with checking of new hydraulic conditions due to closing of section valves and installation of pressure reduction valves.
- ❑ Analysis of technical solutions of pressure regulation (possibility of examining of a large number of different scenarios).
- ❑ Optimization of (re)pumping stations operation with estimate of operating costs.

- ❑ Optimization of dynamics of storage tanks filling and emptying.
- ❑ Predicting of locations of new pipe network failures and leakages.

Independent from the way and methodology of mathematical modeling, absolute accuracy of certain design parameters in separate parts of the model depends primarily on the quality of available data (real picture of construction status).
The first three steps of the suggested methodology are, in the authors' opinion, the most complex and demanding, and at the same time the most significant from the standpoint of sustainable management of the water supply system. Therefore, each of them is described below in more detail.

2.3.1 Creating of the preliminary model of actual situation

The purpose of working out of a sound model of the actual situation is to ensure its permanent value. Namely, modeling of present situation with model calibration is the basis for planning of all future activities, i.e. defining of correct technical solutions, not only of system extension, but also of implementation of adequate improvement measures for its optimization (control and reduction of leaks). This implies improvement of the entire system including extension pressure control, optimization of pumps operation, filling and emptying of the storage tanks, etc.
To be able to calibrate the model properly, it is important to have reliable data on flow and pressure at typical points of the system, which may be obtained only by field measurements. All relevant aspects determined by preliminary actions and analyses (which includes field measurements) are integrated in the mathematical model of the actual situation.
It is desirable to have as many measurements as possible, during longer periods (minimum 24 hours). In some segments it is possible to measure flow and pressure only during the night period with minimum consumption (01-05 h). The larger the number of measurements and required measuring instruments, the higher the cost of field investigations. Therefore, keeping in mind the financial possibilities of the investor and time limitations for conducting of analyses it is important to define, already in the initial phase, a sound plan and program of measurements.
This requires dividing of the system into zones which is in agreement with suggested methodology of water loss analysis, on the basis of analysis of the results obtained on the preliminary mathematical model and noticing of corresponding hydraulic regularities.

2.3.2 Preliminary division of the system in DMAs

Preliminary division off the system into DMAs allows better and more efficient campaign of flow and pressure measurements and subsequent model calibration, as well as more detailed understanding of the water loss problem in the entire system.
The advantages of mathematical modeling are reflected through the possibility of simple, rapid and efficient finding of the optimum division of the system into DMAs, as well as of later control. Optimum zoning of the system implies dividing of the system into a corresponding number of DMAs, taking into account the field and technical properties of the system, availability and possibility of the utility to carry out proper control of each DMA. This includes technical status of the equipment of the utility and its financial possibility to procure additional items. Namely, analysis carried out on paper can stand all requirements, but in practice problems may arise due to impossibility of implementing them. In any case, for better control of water losses it is desirable to divide the system into as many smaller DMAs as possible, with continuous and simultaneous control of flow and pressures in each

DMA. Such practice requires a large number of flow and pressure measurement units, which often becomes the limiting factor with regard to insufficient financial resources of the utility.
Further, specific technical properties of the system imply the present status of development, including all water supply accessories (locations and characteristics of existing section valves, existing regulation equipment, etc.). Hydraulic properties of the system are important as well (distribution of flow and pressures within the water supply network, etc.). In zoning of the system, it is considered optimum to provide one entry into the zone and carry out flow and pressure measurement unit at that point. If water enters into a zone at several locations, it is necessary to install the flow and pressure measurement unit at each entrance. Such practice requires higher financial means for procurement of measurement units. Also, processing and analysis of measured data becomes in such case more demanding. It is important to note that the possibility of error in estimating of the situation in relation to water losses becomes higher with a larger number of flow measurement units controlling water entrance and exit into and from the DMA. If one entrance of water into the DMA is to be provided by closing connecting pipes to other DMAs (by existing or newly designed section valves), considerable changes of hydraulic conditions are possible. In this case, good knowledge of hydraulic conditions within the DMA is required in order to ensure unimpeded water supply to the population and industry in fire-fighting situations. This is the segment where the advantages of application of mathematical models become clearly visible.

2.3.3 Calibration of the mathematical model of the actual situation

Each model whose basic design parameters and system characteristics are defined on the basis of available design documentation and field visits, without detailed basic data (actual situation, geodetic survey data, knowledge of distribution of water consumption, consumption irregularities) must be subject to calibration.
Model calibration is carried out by interpolation of flow and pressure values, obtained by measuring, into the model. In the sequence of iteration steps of varying of input parameters the intention is to adjust the model as much as possible to the actual state determined by measurements. In this process the following parameters are varied: input values of water flow, water consumption in separate nodes, pipe roughness, magnitudes of loss in separate zones/subzones.
It should be mentioned that it is desirable to carry out model calibration in non-stationary conditions, i.e., in conditions of varying consumption within a period of one or several days. Non-stationary flow conditions are caused, in addition to irregular water consumption by the population and industry, also by varying pump operation at the wellfield and by varying dynamics of filling and emptying of the storage tanks.
In a calibrated mathematical model all characteristic categories of consumers should be defined separately, with corresponding irregularities of hourly consumption, including the category of water losses. In this process, water consumption for the population, industry, firer fighting and water losses are defined separately. For each typical category of consumers the corresponding irregularity should be defined. It is also possible to define different irregularities of water consumption for the population in different zones, which is determined and justified on the basis of processing of results of field investigations.
Actual losses determined in separate zones and subzones (based on measurement results) are distributed in doses within each zone/subzone. Losses are not evenly distributed within separate zones and need not be defined in the model in this way. Depending on pipe materials, profiles, age of the system, data on general condition of separate sections, distribution of

pressures, etc. it is considered reasonable to distribute the total quantity of actual losses in a zone into several nodes. Calibration of the model of actual situation is achieved, among other things, by varying of quantities of water losses in individual nodes. In the initial phase, actual losses are defined as fixed values, unchangeable in time.
After achieved matching of the results of the mathematical model with measured values it is confirmed that the model credibly describes real situations in the analyzed system.
In worldwide practice numerous methodologies are used to define actual quantities of losses from the measured data, which continue to develop. It is important to understand all components of actual losses, including background leakages [4], [9].
In the next step of model calibration, the magnitude of water losses should be defined in dependence on the magnitude of pressure, which is a constant variable within the system. Water losses are in this case defined through the “emitter” coefficient:

$$K_i = \frac{q_i}{(H_i)^{N1}} \tag{1}$$

where:

K_i emitter coefficient (leakage coefficient) [1]
q_i water losses in the analyzed node [l/s]
H_i pressure head in the analyzed node [m]
N1 exponent of leakage sensitivity depending on pressure [1]

This approach gives even more precise results on the model, because the value of losses in a separate node is not constant, but changes with time depending on the pressure, which corresponds better to the real situation.
The exponent of leakage sensitivity in dependence on the pressure (N1) usually varies within the range from 0.5 to 1.5 for separate zones and is dependent on the prevailing type of fissures and on the kind of pipe material (rigid or elastic pipes). It is often assumed (for the sake of simplicity) that the average value of N1 exponent in large systems with different pipe materials is 1.0, implying linear connection between the value of loss (leak) and the pressure. However, for the major portion of rigid materials (AC, CI) the value of N1 exponent approaches 0.5, while with the higher portion of plastic pipes (PVC, PEHD, PP) the value of N1 coefficient comes close to 1.5. The reason is that losses manifested as background leaks, as well as leaks on the primary and secondary network that may be detected from elastic (plastic) pipes have the N1 value close to 1.5 and are very sensitive to pressure variations. On the other hand, fissures that may be detected in rigid pipes (CI, AC) have the N1 value close to 0.5 and are less sensitive to pressure variations. Fig. 1 shows in graphic form the dependence of leakage (water loss) changes on the value of pressure at various values of N1 exponent. Thus, for instance, reduction of pressure from 6.0 bar to 3.0 bar, with N1 exponent of 1.25 results in 58 percent reduction of water losses on existing damage points, at the same time reducing the probability of new damages and prolonging the pipe life.
It is possible to determine the magnitude on N1 exponent for individual zones of the system by field investigations. This requires measuring during the night period with minimum consumption.

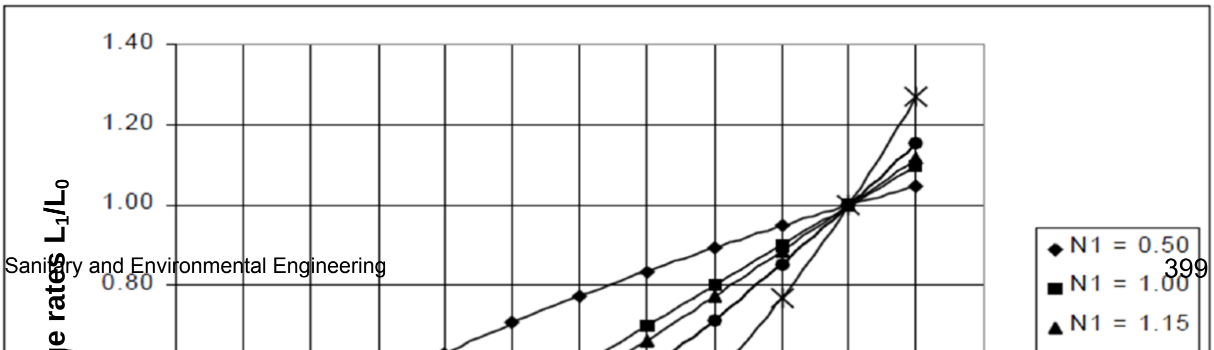

Fig. 1 Dependence of leak values on pressure increasing or reduction within the system, for different values of N1 exponent [4]

On the mathematical model, the magnitude of loss is determined by defining of N1 exponent (as “emitter” exponent) in basic hydraulic settings of the model. N1 exponent should be defined at the level of the system as a whole, because in the EPANET model it is not possible to define the value of N1 exponent for separate parts of the system. Along with N1 exponent it is necessary to define the “emitter” coefficient in corresponding nodes, and the model within its own mathematical algorithm autonomously calculates the magnitude of losses in each defined time step, depending on the relevant pressure (according to Equation 1).
Model calibration implies a sequence of iterative steps in which the values of emitter coefficients and the exponent N1 are changed.
A more advanced form of mathematical modeling of water losses (with defining of their dependence on the pressure) allows a more realistic and precise picture of the hydraulic situation in the water supply network, more precise matching of output results in comparison with results of field investigations during model calibration, and consequently, a more precise economic analysis of water losses and better comparison of different alternative solutions of optimization of the entire system, with suggested measures for reduction of losses.

3 ANALYSIS AND RESULTS

3.1. Description of the system

The object of the analysis is the water supply system of the town Velika Gorica. In the present situation, water supply is secured from one wellfield, in the form of ground water abstraction in the total of five wells. The public water supply system Velika Gorica is characterized by the combination of circular and branching water supply network with pipes of various materials, various profiles and various age (up to 50 years).The total length of the water supply network in the entire area is about 500 km, with the population of 42,500 connected consumers and several industrial facilities. The analysis of the water supply system Velika Gorica was carried out in accordance with the methodology described in Chapter 2.

3.2 Mathematical modeling and calibration

After collecting of available data and determining of the characteristics of the system, the preliminary mathematical model of the present situation has been made. In working out of the model, the computer program EPANET, version2.00.12 was used. The mathematical model of the present situation involved a large number of nodes, section, reservoirs and pumping stations. Generally, the model involves the following elements:

- 1792 nodes,
- 2057 pipes – total length 476 km.
- 1 wellfields with 5 abstraction points (drilled wells),
- 5 water reservoirs,
- 7 pumping stations, out of which 5 in the wellfield, and 2 interpolated within the water supply system.

Following the analysis of the results on the preliminary mathematical model, the entire system has been divided into seven characteristic zones (Fig. 2). Individual zones have been divided into a number of minor subzones. For each zone, the plan of measuring of flow and pressures was prepared. Seven-day measurings of flow and pressures were carried out separately for each zone. In some zones, additional diurnal measurings of flow and pressures were done within subzones. Generally, the measurements included 80 flow measuring points and 65 pressure measuring points.

Field measurements of flow during the continuous period of 7 days determined the real situations of water consumption irregularities that correspond to individual zones/subzones. Hourly irregularities of consumption have been defined for 13 characteristic subzonal units.

Calibration of the model of the existing situation was carried out in a sequence of iterative steps, by interpolation of pressure and flow values obtained by measurements into the model. In each step, individual input parameters were changed in an effort to adjust, as much as possible, to the real situation determined by measurements. The parameters that were changed include: Q-H curve of pumps in the wellfield, water consumption in individual nodes, pipe roughness, and the magnitudes if losses in individual nodes, or emitter coefficient.

On the calibrated mathematical model, the analysis of characteristic hydraulic situations within the entire system was carried out, for the whole-day operation regime, with particular attention to characteristic periods with minimum and maximum water consumption.

In a major part of the system (Zone 1, Zone 3, Zone 4, and Zone 5) in the whole-day operation regime pressures up to 5 bars are generated, which may be considered satisfactory, although the possibility of additional improvement measures is pointed out which may additionally reduce dynamic pressures in some parts of the system.

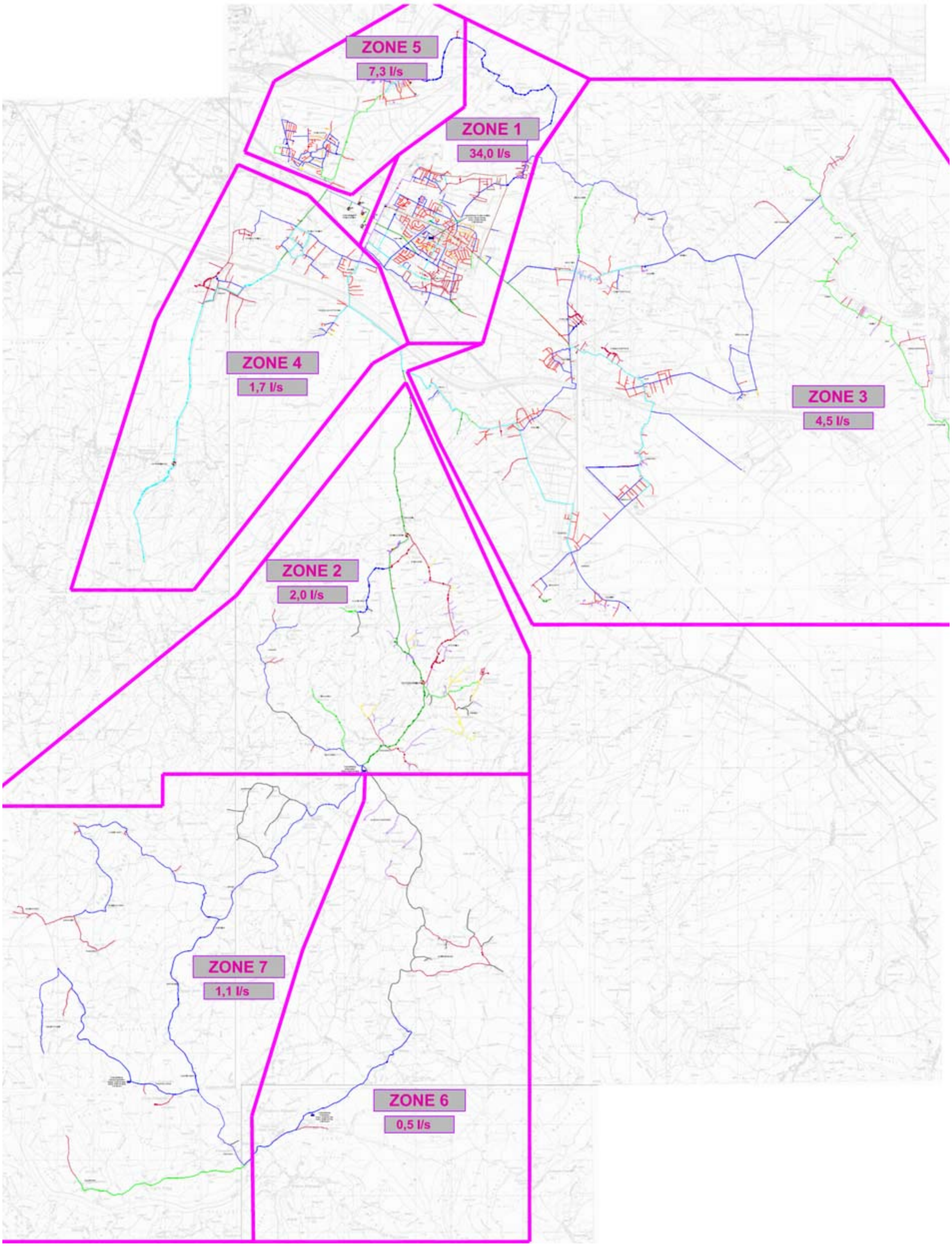

Fig. 2 Situation map of the public water supply system Velika Gorica with division into zones and present distribution of actual water losses

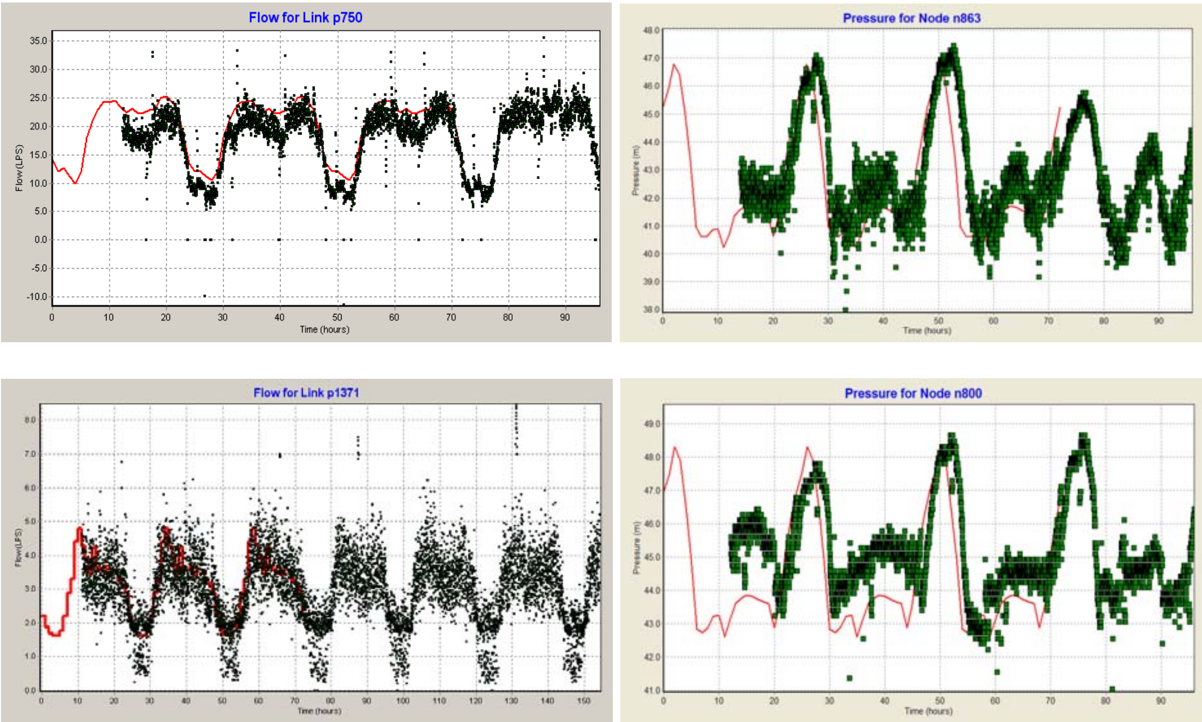

Fig. 3 Results of model calibration for individual points of the system

3.3 Analysis of water losses in actual situation

In continuation, the analysis of water losses was carried out. It may be concluded on the basis of the analysis on the calibrated model of the actual situation that the portion of total losses generated as real losses (leakages) is considerable. The detected water losses are not evenly distributed along the entire system (Fig. 2). In certain parts of the system (Zone 1 and Zone 5) considerable losses are present, while in some other parts of the system water losses do not occur. Additionally, distribution of losses in subzones was also analyzed, and it was noted that distribution of losses within zones was not uniform either.

Actual losses may, on the basis if the actual condition of the entire system, may be assessed as considerable, amounting about 35 percent (50 l/s) in relation to the total quantities of abstracted water. Thus, there is evident need to implement priority activities and measures to improve the entire system. This primarily implies regulation of pressures within the water supply network.

3.4 Priority activities for reduction of losses

In the case of the water supply system Velika Gorica, occurrence of considerable quantities of total losses in some parts of the system (Zone 1 and Zone 5) may be attributed to numerous factors – age of some sections of the pipe network consisting of CI, AC and PVC pipes, and to the occurrence of pressures exceeding 5.0 bar, even in the regime of maximum consumption.

Thus, the priority activity in reduction of losses is mounting of pressure reduction valves to reduce the pressure in sections where the results of the analysis of the present situation indicate pressures higher than necessary.

Tab. 1 Recapitulation of calibrated values determined by measurements on the model

Water Supply System	DMA	System input volume (m^3/d)	DMA input volume (m^3/d)	Authorized consumption			Real water losses		
				Population (m^3/d)	Industry (m^3/d)	Total (m^3/d)	(l/s)	(m^3/d)	(m^3/a)
Velika Gorica	Zone 1	12,400	8,280	4,830	510	5,340	34.00	2,938	1,072,224
	Zone 2		490	300	-	300	2.00	173	63,072
	Zone 3		1,260	865	-	865	4.50	389	141,912
	Zone 4		800	650	-	650	1.70	147	53,611
	Zone 5		1,300	660	10	670	7.30	631	230,213
	Zone 6		72	32	-	32	0.50	43	15,768
	Zone 7		180	80	-	80	1.10	95	34,690
Total		**12,400**	**12,382**	**7,417**	**520**	**7,937**	**51**	**4,415**	**1,611,490**

Note .... Industry in the area of the town Velika Gorica is not a significant factor in the overall water balance. Handicraft and catering are included in real consumption through population.

* .... Refers to quantities entering and staying within the zone, without transit quantities transported into another zone.

Also, along with mounting of new pressure reduction valves, some sections of the pipeline network should be closed at existing section stop valves, for hydraulic isolation of the sections and avoiding generation of increased pressures downstream. The analysis of pressure regulation in the system was carried out in an effort to use as few regulation valves and closed sections, i.e. section stop valves as possible.

Various possibilities of pressure reduction (locations and number of pressure reduction valves) have been tested on the mathematical model. In relation to the configuration of the entire system, the results impose the optimum solution with installation of 13 pressure reduction valves. The valves are automatically controlled with adjustment of output pressure values to 3.0-4.0 bar during the day, and 2.5-3.0 bar during the night. Fig. 4. within results of mathematical modeling shows the influence of mounting of only one pressure reduction valve in Zone 5. Only one valve is planned at water entrance into Zone 5. Considerable reduction of pressure is noticed, both in the period with minimum night consumption and in the daily regime with maximum hourly water consumption.

Along with advanced modeling of water losses, it is possible to follow variation of losses in real time in relation to pressure variations within the system. This opens the possibility of insight into real values of reduction of water losses after application of the suggested improvement measures. This is also important from the standpoint of economic feasibility of the suggested optimization measures.

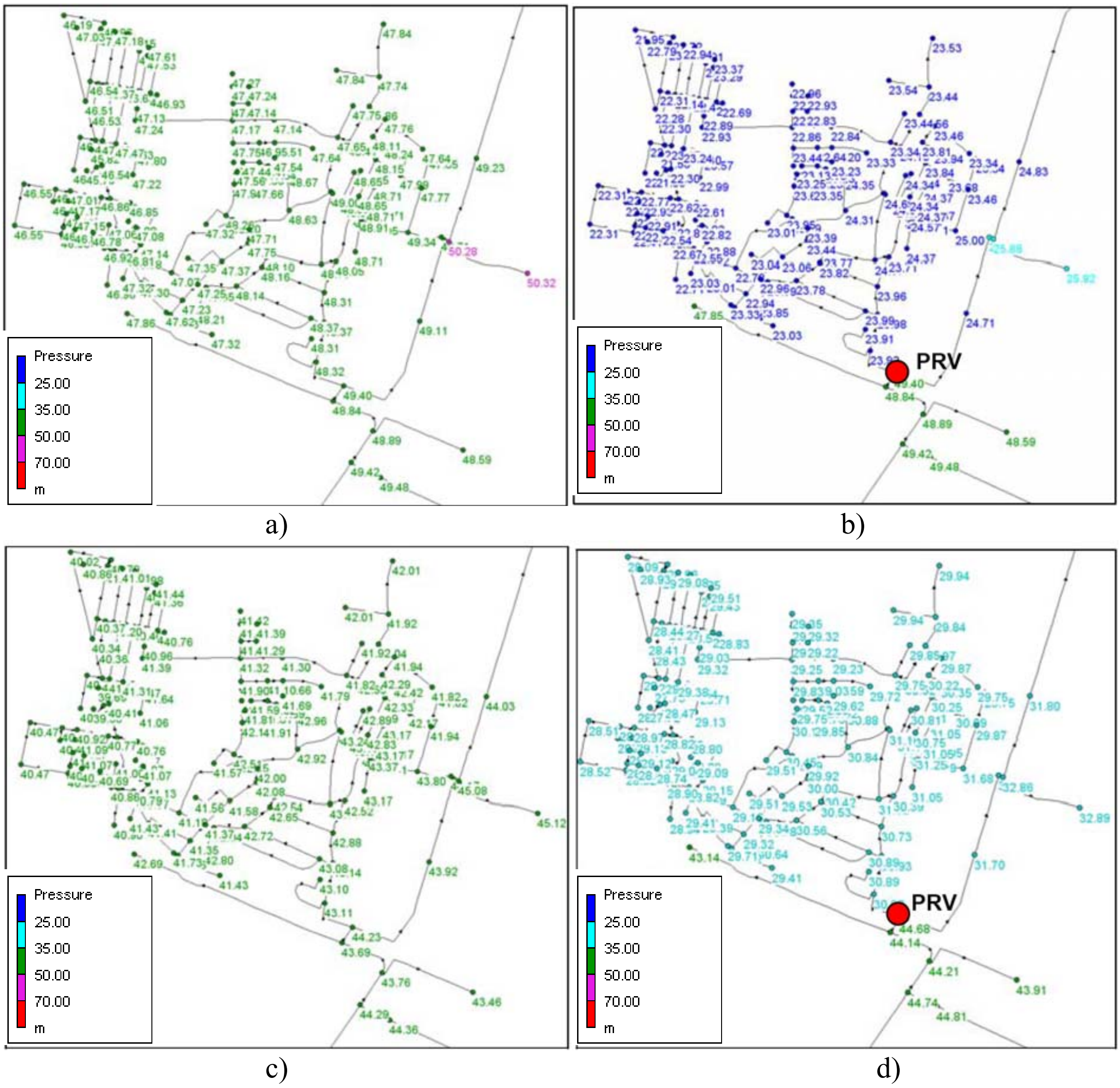

Fig. 4 Pressure distribution within Zone 5: (a) period with minimum consumption (present situation); (b) period with minimum consumption after installation of pressure reduction valve (PRV); (c) period with maximum consumption (present situation); (d) period with maximum consumption after installation of pressure reduction valve (PRV)

Tab. 2 Reduction of losses after implementation of priority activities for pressure regulation

Water Supply System	DMA	Real water losses	
		Present state (m^3/a)	After priority activities (pressure reduction) (m^3/a)
Velika Gorica	Zone 1	1,072,224	684,331
	Zone 2	63,072	40,997
	Zone 3	141,912	91,454
	Zone 4	53,611	31,536
	Zone 5	230,213	122,990
	Zone 6	18,922	18,922
	Zone 7	34,690	34,690
Total		**1,614,643**	**1,024,920**

4 CONCLUSION

The paper presents the methodology and importance of using of mathematical models in analyses of water losses in water supply systems. The basis of the suggested methodology is conducting of system analysis of the present situation founded on working out of detailed mathematical model and its calibration. A calibrated mathematical model is the basis of all further analyses and defining of sound system improvement measures, with particular stress on defining and recommending of optimum measures for reduction of water losses.
The possibility of constant expanding of the mathematical model of the water supply system, in accordance with implementation of adequate measures to improve the hydraulic operation conditions and reduction of water loses, contributes to its permanent value.

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IMPROVEMENT OF HIGH DISCHARGE WATER PUMPING STATION COMPLIANCE WITH STAKEHOLDERS VARIABLE WATER DEMAND

A. Constantin[1] , G. Iordache[2] , C.St. Niţescu[3] and M Stănescu[4]

Abstract

The decrease of the irrigated surface, during the last two decades, led to less pumping stations water demand. The paper presents a hydraulic analysis of the most suitable variants for the rehabilitation of an irrigation water supply pumping station of 46,1 m3/s discharge at 55m total head. One single pump of the eight the station is equipped with has a discharge of 5,7 m^3/s which is much more than the stakeholders demand in the beginning and in the end of the watering season. Therefore, the technical variant to be adopted for rehabilitation should assure a good compliance of the discharge with the new water demand. Pumps replacement, in a few variants, allows the delivery of small discharges, resulting in a decrease of power consumption and a consistent improvement of the energetic efficiency of the pumping system.

Keywords

Duty point, energetic efficiency, hydraulic pressure system, irrigation water supply, pumping station.

1 INTRODUCTION

Dobroudja County, in Romania, has an old irrigation system conceived to deliver water to a large area of agricultural land. Nowadays, in spite of low precipitation amount, the water demand decreased due to the high price of pumped water. The pumping stations with constant

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speed pumps and large discharge face a new problem: they cannot adapt the discharge to low water demand in the beginning of the watering season. Engineers look for the best variant for modernization, trying to achieve a good compliance between the pumping systems and the water demand, and a higher efficiency of the old pumping stations.

Sinoe pumping station (PS) is an irrigation water supply PS, which delivers 46,1 m^3/s of water at 55m total head. Sinoe PS is equipped with 8 vertical mounted pumps, driven by 5Mw electric motors. The discharge of one pump is $Qp=5.76\ m^3/s$ at a total head of H=55m. The 8 pumps are hydraulically connected in 2 groups of 4 pumps mounted in parallel. Each group has its own discharge duct of 800m in length.

Sinoe PS takes water from the Golovita Lake and discharges it into the irrigation system with the same name, Sinoe, Fig.1.

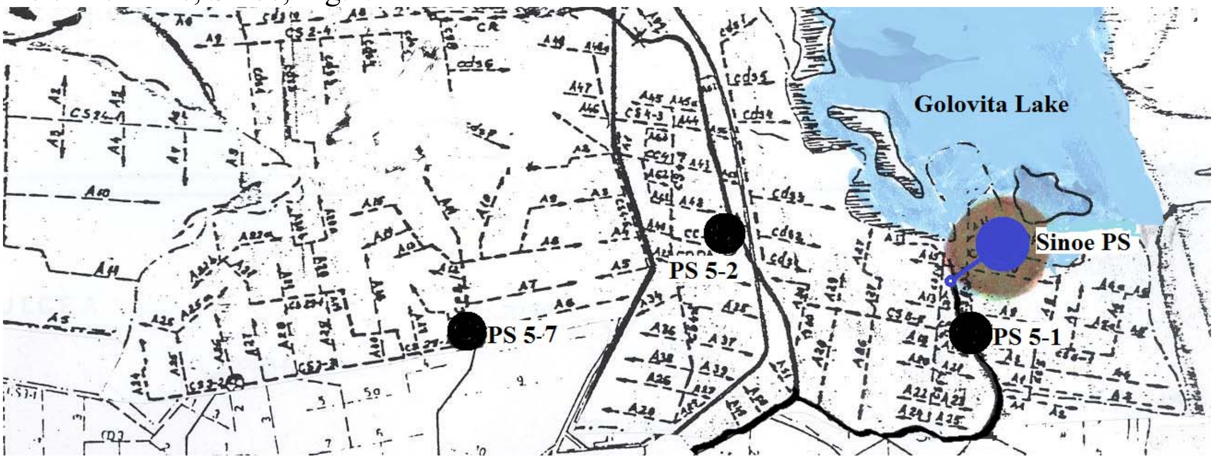

Fig. 1 Part of Sinoe irrigation system. Sinoe PS emplacement

This PS was built to supply water to an agricultural land of over 60 000 ha. From the very beginning it was a huge electric energy consumer. Now, after a long operation time, more than 30 years, the energetic efficiency decreased very much.

Although the region has a dry climate with a temperature average of 10^o - 11^o C and annual average precipitation less than 400mm, water demand for irrigation decreased very much during the last years. The main causes are the high price of pumped water and the decrease of agricultural surface. The price of water supplied by Sinoe system varies from $100EU/1000m^3$ up to $220\ EU/1000m^3$, therefore many of the stakeholders can't afford a proper watering of the land they own.

Moreover, the discharge of one single pump in this PS is bigger than the water need in the beginning of the watering season.

Our study aims to provide the best solution, from both technical and economic view point, for the modernization of Sinoe PS. The technical solution should offer the compliance of the PS discharge with the variable demand of the stakeholders, especially with their low water demand. The new requested discharge ranges between: (0.3-1.5) Qp.

2 ASSESMENT OF HYDRAULIC AND ENERGETIC PARAMETERS OF THE EXISTING SINOE PUMPING STATION

The hydraulic and energetic parameters of the Sinoe PS: discharge, total head, consumed power and efficiency were experimentally determined for one of the two groups of pumps. The group is composed of 4 pumps on a common discharge duct, one of them, P3 being out of

order. Therefore, the results presented below will refer to the pumps: P1, P2, P4, as they are represented in Fig.2.
The total pumping head was determined by measuring the pressure in the duct upstream and downstream the pump, using static pressure gadgets. Assuming the water velocity has the same value at pump inlet and outlet, the total head is given by the relationship [1], [2],[3]:

$$H = \frac{p_e - p_i}{\gamma} + \Delta z \tag{1}$$

where p_i-pressure at pump inlet, $\left[N/m^2\right]$;
p_e-pressure at pump outlet, $\left[N/m^2\right]$;
Δz -elevation difference between pressure gadgets, [m];
γ-specific weight of water, $\left[N/m^3\right]$.

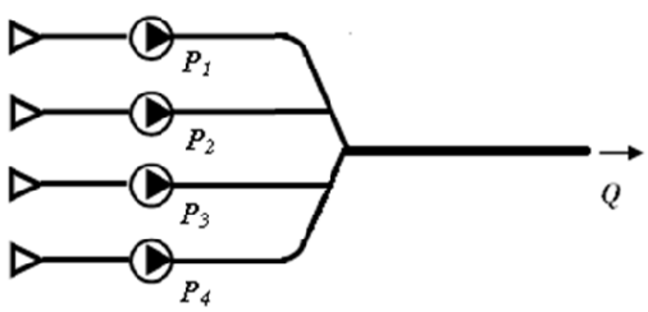

Fig. 2 Layout of one group of pumps

The relative error for head measurement was .
The discharge was determined by investigating the velocity field in one cross section of the discharge duct, far from any disturbance source.

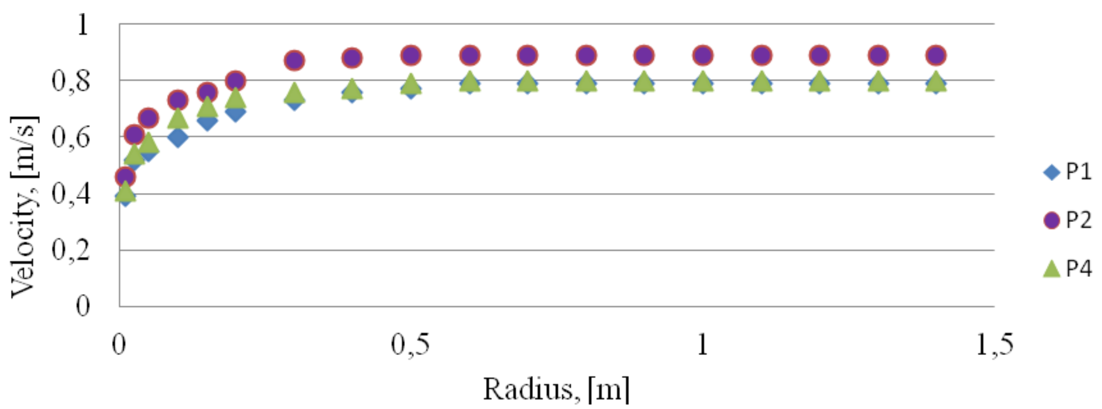

Fig.3 Velocity distribution along the radius of the discharge duct, for individual operation of the pumps

A Pitot-Prandtl tube was used to measure the difference between total and static pressure in different points along the diameter of the cross section of the discharge duct [7]. In Fig.3 there is represented the velocity field experimentally determined for the individual operation of the existing pumps. The discharge was calculated by numerical integration. The integral relationship of the discharge, for a circular duct of radius ***r***:

$$Q = 2\pi \int_0^R v_{(r)} \cdot r \cdot dr = 2\pi \int_0^R v_{(r)}\, d\left(\frac{r^2}{2}\right) \tag{2}$$

may be approximated by the following sum [7]:

$$Q \cong 2\pi \sum_{i=1}^{n} \frac{v_i + v_{i+1}}{2} \cdot \frac{r_{i+1}^2 - r_i^2}{2} \tag{3}$$

The relative error for discharge determination was .

The electrical power, P, absorbed by a pump was directly measured by means of a wattmeter. The overall efficiency of a pump is given by the relationship [3], [4]:

$$\eta = \frac{\gamma Q H}{P} \tag{4}$$

Considering a constant electrical motor efficiency, , the pump efficiency, , may be determined as [1], [3]:

$$\eta_p = \frac{\eta}{\eta_m} \tag{5}$$

The pump efficiency was determined with an error of 3.2%.

3 PROPOSED PS EQUIPPED WITH CONSTANT SPEED PUMPS

In order to satisfy low discharge values requested by the stakeholders, and to give more possibilities to adjust the discharge according to the water demand, we proposed to replace the old pumps with new ones, as follows:

- ✓ One high discharge pump (HDP), of $17800\ m^3/_h$ at 49m head;
- ✓ Three low discharge pumps (SDP), of $4700\ m^3/_h$ at 49m head.

The decrease of the total head was obtained by replacing the old metal made discharge duct with a new one made of reinforced polyester, which has a low friction coefficient [3]. The new discharge duct is 1.8m in diameter. A lower pumping head means energy savings and decreases the maximal pressure during the hydraulic shock [5].

This combination of pumps may cover a wide range of discharge values. The hydraulic analysis of the pumping installation showed the main duty points of the pumps working individually or in parallel.

In Fig.4 it is shown the duty point which offers the smallest discharge, $Q = 5940\,{}^{m^3}/_{h}$. It is the intersection of the pump head curve with the head curve requested by the system, Hr(Q) [2], [3]. This duty point meets the minimal discharge requested from the new PS.
According to the hydromodulus specific for the plants cultivated in the area of the Sinoe irrigation system, this discharge can be used to irrigate a 2600ha surface of agricultural land. The pump operates at a 85% overall efficiency, which leads to consistent energy saving. Furthermore, we may notice a more rational use of water resources, a decrease of water waste, as the delivered amount of water is closer to the requested one, and not higher.

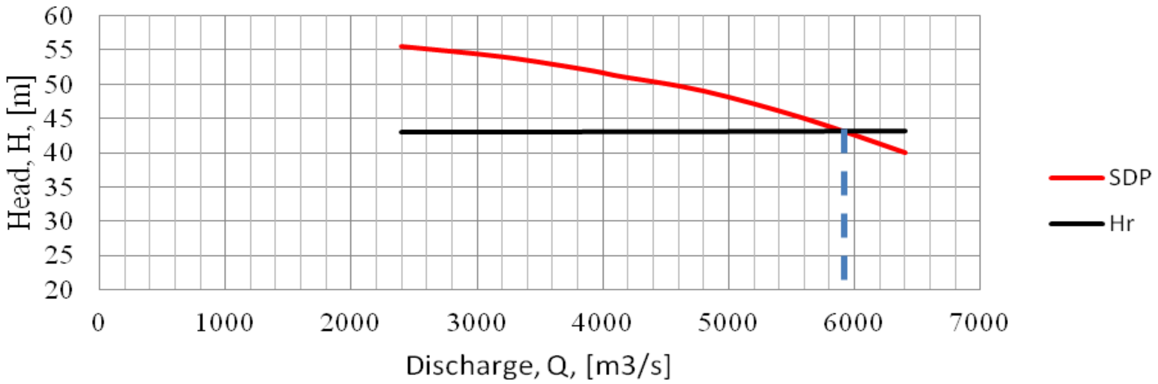

Fig. 4 Duty point for one SDP working individually:

$$Q = 5940\,{}^{m^3}/_{h}\ ;\ H = 43.4m$$

The duty point obtained in Fig.5 corresponds to the operation in parallel of two different pumps; they can operate in parallel only in the discharge range corresponding to the head field 52m - 40m [3].

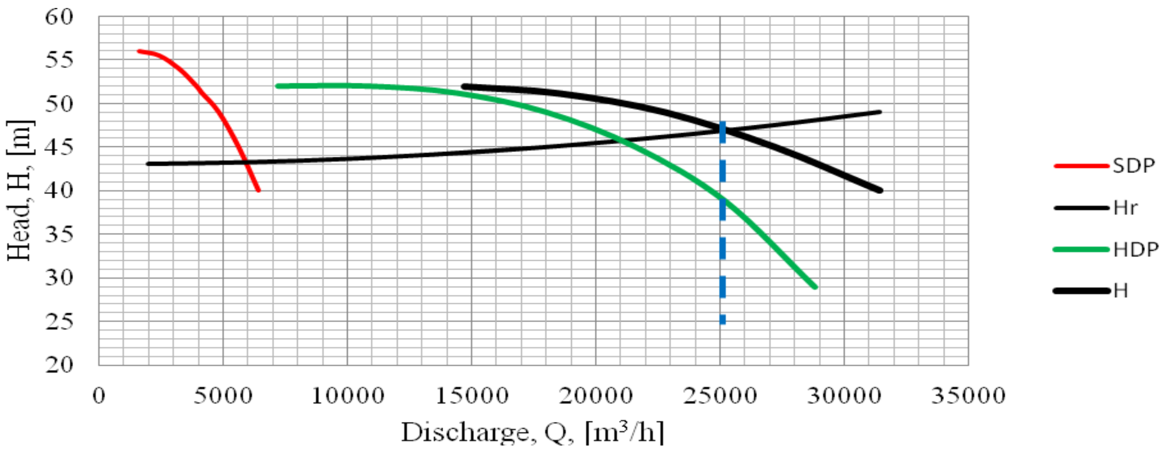

Fig. 5 Duty point for HDP and one SDP working in parallel:

$$Q = 25400\,{}^{m^3}/_{h}\ ;\ H = 47m$$

The maximal discharge value is given in the case all the 4 pumps work in parallel, as it might be seen in Fig.6.

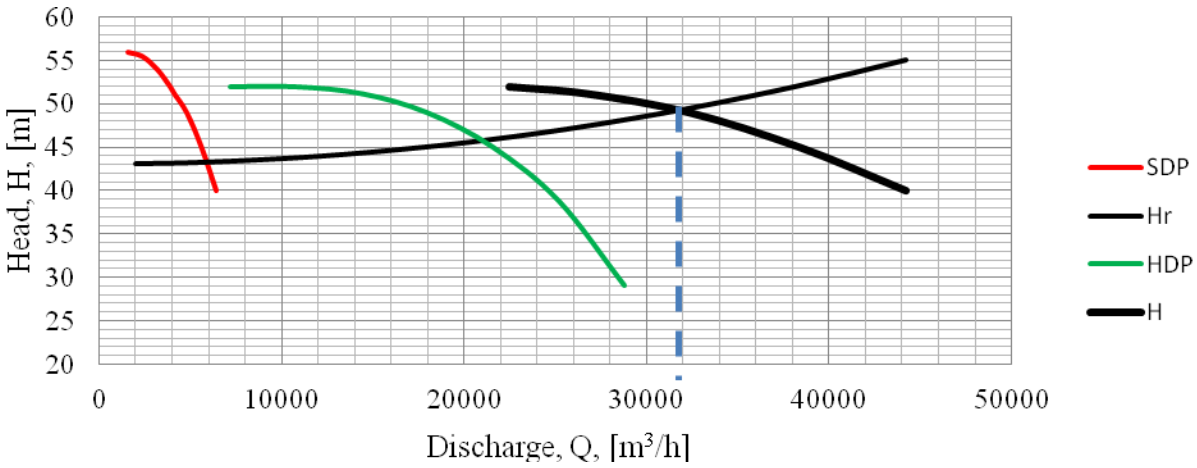

Fig. 6 Duty point for HDP and three SDP working in parallel:

$$Q = 31900\ m^3/_h\ ;\ H = 49m$$

The duty points were analysed for each combination of operating pumps, determining not only the discharge and total head, but also the energetic efficiency and consumed electrical power.

4 RESULTS

4.1 Operation parameters of the existing PS

The experimental investigations allowed us to assess the technical status and performance of the existing pumps in Sinoe PS. The values obtained by direct or indirect measurement are shown in Tab.1 [7]. None of the three investigated pumps offers the initial nominal value of the discharge, Qp=5.7 m^3/s. Pump P2 has the highest efficiency, 63.6%, among the three pumps. But it is a low value. We may also notice that pump P1 has the highest wear degree.

Tab. 1 Duty points and operation efficiency for the existing pumping station

Pumps	Head	Discharge	Power	Efficiency
[-]	**[m]**	**[m3/h]**	**[kw]**	**[%]**
P1	51,9	16.132	3.920	58,2
P2	50,87	18.468	4.025	63,6
P4	52,9	16.664	3.960	60,6
P1+P2	52,9	32.814	7.586	62,4
P2+P4	53,7	33.196	7.850	61,9
P1+P4	52,9	30.744	7.465	59,5
P1+P2+P4	55,29	47.214	11.370	62,6

These results highlight the best operation possibilities of the old pumps, from energy efficiency viewpoint.

4.2 Operation parameters of the proposed PS

The new proposed PS was conceived to deliver a wider range of smaller discharge values than the old PS. Aiming a low investment cost, the new proposed pumps have constant speed. Theoretical analysis of the proposed PS operation pointed out its main hydraulic and energetic parameters, which are gathered in Tab.2.

Tab. 2 Duty points and operation efficiency for the proposed pumping station equipped with constant speed pumps

Pumps	Head	Discharge	Efficiency	Power
[-]	**[m]**	**[m3/h]**	**[%]**	**[kw]**
1HDP	46	20.100	86	2.930
1SDP	43,4	5.960	85	829
1HDP+1SDP	47	25.400	86,6	3.756
1HDP+2SDP	48	29.000	85,2	4.452
1HDP+3SDP	49	31.900	85,2	4.999
2SDP	44	11.000	87	1.516
3SDP	44,8	17.000	87	2.385

It may be noticed a good compliance of the delivered discharge with the small requested ones. The energetic efficiency is very much improved, the highest values corresponding to the most likely requested discharge values: the small ones. The higher discharge values will meet the peaks of water demand in the watering season, when low precipitations are registered.

5 DISCUSSION

On the basis of the results presented above, a few variants for adapting the delivered discharge to the smaller actual water demand were proposed. We'll present them bellow. The first two were proposed by the stakeholders.

The first and the cheapest technical solution was to adapt a frequency convertor to one of the existing pumps, which will turn that pump from a constant to a variable speed one. But in an open circuit pumping system, the efficiency of the variable speed pump changes with the rotation speed [6]. The old pumps are already worn and a decrease of the rotation speed will lower the efficiency further more. This solution has a major drawback: low energetic efficiency which results in high operation cost.

The second variant considered a couple of new SDP with constant speed to replace the broken pump P3 and maintain the old P1, P2 and P4 pumps. This variant offers a good possibility to deliver low discharges, with a good energetic efficiency. But high discharges would have been delivered by one or two old pumps operating in parallel and their operation proves a low efficiency. This results in high operation costs for high values of the discharge. The variant is not recommendable, although it involves small investment costs.

The recommended variant is the third one, depicted in the above section 3, which proposes the replacement of all 4 pumps with new ones: one HDP and three identical SDP, with constant rotation speed. The investment cost is higher than in the previous variants, but operation costs are much lower and allow a rapid payback.

The fourth variant refers to the same configuration as that in the variant 3, but one of the pumps should have variable rotational speed. This measure will enlarge the range of discharge, with a better efficiency. This variant offers the best compliance of the discharge with the water demand, at high efficiency, but the investment costs are the highest.

6 CONCLUSION

Rational and efficient use of water resources requests the modernization of old irrigation water supply pumping stations, which are huge energy consumers. Along with the achievement of required hydraulic parameters, the engineers in charge with the modernization of pumping installations have two more important goals: the improvement of energetic efficiency and the reduction of electric energy consumption. Both lead to important energy saving and furthermore to lower cost of pumped water and cleaner environment.

We consider that the replacement of the existing pumps with new ones, of smaller discharge and of constant rotational speed, as depicted in variant 3, is the best variant from either technical or economic point of view. The replacement of the discharge duct with a new one made of reinforced polyester decreases the total head, due to the smaller friction coefficient.

The recommended variant of rehabilitation is more affordable than the fourth variant and comparable as energetic efficiency.

A comparison between the recommended variant and the existing pumping station shows important energy savings for a volume of 1000 m^3 of pumped water:

- 103kWh/1000 m^3 for the minimal value of water discharge;
- 110 kWh/1000 m^3 for the same value of discharge: 31900 m^3/h.

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WATER LEVEL, PRECIPITATION AND AIR TEMPERATURE REGRESSION ANALYSIS-CASE STUDY LAKE DOJRAN

Violeta Geshovska[1], Cvetanka Popovska[2], Ognjen Bonacci[3]

Abstract

Lake Dojran is a transboundary lake in the south-eastern part of the Balkan Peninsula and is shared by Macedonia and Greece. The lake is small and shallow but very significant as a water resource and natural wealth. This paper deals with some answers related to the clarification of natural causes. Very often on scientific and institutional level may be heard that the present state of the lake is basically due to decreased precipitation and increased air temperature, or due to climate change effects. The analysis of the basic hydro meteorological parameters, such as precipitation and air temperature, is presented and their correlation with observed lake water level is established. Data that have been used are for the period 1951-2010 from the hydro meteorological station at Nov Dojran on Macedonian Lake side.

Keywords

Precipitation and air temperature regression, Lake Dojran, climate change effects

1. INTRODUCTION

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Dojran Lake is a tectonic lake situated at an average altitude of 148 m asl. The watershed and the lake itself are shared by two countries, Greece and Macedonia. The lake was formed in a karstified basin created by combination of Tertiary and tectonic activity. The sediments of the lake watershed are composed of mineral-rich ancient alluvial and limestone sediments. A minor part of the watershed on Macedonian side is composed of diluvia clay sediments. The northern and eastern belts of the watershed are rocky and covered with low forests and weeds. The watershed within the Macedonian territory is characterized with relatively large annual production of erosive material of about 29.000 m^3/annually, out of which only 323 m^3/annually is transported towards the lake.

The shape of the lake is rather regular with maximum length of 8.9 km and maximum width of 7.1 km. The volume of the lake at normal water level is 262 million m^3 which corresponds to the average depth of 6.5 m (maximum depth is 10.4 m). The lake doesn't have surface outflow. The only natural outflow is the lake water surface evaporation. Total watershed area of the lake is 271.8 km^2 out of which 32% belongs to Macedonia. The water surface area of the lake at normal elevation is 42.2 km^2 out of which 63.6% belongs to Macedonia.

The last two decades Dojran Lake water level decreased seriously. This water declination together with the simultaneous water quality deterioration resulted with biodiversity diminishing and plankton reduction. Also a number of birds decreased dramatically. The attack on the ecosystem had a harmful impact on the economy in the region. Because of this, various efforts were initiated, mainly on the Macedonian side, to improve the hydrological and ecological state of this valuable water body in the region. How much the present state of the lake is impacted by hydrology and climate, and how much by human activities in the watershed, is the question that is not answered yet. Therefore, the scientist and researchers are focused on causes identification and clarification. The answers should help in water management plan development and implementation in both countries that share the lake and its watershed. The base of such management plan is a water balance model that can not be done with limited and incomplete data. Although, water balance equation is rather simple, the most difficult is to define its components.

2. PRECIPITATION

The precipitation regime and other basic meteorological parameters are analyzed for the period 1951-2010 with data from hydro meteorological station at Nov Dojran on Macedonian side. The location and elevation at 180 m asl of this station are not representative for the entire watershed of the lake, but are suitable for obtaining the precipitation quantity over the lake. It is very probable there is increasing precipitation gradient with the altitude that can not be neglected as well as the snowfalls that are feeding the lake with water. The average water level within the period 1952-1974 is 146.7 m asl that corresponds to the lake volume of about 250 million m^3. The minimum water level observed in 2002 is 141.05 m asl corresponds to the volume of 50 million m^3. Comparative analysis between the annual precipitation sums and average annual water levels are presented in Figure 1. The precipitation trend line has small increasing gradient that is not followed by the water level oscillation. So, it is obvious there is no uniform relation between the precipitation and water level. The computed correlation coefficient R=0.066 shows that there is no strong relation between the precipitation and water level oscillations.

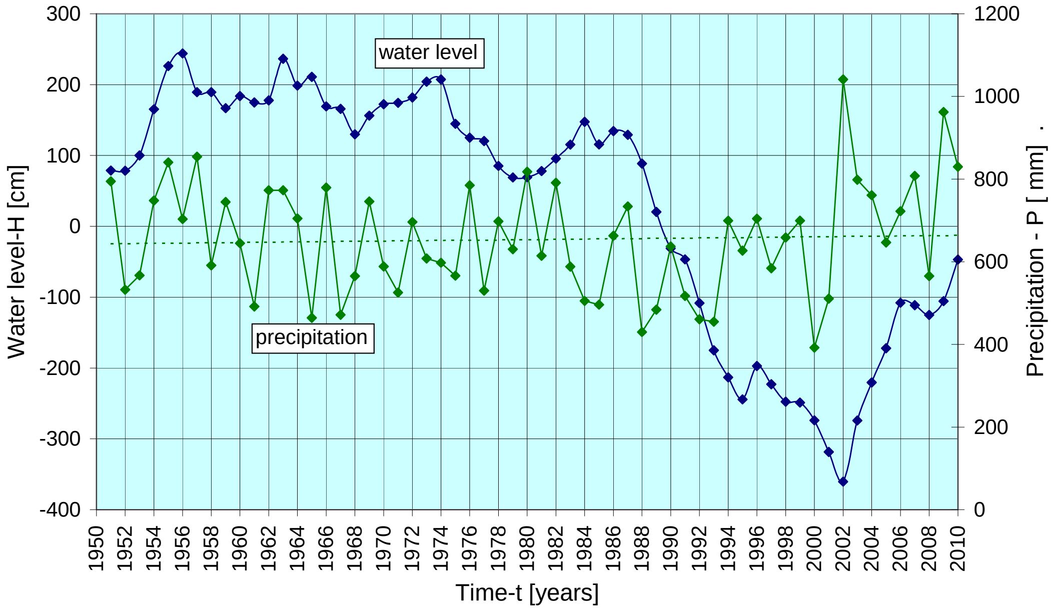

Figure 1 Comparative analysis between the average annual water levels and annual precipitation sums

Looking at the observed water levels it is obvious that the following periods can be analyzed separately by the following sub periods: 1951-1988, period before water level declination, 1988-2002, period whit rapid water level declination, and 2002-2010, period with water level increase. The obtained correlation coefficients for these periods are also very low: $R_{1951-1988}=0.117$, $R_{1988-2002}=0.50$ and $R_{2002-2010}=0.148$. The results are shown in Figure 2. It is obvious that only for the second sub period is obtained relatively good correlation, which can be explained by the observed dry period.

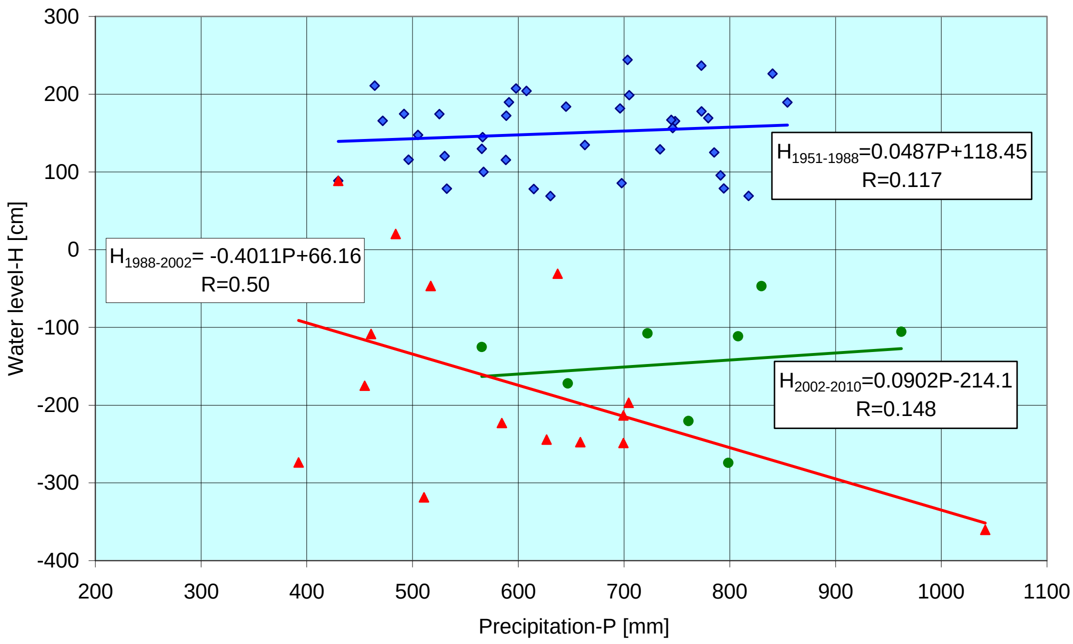

Figure 2 Relation between the average water levels and annual precipitation sums for the analyzed sub periods

3. TEMPERATURE

The comparative analyses of the average annual air temperatures and average water levels show very important mutual relation. While the temperatures have increasing trend for the entire period, water levels show changeable trends, Figure 3. The computed correlation coefficient R=0.5 shows good relation. When air temperature increases the water level decreases due the water loss by evapotranspiration, mainly by evaporation from the lake water surface. It is important to mentioned that the relation between the temperature and water level for the period 1951-1988 is very week and statistically insignificant, but for the other two periods 1988-2002 and 2002-2010 the correlation coefficients show rather good relation, $R_{1988-2002}$=0.54 and $R_{2002-2010}$=0.66. The results are shown in Figure 4. The relation between the precipitation and air temperature was also analyzed, Figure 5. It was obtained rather week relation for the entire period R=0.012 and for the sub periods as well: $R_{1951-1988}$=0.268, $R_{1988-2002}$=0.19 and $R_{2002-2010}$=0.11.

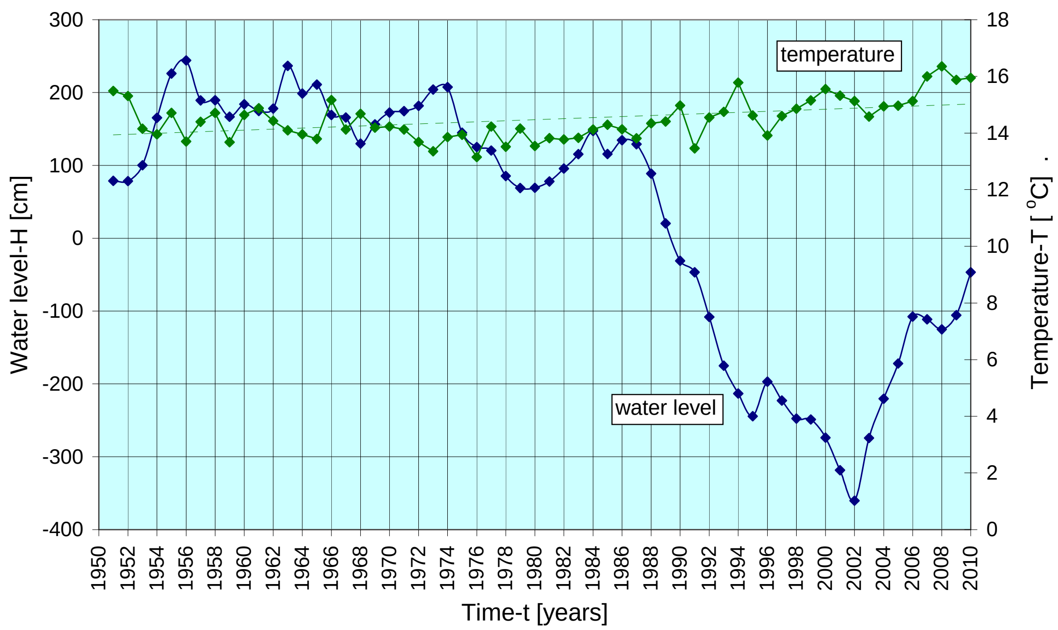

Figure 3 Comparative analyses between the average annual water levels and the average annual air temperature

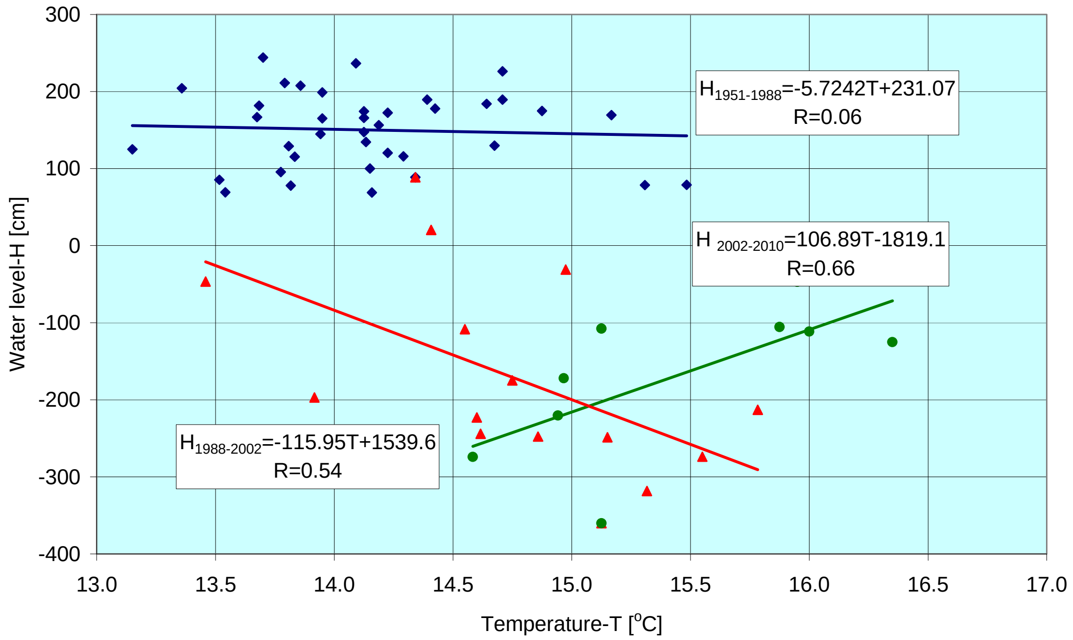

Figure 4 Relation between the average annual water levels and average annual air temperature for the analyzed sub periods

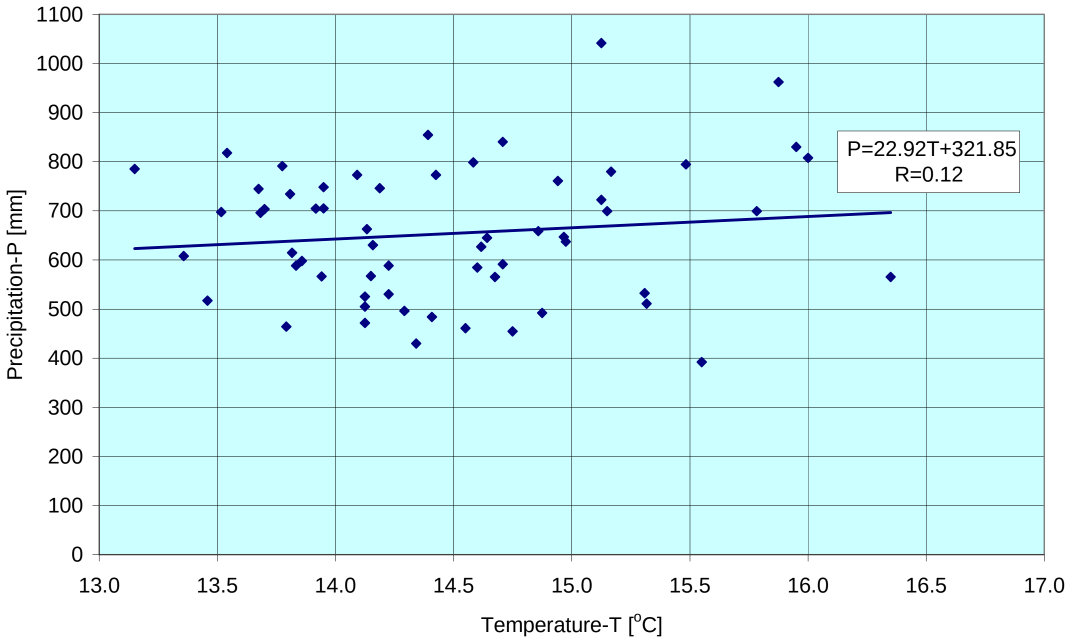

Figure 5 Relation between the annual precipitation sums and average annual air temperature for the period 1951-2010

4. CONCLUSION

The Dojran Lake till 2002 has lost its storage to the stage of ecological catastrophe. The basic economy in the region, tourism and fishery, has stopped almost completely. The last decade it is observed continuous water level increase which is not reaching the long-term average water level yet. The regression analysis between water level, precipitation and temperature has been performed to identify natural causes of the observed rapid water level decrease.

Three sub periods 1951-1988, 1988-2002 and 2002-2010 were analyzed. For the relation between the precipitation and water level it is concluded that there is no uniform relation. The computed correlation coefficient for the entire analyzed period shows that there is no direct relation between the precipitation and water level oscillations, $R_{1951\text{-}2010}$=0.066. Only for the second sub period is obtained relatively good correlation $R_{1988\text{-}2002}$=0.50, which can be explained by the observed dry period.

For the relation between the air temperature and water level oscillations, it is observed very important changeable trends. The computed correlation coefficient for the entire period is rather week and insignificant, but for the second and third sub periods the correlation coefficients show direct relation, $R_{1988\text{-}2002}$=0.54 and $R_{2002\text{-}2010}$=0.66.

The authors would like to stress out the necessity of precipitation and snow measurements on higher altitudes. The temperature regime is the warmest in the region that results with very high evaporation from water surface – almost twice the annual precipitation sum.

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TRENDS IN WATER USE

Gordon Gilja[1], Marin Kuspilić[2] and Živko Vuković[3]

Abstract

Over the twentieth century world's population rapidly grew followed by land urbanization: at the turn of the century proportion of urban population was only 13 %, in 2009 it surpassed rural population and projection shows that it's likely to rise up to 69 % by middle of 21st century. Water resources are integral part of everyday municipal, as well as economical and ecosystem needs - which are in essence mutually exclusive. As populations and economic activities grow, many countries are rapidly reaching conditions of water scarcity and misuse of fresh water that pose a serious and growing threat to sustainable development and protection of the environment. Aforementioned problems are many addressed by forcing development of ever more distant water sources. Globally, the water resources in various regions and countries are expected to face unprecedented pressures in the coming decades as a result of continuing population growth and uneven distributions of population and water. More stringent wastewater treatment and effluent discharge regulatory requirements demand adaption of new approaches in urban water supply management. New approaches now incorporate the principles of sustainability, environmental ethics and public participation in project development and revise current planning, construction and management practices. This paper presents trend in water demand and its variation in distribution across economic sectors. New approaches in water resources management are presented with critical overview of their characteristics.

Keywords

Sustainable development, water demand trend, water resources management

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1 INTRODUCTION

Freshwater resources are essential component of all ecosystems and a vital element needed to sustain almost all life on the planet. Trends in population growth, urbanization and water resources exploitation are relevant indicators of water resources management efficiency and sustainability. Pessimistic projections on the availability of freshwater resources in the upcoming decades and more stringent water quality and wastewater treatment regulatory requirements demand a new approach in water resource management. New approaches now incorporate the principles of sustainability, environmental ethics and public participation in project development and revise current planning, construction and management practices.

2 WATER DEMAND INCREASE

20th century was characterized by rapid increase in world population as well as land urbanization and this trend is projected to continue throughout 21st century. The world population has reached 7 billion in late 2011, and according to the 2010 "Revision of the official United Nations population estimates and projections" it will surpass 9 billion people by 2050 (**Fig. 1**). Virtually all of the expected growth in the world population will be concentrated in the urban areas of the less developed regions. Overall, the world population is expected to be 67 % urban in 2050 [1].

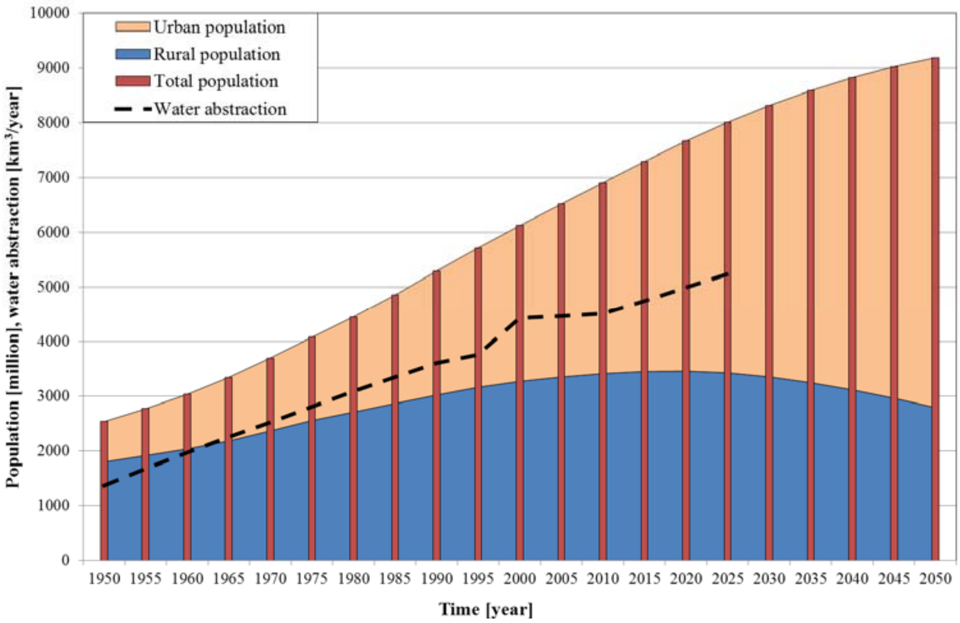

Fig. 1 Urban and rural world population [1] and the volume of total abstracted water [2].

Population growth and technological advances have resulted in a triple increase of water withdrawal since 1950; if this trend persists, by 2025 the quantity of withdrawn water will be four times the amount abstracted in 1950. Increase in mainly urban population will result in a higher water demand which will be concentrated in major cities and surrounding slums. In order to satisfy this increased water demand current water supply providers will have to expand system capacities and resort to costly and inefficient solutions to solve this predicted problem [3].

3 WATER SCARCITY AND THE MILLENNIUM DEVELOPMENT GOALS

Although the volume of available freshwater is sufficient to meet the current and projected human needs, it is not evenly distributed across the globe and much of it is wasted, polluted and unsustainably managed. Historically, cities have relied on their closest sources of water, which in most cases were freshwater streams, lakes or springs. Soon, these resources were exhausted or degraded, and the cities were forced to invest considerable funds in the construction of dams and pipelines to bring water from more remote sources. Continually expanding cities have outgrown most water-supply systems and new sources must be developed to satisfy the needs of their population.

In the year 2000, nearly 1.1 billion people still remained without access to improved sources of water [4]. In order to deal with this problem, during the Millennium Summit in New York in 2000, 189 UN member states have adopted the Millennium Declaration. One of the set Millennium Development Goals (MDG) was to halve the proportion of people without sustainable access to safe drinking water by 2015 [5]. It has been estimated that meeting this goal would require between US$ 14 billion and US$ 30 billion a year on top of the roughly US$ 30 billion already being spent [6].

Initiative to improve access to clean drinking water since the Millennium Declaration has been strong, resulting in a significant improvement in drinking water coverage. In 2010, 89 % of the world’s population was using improved drinking water sources, up from 76 % in 1990. This means that the MDG target of halving the proportion of the population without sustainable access to safe drinking water has been met, five years ahead of the 2015 target. If current trends continue, 92 % of the global population will be covered by 2015. Still, coverage with improved drinking water sources for rural population is lagging. In 2010, 96 % of the urban population used an improved drinking water source, compared with 81 % of the rural population. The gap between urban and rural areas remains wide, with the number of people in rural areas without an improved water source (653 million) five times greater than one in urban areas (130 million) [7].

4 EXPLOITATION OF WATER RESOURCES

The proportion of water resources used by a country is a complex indicator reflecting development, national water policies, and physical and economic water scarcity. At low levels of development, it is generally advantageous to increase total water withdrawal for satisfying increase in water demand. But beyond a certain “inflection point”, ecosystems will be strained and competing water uses will not allow all users to receive their fair share [7]. The abovementioned inflection point cannot be easily determined because of large annual differences in precipitation. Next figure (**Fig. 2**) shows that globally water resources exploitation is exponentially increasing. Once extracted water may be used, recycled (or returned to rivers or aquifers) and reused several times over. Consumption is a final use of water, after which it can no longer be reused. Consumed water is either “lost” through evaporation, transpiration, food and industrial goods production, or it has been severely contaminated. The striped band represents the difference between the amount of water extracted and that actually consumed. When the extracted water is not returned to its aquifers, they gradually have less water available for following rounds of exploitation. The amount of water not returned to its initial aquifer is still available in a hydrological cycle, but is located in different, inaccessible forms or in a different aquifer. The most significant volume of water being globally consumed is generated by agriculture.

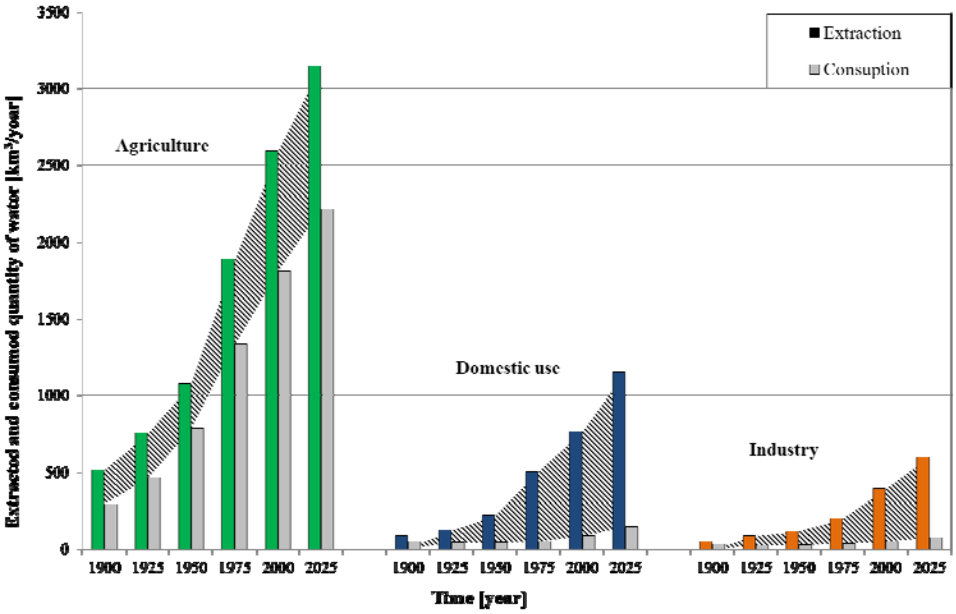

Fig. 2 Global water extraction and consumption by sector [8].

At global level, the withdrawal ratios are 70 % agricultural, 11 % municipal and 19 % industrial. These numbers, however, are biased strongly by the few countries which have very high water withdrawals. Averaging the ratios of each individual country, we find that "for any given country" these ratios are 59 %, 23 % and 18 % respectively. The ratios also vary much between regions, going from 91 %, 7 % and 2 % for agricultural, municipal and industrial water withdrawal in South Asia to 8 %, 16 % and 77 % respectively in Western Europe [9]. These relations are presented in figure (**Fig. 3**) along with the Croatian ratios which are significantly different from the worlds or European average [10].

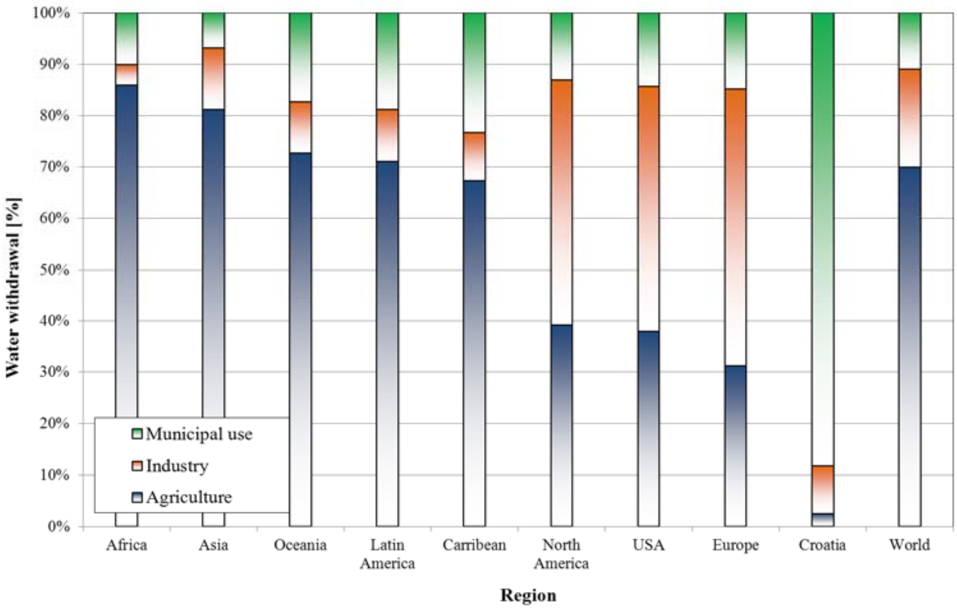

Fig. 3 Ratio of water withdrawal by sector [8].

Quantities of abstracted water differ greatly from region to region: East Asia, Latin America, Africa and a few smaller regions withdraw 3 times less water than an average OECD country and 5 times less then USA [11]. There are considerable differences in the per capita amounts of freshwater abstracted within each of the European countries as well: from a 24 $m^{3}inhabitant^{-1}year^{-1}$ (Cyprus) to 269 $m^{3}inhabitant^{-1}year^{-1}$ (Iceland). Croatia withdraws 129 $m^{3}inhabitant^{-1}year^{-1}$, which is higher than the European Union average of 93 $m^{3}inhabitant^{-1}year^{-1}$ [12]. USA abstracts an amount of 1602 $m^{3}inhabitant^{-1}year^{-1}$ [13]. When examining water abstracted for municipal use, there are even higher discrepancies between countries, from 71 $L^{1}inhabitant^{-1}day^{-1}$ (Romania) up to 1861 $L^{1}inhabitant^{-1}day^{-1}$ (Lithuania). In the year 2011, Croatian municipal use of water averaged at 183 $L^{1}inhabitant^{-1}day^{-1}$ [10] while an average USA citizen used 370 $L^{1}inhabitant^{-1}day^{-1}$ of water [13].

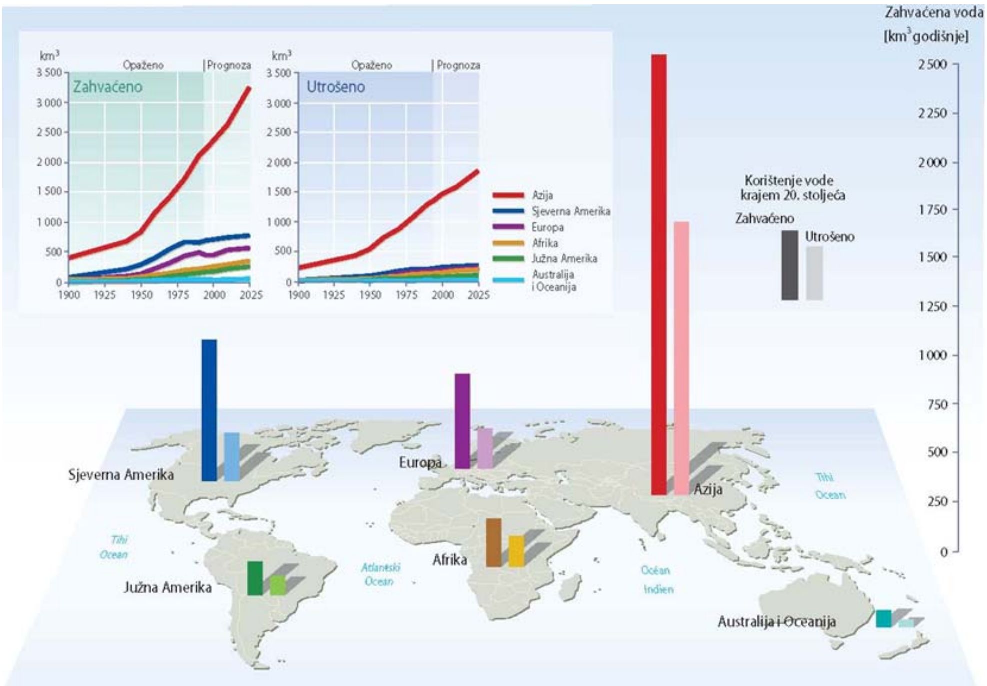

Fig. 4 Total withdrawn and consumed water by continent [2].

OECD member countries have reduced their average water use per capita by 11 %, and a reduction in water use has been achieved in 50 % of member countries. However, despite this trend, population growth caused an increase in the overall volume of water withdrawn in most of the countries. Only 9 OECD countries (all located in Europe) have managed to decrease their overall water abstraction in a period from 1980 to 1997 [11].

5 RENEWABLE WATER RESOURCES

Internal renewable water resources (IRWR) are that part of the water resources (surface water and groundwater) generated from endogenous precipitation. External water resources are the part of a country's renewable water resources that enter from upstream countries through rivers (external surface water) or aquifers (external groundwater resources). The total external resources are the inflow from neighbouring countries (trans boundary flow) and a part of the resources of shared lakes or border rivers. Total actual renewable water resources (TARWR) present the sum of internal renewable water resources and incoming flow originating outside the country and is valued as a maximal theoretical quantity of available water [14]. Globally, total water withdrawals still represent only a small share – about 9 % of internal renewable water resources (IRWR), but this average masks large geographical discrepancies (**Fig. 5**). The rate of withdrawal varies greatly by country or region. Europe withdraws only 6 % of its internal resources and just 29 % of this goes to agriculture [9]. Croatia withdraws only 1.51 of its IRWR [14]. The intensive agricultural economies of Eastern Asia withdraw 20 % of their internal renewable resources, of which more than 80 % goes to irrigation. In many of the low rainfall regions of the Middle East, Northern Africa and Central Asia, most of the exploitable water is already withdrawn, with 80 % - 90 % of that going to agriculture, and thus rivers and aquifers are depleted beyond sustainable levels [9].

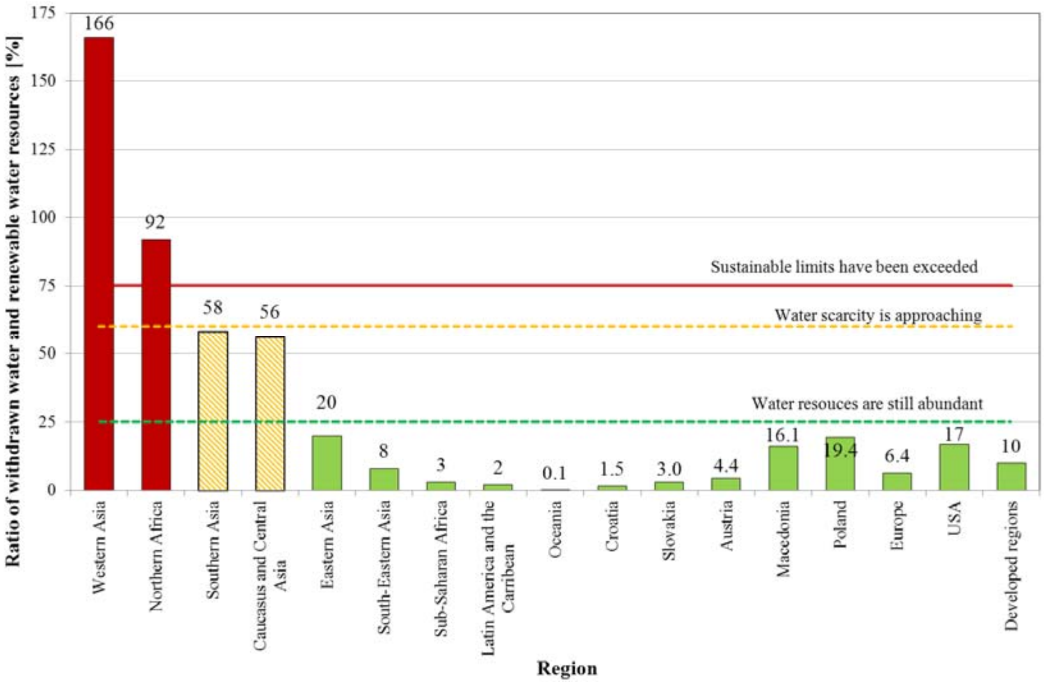

Fig. 5 Ratio of withdrawn water and internal renewable water resources [11, 14].

In terms of physical water scarcity it is estimated that on average withdrawal rate above 25 % of renewable water resources represents substantial pressure on water resources – and more than 60 % is 'critical'; at 75 % sustainable limit has been exceeded. Two regions which have exceeded sustainable limits are Western Asia and Northern Africa, while Southern Asia and Caucasus and Central Asia are approaching water scarcity levels [14].

6 SUSTAINABLE MANAGEMENT OF WATER RESOURCES

Water resources face competing demands from uses to support human health, economic development, and environmental services. In this sense, water is the perfect example of a sustainable development challenge – encompassing environmental, economic, and social dimensions. Reconciling these three aspects through appropriate water management is a significant policy challenge for governments [11].

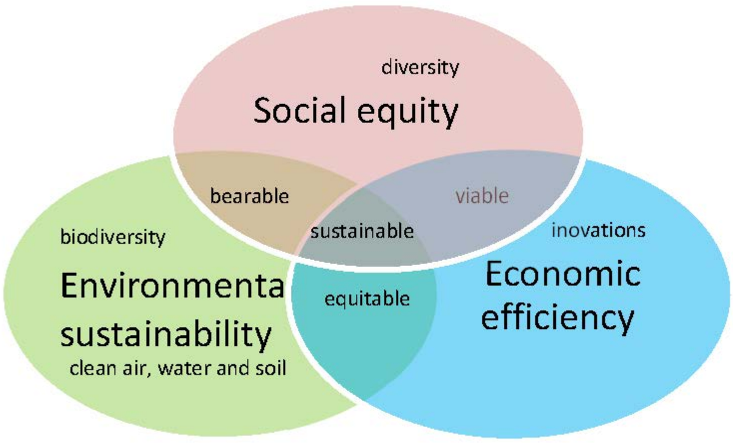

Fig. 6 Three dimensions of sustainability [15].

Poor water management is considered to be a limiting factor of sustainable development in the upcoming decades. Social, economic and ecological impacts generated by current water management techniques and an increasing water scarcity call for a new approach which involves sustainability principles, environmental ethics, public participation and re-evaluates the current methods of designing, building and management of water supply systems [16].
It is important to distinguish at the outset two main dimensions of water management:

1) water as a natural resource that is an integral part of the natural ecosystem; and
2) water as the key element of water services, which are generally infrastructure-intensive.

The first dimension involves the abstraction of water and its allocation among competing uses (e.g. industry, agriculture, municipal water supply, and ecological, aesthetic and recreational purposes). It also involves protection of surface-water bodies and groundwater reservoirs from degradation. The second dimension involves investment, operation and management of the infrastructure systems, and delivery of water services to final customers (i.e. treatment and distribution of piped water, wastewater collection and treatment, and irrigation networks) [11]. About 40 % of the world's population lives in trans boundary river basins, and more than 90 % live in countries with basins that cross international borders [17]. In order to successfully sustainably manage water resources an "ecosystem" principle should be implemented; water resources should be managed on a basin level so an equitable and reliable division of water resources is achieved for current and future generations. Sustainability can be achieved only by implementing technologically and economically viable solutions, especially in existing water supply systems which need to be expanded or improved to meet the increasing water demand. Clear understanding of availability and allocation of water in a hydrologic cycle as well as a clear concept of human impact on environment are prerequisites for achieving sustainable water management [16].
Water scarcity and overexploitation of freshwater resources present constantly growing threat to sustainable development. If management of water and land resources doesn't become more effective, food production and technological growth as well as ecosystem upon which they rely are endangered. Despite this negative trend, some progress is being made. For example, OECD countries have significantly reduced industrial and urban discharges to waterways, with the total share of the population connected to public wastewater treatment plants in OECD countries reaching an average of 65 %, and many of the rest using private sewage treatment. OECD countries have also cleaned up a number of the worst polluted freshwater

bodies. They have increased their water use efficiency, with several realising overall reductions in water use over the last two decades. Many have started to apply more integrated approaches to water management, following a “whole-basin” or “ecosystem” approach. Four essential conditions set for support of sustainable water management are:

1) Making markets work;
2) Improving the coherence of decision making;
3) Harnessing science and technology;
4) Working in partnership with developing countries.

The goal of sustainable water resources development and management is to meet water needs reliably and equitably for current and future generations by designing integrated and adaptable systems, optimizing water-use efficiency, and making continuous efforts toward preservation and restoration of natural ecosystems [16].

7 CONCLUSION

Current water exploitation practices and projected trends presented in this paper are created from datasets published by national statistical agencies. Methods used for data acquisition in certain world areas may not have followed “best practices” as many of the world’s countries do not have adequate personnel, technology or guidelines for collecting the desired information. Any conclusions that may be drawn from this paper should take in account the accuracy of gathered information. Nevertheless, water scarcity induced through increased global water demand is an on-going issue which affects numerous world regions and the demand for water resources will definitely increase in the upcoming decades. The limits of sustainability may be loosely defined but the apparent degradation of an ecosystem in some regions caused by overexploitation of water resources is an evident indicator of an unsustainable and wasteful water management practices. Global water resources are plentiful, but often unevenly distributed, polluted and mismanaged. Implementation of new technical solutions, improved water management techniques and more coherent decision making is a crucial step in achieving sustainability and satisfying world’s current and future needs for water.

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OVERVIEW OF THE SUSTAINABILITY ASSESSMENT METHODOLOGIES FOR WATER RESOURCES SYSTEMS

Ivan Halkijević [1] and Živko Vuković [2]

Abstract

Since the introduction of the term "sustainable development" in 1987 as one of the conclusions of a report called Our Common Future, sustainable development has been extensively challenged in terms of quantifying its meaning. Despite significant efforts by numerous researchers, institutions and organizations at local, national or international level, a strong degree of consensus upon definition of sustainability and sustainable development does not exist. However, it is generally considered that sustainability evaluation or assessment must take into account the social, economic and environmental objectives. Various authors have proposed many different methodologies for quantifying a degree of sustainability so that it can be compared to another system or process or used in a decision making process. This paper deals with the concept of sustainability in relation to water resources systems. Issues of water management sustainability as well as the most commonly used definitions and sustainability assessment approaches are presented. Herewith the most emphasized methodologies are related to probability assessment and the development of the sustainability index through performance indicators. Basic chronological development and the latest achievements are presented.

Keywords

Sustainability assessment, sustainable development, criteria, sustainability index

1 INTRODUCTION

In 1987 *World Commission on Environment and Development* of the *United Nations* released a report called *Our Common Future*, [28], in which most often quoted definition of sustainable development is given [5]. From that moment the question of measurement,

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quantification and evaluation of sustainability was raised. The purpose of this issue was to rate the degree of sustainability of individual process or system and to provide more precise definition of the newly introduced concept [23]. As the sustainability phrase soon became woven into all aspects of human life, it became clear that the measurement, evaluation and quantification of sustainability due to vague definition and multidimensional aspect (environment, economy, society) is a very complex problem, [8], [10], [14].

Despite significant efforts by numerous researchers, institutions and organizations at local, national or international level, a strong degree of consensus upon definition of sustainability and sustainable development does not exist. Various authors have proposed many different methodologies for quantifying a degree of sustainability so that it can be compared to another system or process or used in a decision making process [18]. Numerous proposed methods and methodological approaches are primarily characterized with a significant influence of the scientific areas which was the starting point of the research [19].

Since the middle nineties the concept of sustainable development clearly establishes the need for participation and interaction of economic, social and environmental dimension (component) [11].

In general, literature regarding water resource sustainability assessment is characterized by a conceptual inconsistency, terminology discrepancy, lack of uniformity and different methodological approaches between authors that suffer from a large degree of subjective judgment involved in determining which issues to include or not to include in the analysis.

2 SOME WATER RESOURCE SUSTAINABILITY DEFINITIONS

The most commonly used definition of sustainable development is "development that meets the needs of the present generation without compromising the ability of future generations to meet their own needs" [28]. This definition calls for balancing economic, social and environmental requirements, and to accept that all human individuals have equal rights, whether living today or in future. Such concept prevailed in defining development policies around the world.

There are many criticisms and disagreements on just what this definition suggests and states that should be done. How a person today can know what the future generations will need? There are also disagreements on just how the sustainability of any system, including water resource systems, can or should be achieved. Are we supposed to provide the welfare of future generations by preserving or enhancing the current state of natural resources? Should they be consumed at all? What about non-renewable resources? Maybe technology improvement and enhance knowledge will allow a different use and regeneration of resources.

Other definition states that sustainable water use is "the use of water that supports the ability of human society to endure and flourish into the indefinite future without undermining the integrity of the hydrological cycle or the ecological systems that depend on it" [7].

A definition of sustainable water resources from an *American Society of Civil Engineers* (ASCE) working group on developing sustainability criteria for water resources systems is given as follows: “Sustainable water resource systems are those designed and managed to fully contribute to the objectives of society, now and in the future, while maintaining their ecological, environmental and engineering integrity" [3].

The *United States Environmental Protection Agency* (USEPA) defines sustainability as "the satisfaction of basic economic, social, and security needs now and in the future without undermining the natural resource base and environmental quality on which life depends" [4].

The *International Water Association* states that "sustainable water systems should provide adequate water quantity and appropriate water quality for a given need, without compromising the future ability to provide this capacity and quality."

According to the *American Water Works Association* sustainability means "providing an adequate and reliable water supply of desired quality, now and for future generations, in a manner that integrates economic growth, environmental protection and social development."

Conceptual Model group of the *Sustainable Water Resources Roundtable* defines the sustainable development of water resources "as a multi-dimensional way of thinking about the connections or interdependencies among natural, social, and economic systems in the use of water" [6].

One can conclude that the sustainable water use is that pattern of use which ensures satisfaction of needs for both the present and future generations. Such a definition is not far from the general definition of sustainability and becomes problematic whenever the current and future generations somehow find themselves competing for access to water resources. Crucial issue is to establish a criterion to resolve the competition between present and future generations to access water resources of good quality.

Best picture of wide definition of sustainable development is portrayed by John Pezzey in his work *Sustainable Development Concepts - An Economic Analysis*, in which he collected 190 definition of sustainable development [21]. Andrew Manderson [17] states that over 300 attempts have been done to define sustainable development.

The key elements which should be included in the sustainable development definition are proper assessment of relevant environmental, economic, and social factors, consideration of expanded temporal and spatial horizons, intergenerational equity and the need for multidisciplinary considerations [3].

3 WATER RESOURCE SUSTAINABILITY ASSESMENT

Rating sustainability is both in literature and practice used in two different contexts. The first covers an assessment of the current state of the system in relation to sustainable (ideal) state, while the second case involves an assessment after the implementation of future planned activities in the system [1]. Water resource system sustainability analysis often uses system analysis approach, which analyzes the components of the system and observing their interrelationship [26].

Sustainability assessment of water resource systems can be generally divided into three methodological approaches [1] which have been developed successively over time, but which are still in use today and which are mutually complement, enhance and interconnected.

The first relates to the period prior to the definition of sustainability given by the report *Our Common Future*, and mainly includes economic approach. Methods used are examining viability of the environmental issues, such as depletion of natural resources, environmental pollution and the impact on biodiversity. Commonly used methods included the evaluation by filling out the questionnaires, project planning, cost analysis and effectiveness, cost-benefit analysis, contingent valuation method and multi-criteria evaluation. Since these methods are used primarily from the economic aspect of the planning and development of investment programs without integral evaluation of sustainability problems, they ceased to be used in water systems sustainability assessment.

The second methodological approach refers to the period after *Agenda 21*, which called for an integrated approach to environmental, social, institutional and economic aspects [2]. As a result, the degree of sustainability is assessed, primarily through the life cycle of the process or system.

Life-cycle assessment is a systematic, quantitative approach that evaluates the impact of matter, project or process through their entire life span, from the extraction of raw materials to the treatment after use, taking into account energy and environmental implications [27]. This way of thinking has evolved from an engineering approach to evaluate the impact of certain processes or products on the environment. Most often life-cycle analysis is used in the context of the impact on the environment and usually is called *environmental life-cycle assessment* (E-LCA).

This approach attempts to look at the bigger picture of sustainability through the integration of sustainability dimensions but often fails because it is primarily addressing environmental issues [22]. The lack of economic consideration resulted in methods of *Life Cycle Cost* (LCC) and later various methods of social life score (*Social Life Cycle Assessment*, SLCA) were developed.

Life-cycle methods are often seen as methods based on standardized methodology that can quantify various impacts on sustainability but still shows weaknesses in terms of complex and time-consuming analysis with multitude of required data [14].

The third methodological approach relates to the current and commonly used methodology based on the use of indicators and criteria for the assessment of sustainability. The methodology is characterized by a holistic approach, with special emphasis on the participation of all stakeholders.

This methodological approach is generally characterized with intermix of different methods as well as the efforts to upgrade the methods from the previous generation that ultimately have a similar or the same purpose. Methods for qualitative evaluation of sustainability through questionnaires are used that result in suggestions for specific activities that leads towards sustainability. This approach uses variety of multi-criteria analysis methods and various mathematical methods of creating the overall sustainability index based on subjectively allocated dimensional indicators.

This third methodological approach considers two types of metrics for measuring sustainability, namely indicators and criteria [15], [22], [9], [12]. Indicators are typically qualitative or quantitative parameters, a piece of information, selected to represent particular dimension of sustainability. An aggregation of indicators is called index. The development of index is usually comprised of the following steps: Building a theoretical framework, which provided the underlying basis for indicator selection, indicator selection, indicator data processing, weighting and aggregation of indicators into an overall index.

Indicators play three important roles in sustainability assessments. They help depict the present condition, facilitate the evaluation of various management actions and they alert users to impending changes. Indicators operate by reducing complexity to simpler but still accurate information to aid decision making at various levels. They represent a comprehensive way of measuring and describing efforts in reaching sustainable state of a process or a system.

Indicators monitor progress, show trends towards sustainability, assist decision-makers and act as a public relation tool. They cannot be used directly in the decision-making process as they only present a static view of the state of the system. Several studies at the urban, regional, and national levels have compiled extensive lists of sustainability indicators. However the general endeavor is at reducing the number of indicators due to the lack of adequate data for matching indicators.

Explicit criteria (rules) should be used to select indicators to ensure that the resulting evaluation is robust and usable in decision - making. These criteria include: availability of high quality data, long term data affordability, system representation, sensitivity to change over time, independence of indicators from one another and supports management decisions and actions. Although all are important criteria, it is possible that a really good indicator does not meet all criteria; however, each indicator should meet most of these criteria.

The other approach in measuring sustainability is by means of criteria. Sustainability criterion is the reference value against which a sustainability indicator is measured. In fact, the literature does not always distinguish between indicators and criteria, and the terms have been used interchangeably. In a broader sense, the term criteria can be identified with the term indicators with known limit (range) values. Many authors advocate a probability based criteria called resilience, vulnerability, and reliability as the crucial criteria when considering sustainability in the water sector. Other authors usually supplement these criteria with some others, such as risk, robustness, reversibility, consensus, entropy, intra-generational equity as well as some others are used [13], [15].

Reliability, resilience and vulnerability criterion was proposed by Daniel Loucks (1997) based on the idea that system sustainability is related to a high degree of reliability and resilience and low vulnerability [15]. Loucks suggested that sustainability can be defined as a separate or weighted combination of the reliability, resilience, and vulnerability indicators of various economic, environmental, ecological, and social components in any system.

Reliability is defined as a probability of failure, resilience is a likelihood of return to normal operation after a failure and vulnerability is considered to be a magnitude of failure. The risk criterion measures the product of the magnitude of a negative effect and the probability of occurrence [24]. Since the reliability and resilience are probabilistic parameters, they can range from 0 to 1. If we determine the vulnerability within the [0 1] interval, then we can combine these three indicators in a way which quantifies the sustainability based on risk criteria. Usually it is done by summing these criteria into single value called relative sustainability.

$$Sustainability = Reliability + Resilience + (1\text{-}Vulnerability)$$

Weighted multi criteria approach is usually used to quantify system sustainability with the following steps: select various environmental, economic, and social indicators that contribute to sustainability, define satisfactory and unsatisfactory ranges of values for each indicator, collect data on indicators over time and express as a time series, analyze time series using statistical measures such as reliability, resilience and vulnerability and calculate the relative sustainability of the system as a weighted combination of the aforementioned criteria. [37]

Most of the proposed sustainability criteria are based on the system analysis approach.

Determination of indicators / criteria is possible in two ways, [16]. The first approach is "top-down", where indicators / criteria are defined by experts and researchers. The other approach is "bottom-up", characterized by defining indicators / criteria through the involvement and interaction of all stakeholders. The second approach has become particularly used after projects of urban recycle water systems failed to be finalized due to lack of public support [14].

The main disadvantage of using indicators in the overall assessment of sustainability, in addition to subjective assessments and usual subjective indicator weighting, is the lack of unified or specific selection criteria as well as the lack of the classes (ranges or degrees) of sustainability. The mere use of indicators without the reference value, that characterize sustainability, does not provide the information about the level of sustainability [25]. The use of indicators gives varying results in different case studies and they usually little influence the management of water resource systems.

4 CONCLUSION

Many different methodologies have been developed which attempt to simplify the assessment of sustainability. The first step in this process includes definition of overall goals, system boundaries and sustainability criteria and indicators that reflect the various dimensions of sustainability, namely, environmental, economic and social. The goals of the sustainability assessment must be well defined. Typically, the purpose of such an analysis is to assess the system-wide sustainability impacts of potential changes.

Quantification of sustainability using indicators is characterized by the ease of measuring indicators value, their interpretation and the possibility of mathematical processing which produces integrated value of particular sustainability dimension. They also indicate how much particular dimension participates in the overall assessment of sustainability.

A summary of sustainability indicators is index that measures the sustainability of water resources systems. It can be used to estimate the sustainability for water users and to obtain the change in sustainability by comparing the index among several water policies proposed. The indicator-based sustainability assessment approach has been used to develop water sustainability indices. Widely used indices are: *Water Poverty Index* (WPI, 2002), *Canadian Water Sustainability Index* (CWSI, 2007), *Watershed Sustainability Index* (WSI, 2007) and *West Java Water Sustainability Index* (WJWSI, 2010) [10], [20].

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GROUNDWATER POLUTION RISKS IN THE REPUBLIC OF MACEDONIA

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Abstract

This paper deals with groundwater pollution risk analyses from industrial, urban, waste dumps and other sources of contamination in the Republic of Macedonia. Among the discussion on main contaminants, example of prepared ground water vulnerability maps isl presented showing the groundwater vulnerable zones. Identified and potential ground water pollution sources are located on the map using computer software. Some historical data on ground water quality are presented, as well as heavy toxic metal distribution maps. The significance of the problem from practical point of view is underlined.

Keywords

Groundwater, map, risks, pollution, vulnerability

1 INTRODUCTION

Due to the groundwater characteristics, their use as drinking water is most acceptable and most common in all areas inhabited by people. Because of this fact, groundwater resources shall be protected, both in aspect of their sustainability as well as in aspect of their protection and quality preservance, in some cases even in terms of their quality improvement. In general, two major problems in regard to the use of ground waters exist:

-Maintenance of the resources yield;

-Maintenance of groundwater quality.

Groundwater pollution can occur as a result of natural reasons (rarely) or as a result of anthropogenic factors, which is the most frequent case. Once the groundwater is polluted, the consequences are long-lasting and they may be present for years, decades and even centuries.

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To avoid problems, it is necessary to have a clear picture of possible pollutant source in the area, and not to build up structures in the vulnerable groundwater areas. Based on this fact, in a frame of this article, the experiences from Republic of Macedonia connected with groundwater pollution risks are explained.

2 METHODOLOGY OF ANALYSES

To predict possibility of groundwater contamination, the steps given in Figure 1, are recommended.

Step 1: Preparation of groundwater vulnerability maps
↓
Step 2: Pollution risks analyses
↓
Step 3: Defining of protection zones for groundwater protection

Fig. 1 Presentation of main steps to predict protection zones for groundwater protection

In broader terms, the assessment of ground water pollution risk comprises the a lot of elements as: potential pollutant characteristics (chemical, physical, biological, temperature, organic or nonorganic origin), geologic formation characteristics, topographic terrain characteristics, climate characteristics of observed area around the aquifer, source of pollutant in terms of special emergence of polluter (spot, line or special source) etc.

Potential sources of pollution on the surface are: infiltration of polluted surface water, landfills, storage of sewerage water or sludge by water treatment plants, road defrosting salt, pesticides and fertilizers, animal breeding plots, surface leakage from urban areas, incidental pollutions etc.

Potential sources of pollution under the earth surface and under the water surface are: septic tank and outdoor toilets, sanitary landfills, waste disposal in excavations, leakages from underground tanks or pipelines, graveyards, mines, dried up wells, ground excavations etc.

Potential of pollution under the water surface can be connected with: drainage wells and channels, underground waste disposal, underground warehouses, mines, exploitation wells, abandoned wells etc.

Knowing potential sources for pollution from one, and the geological, morphological, hydrological and climatic characteristics for defined aquifer area, from the other side, is the basis for further analyses of this important problem. All this gives information about the possible pollutants that shall be taken into consideration. Al these aspects are usually analyzed using field measurements, data analyses, geological, hydro geological and other testing's and using different numerical methodologies for analyses, in order to obtain reliable results for further analyses.

3 RESULTS

As it is presented in Figure 1, the important step in defining the groundwater pollution risks is to prepare groundwater vulnerability maps of the analysed area. Methodology for preparation of vulnerability maps in the country mainly use criteria connected with a permeability of the media □□□□ This is so-called first approximation in prognosis of conditions for possible ground water contamination.

An example of prepared Groundwater Vulnerability Maps based on the methodology for preparation of Basic Hydrogeological maps in the country is given in Figure 2.

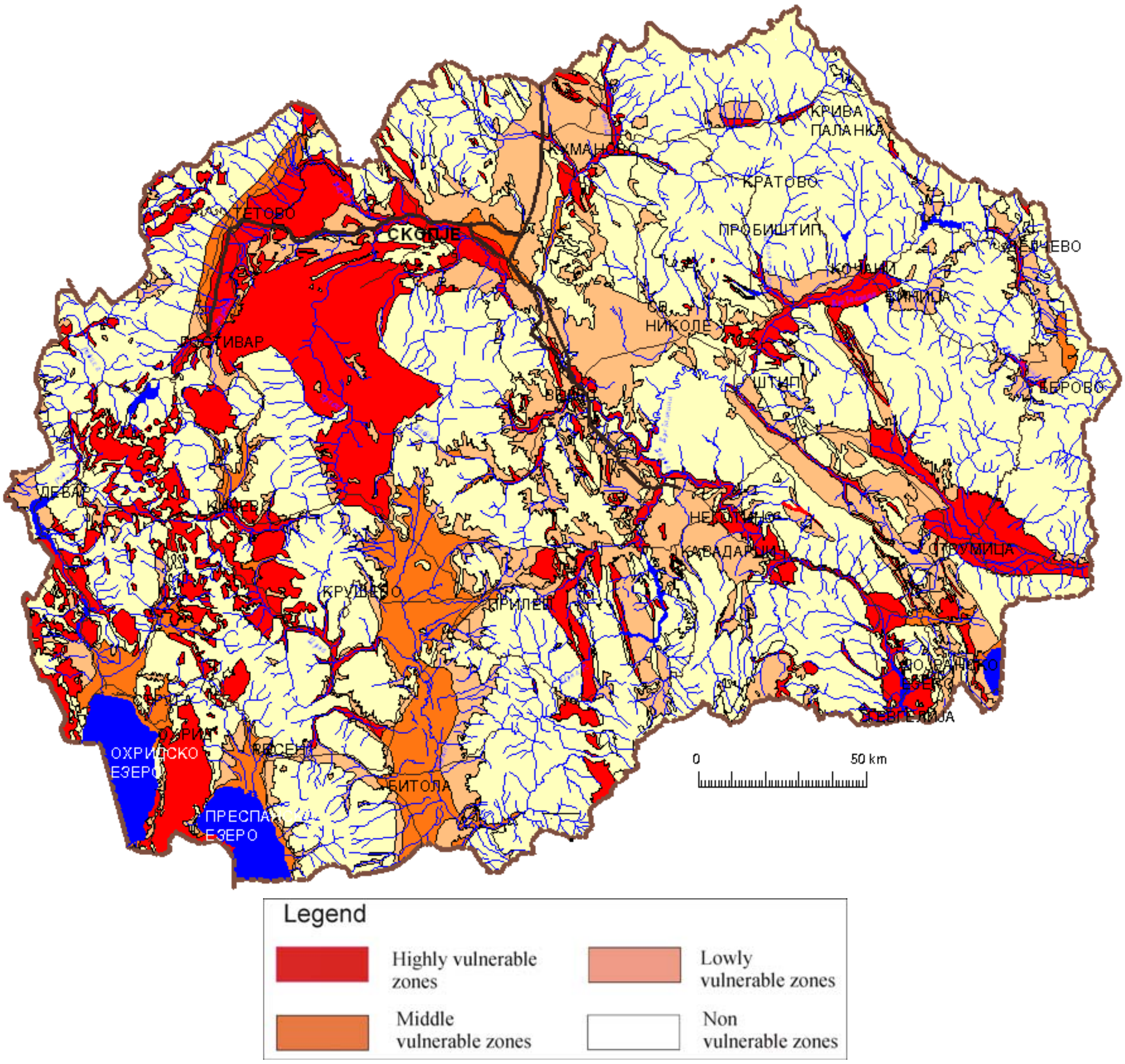

Fig. 2. Example for groundwater vulnerability map of R.Macedonia [5]

It can be noted, that several main vulnerability classes can be defined. Highly vulnerable zones are mainly connected with karts formations and gravel-like sediments without coverage, while middle vulnerable zones are defined mainly with alluvial deposits around river zones. Lowly vulnerable zones are mainly connected with class of low permeability, while non-vulnerable zones with impermeable formations.

Initial step in risk analyses is to define locations of possible pollutants that can affect the aquifer media. Several maps, prepared using some historical data and field analyses are presented in Figure 3 and Figure 4.

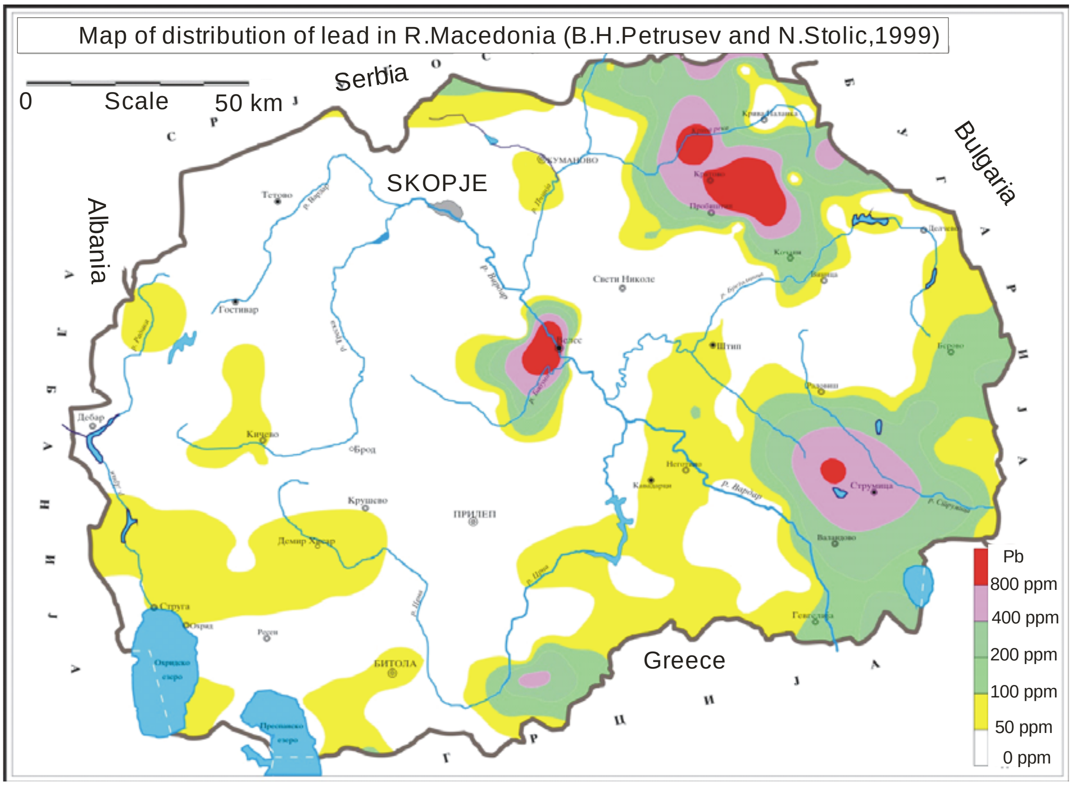

Fig.3. Map with presentation of distribution of lead in the country [9]

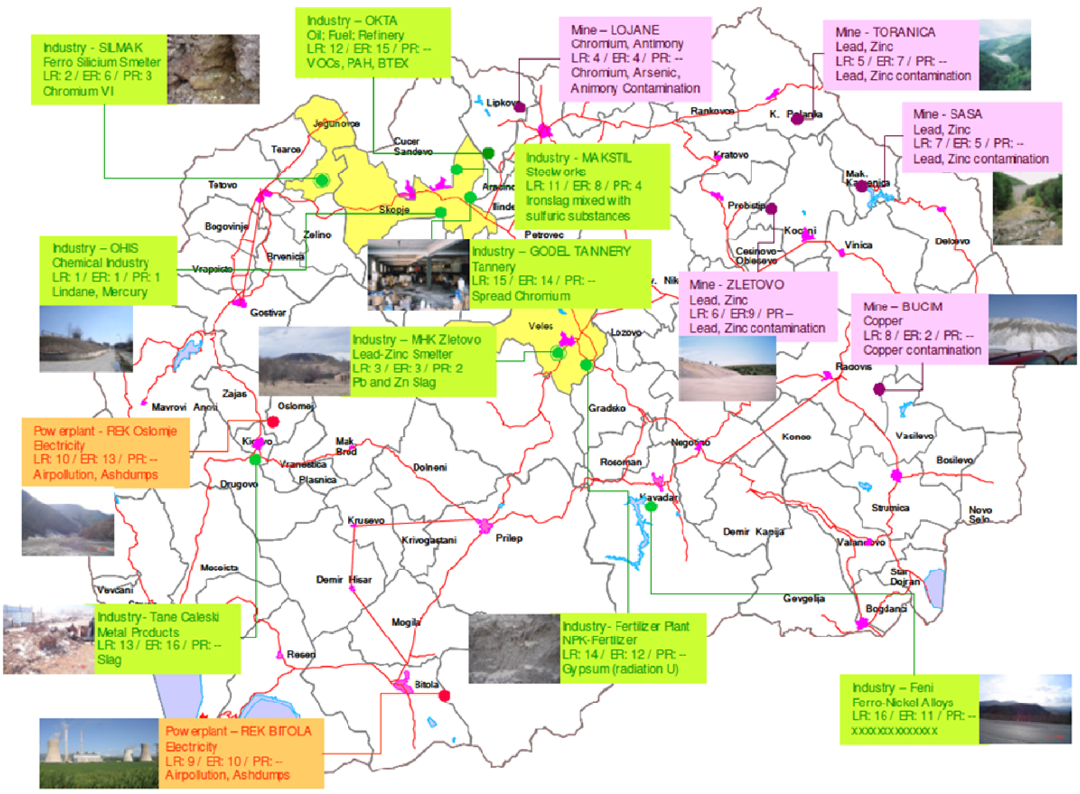

Fig.4. Presentation of main pollutants in the country [4]

4 DISCUSSION

The starting point for groundwater pollution risk assessment can be the application of groundwater data base, of course if there are such bases and if they are regularly updated. In general, all data basis shall have several main entities. Final result shall be preparation of groundwater risk maps (Figure 5).

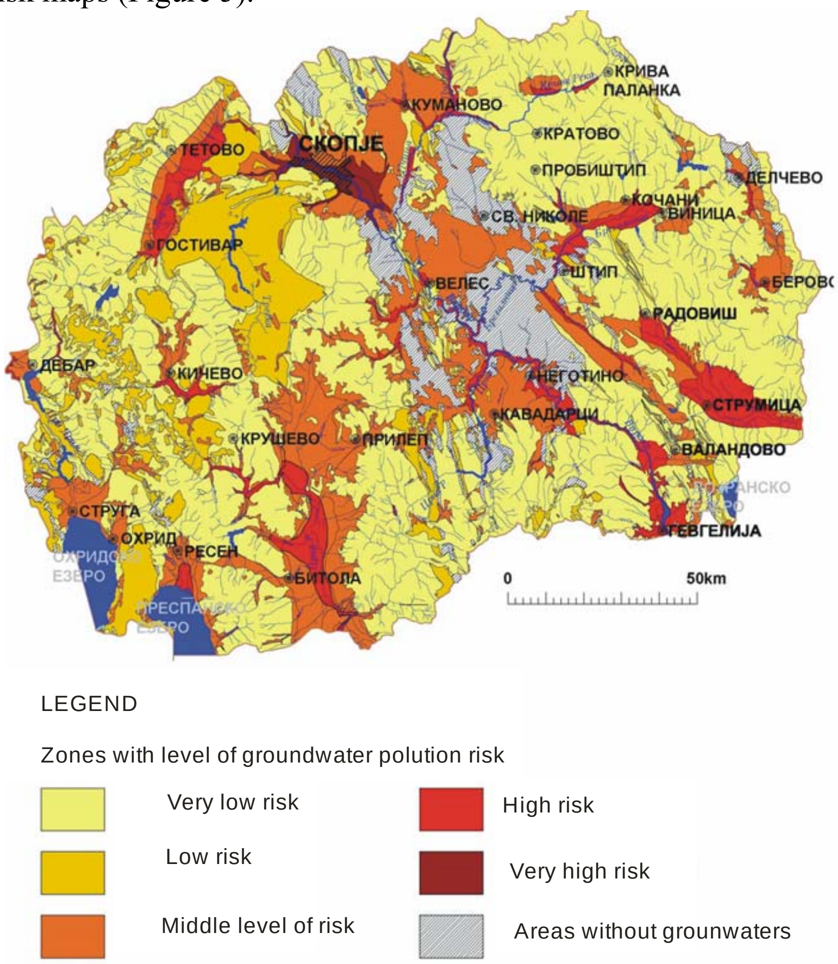

Fig.5. Example for groundwater vulnerability map of R.Macedonia [5]

The main entities that shall be used in preparation of such maps are following [7]:

-Groundwater feature entity.

A groundwater feature is a natural or constructed structure that has allowed access to groundwater i.e. allows use and observance of the water resource. It is not important for pollution risk assessment. It allows recording of the underground flow changes, both in terms of quality and quantity.

-The site entity.

A point on the land surface defined with geographical coordinates that locates the groundwater feature. It has very slight significance for the pollution risk assessment, i.e. may provide some information about the climate impacts.

-The datum plane entity

The datum plane entity is a horizontal plane used as a reference to determine the position of components of a groundwater feature and the elevation of measurements derived from a groundwater feature. It has very slight significance for the pollution risk assessment, i.e. in

conjunction with the site entity; it may provide information about the possible climate impacts.
-The construction element entity
The construction component or element is a component of the man made structure within a groundwater feature. It is not important for pollution risk assessment.
-The construction activity entity
A construction activity is work undertaken on a groundwater feature to maintain or enhance the groundwater supply or quality. It is not important for pollution risk assessment.
-The groundwater source entity
The groundwater gives information about the direct interval of the aquifer cut with the construction. Such discrete intervals of an aquifer may have been isolated in may aquifers in one water resource. It is not important for pollution risk assessment.
-The locality entity
This entity is used to define the site of a groundwater feature in relationship with other geographical features. It has slight significance for the pollution risk assessment, i.e. may provide some information about the topographic impact in terms of surface leakages.
-The setting entity
This entity gives a description of the general environment in the vicinity of the site of a groundwater feature. It is of great importance for the pollution risk assessment and the groundwater quality degradation. It is very useful in hydrogeological studies since it gives detailed information about the use of the surrounding land. For example, land use descriptions may give insights in issues such as potential hazard for groundwater contamination facilities (ex. proximity to heavy manufacturing industry, intensive agriculture, slaughter houses, etc.), traffic, the use of the land which implies presence of fertilizers and pesticides. Also, if the surrounding land is used as pasture or animal breeding farms, it may be a potential hazard for the water resource quality. On basis of the foregoing, this entity is the basic entity for pollution risk assessment and thus it shall always be taken into consideration when conducting pollution risk assessments.
-The status entity
This entity refers to continuous monitoring of certain characteristic of the groundwater feature, or of a component of a groundwater feature, that has been consistently observed over a period of time. Generally, it is not important for pollution risk assessment, but for cases when certain metering instruments for recording changes in the water resource are installed.
-The event entity
An event is a discrete episode of data collection relating to a groundwater feature. This entity may contribute to the assessment of the characteristics of pollutant transmission through the soil, especially in the research works phase because it provides information about the stratigraphy and characteristics of the soil layers that are in immediate proximity of the water resource. This entity provides information about the time necessary for transport of the pollutant to the aquifer and the capacity for its storage, but it is not important for the pollution risk assessment.
-The sample entity
A sample is the object for which properties are observed, measured or interpreted (graphically, numerically or in writing) in association with a certain water resource. This entity has a certain impact on pollution risk assessment, i.e. in the research works phase in the water resource area, it provided information about the soil materials characteristics which is of interest for the forecasting of chemical or physical pollutants present in the soil.

-The parameter entity
This entity is a certain characteristic that parameter is measured, observed or interpreted during or associated with an event. It is not important for pollution risk assessment.
-The result entity
It is a recorded value or outcome of some parameter and it is not important for pollution risk assessment, apart from in connection with the entity parameter.

5 CONCLUSION

According to longer time analyses the special methodology for preparation of ground water vulnerability maps is developed in Republic of Macedonia. This problem is related to the existing groundwater management situation in Macedonia. The given approach here is the basis for definition of groundwater pollution risks as one of the key problems regarding the evaluation of the degree of the exposition to the risk for contamination of aquifer zones. Groundwater pollution risk assessment based on the permanent data bases shall mainly rely on information about the natural properties of the analyzed area and on the analyses of distribution of pollutants related to the natural surroundings. So, in a frame of the article, the risk map as a tool for defining of protection zones is presented.

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METHODOLOGY OF GROUNDWATER RISK ASSESSMENT

Z. Kacevski[1], Z. Ilijovski[2]

Abstract

Recently, the use of methods of risk-based assessment of groundwater systems has become very important. During research into the applicability of risk analysis methods in the Republic of Macedonia, several approaches have been studied. The given method here, is based on the semi-quantitative approach, which includes several Risk Hazard Evaluation Factors (RHEF) grouped into several ranges of values. Since the various parameters are not equally important for estimation of Total Groundwater Risk Evaluation Factor (TGWREF), importance ratings are allocated to the different value ranges and they are connected with polynomial interpolation. With this procedure, each evaluation factor is related to the corresponding ratings using interpolation charts. The results of the analysis suggest the high applicability of the method used.

Keywords

Groundwater, hazard, risk evaluation, vulnerability, zonation

1 INTRODUCTION

Susceptibility of a given area to groundwater pollution is usually determined using hazard and risk zonation techniques. The best approach is when such maps are prepared early in the planning study and developed in more detail as the study progresses. It can be used as a tool to help identify land areas best suited for development by examining the potential risk of groundwater contamination.

Knowing the importance of these aspects, a special attention is given in the European COST Action 620 Project – "*Vulnerability and risk mapping for the protection of carbonate (karst) aquifers*". The scientist from most of European countries tried to develop an approach for

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vulnerability mapping and risk assessment motivated by the demands of the European Water Framework Directive (2000). In the present state of the art, the trend is to use GIS techniques in defining of zones with different vulnerability and risks for groundwater pollution [3]. Special techniques are used for karstic formations as "VURAAS" (**VU**lnerability and **R**isk assessment for **A**lpine **A**quifer **S**ystems) concept was developed in Austria [2].
In the practice, several methods are widely used. The GOD method presented by Foster, 1987, is described as an empirical ranking system for the rapid assessment of aquifer vulnerability to contamination [4]. The reference mentioned this method as having a simple and pragmatic structure. It accounts only three properties that are: Type of aquifer, Grade of consolidation and lithological character and Depth of groundwater. The DRASTIC method is developed for the USEPA by Aller *et al* [4]. The authors analyses, are given in references [5] [6] and [7].
The risk map is therefore a useful tool for decision makers and land-use planners. Anyhow, it is clear that the degree of groundwater contamination hazard is considered relative since it represents the expectation of future pollution occurrence based on the conditions of particular area.
Even with detailed investigation and monitoring, it is very difficult to predict hazards in absolute terms.
Having in mind the complexity of the problem, in an article a methodology that shows how to integrate several approaches in a problem of groundwater risk assessment is presented.

2 METHODOLOGY OF RISK ASSESMENT

The basic of suggested methodology is to find a way to integrate the vulnerability of the groundwater systems with elements of pollution sources.
In this case, vulnerability is defined using method called **MVCRS** (Multiparametar Vulnerability Class Rating System) [7]. For all classification parameters, a correlative regression curves are defined in order to have possibility to assign an adequate rating for all range of values. For Lythological and Aquifer Types, an arbitrary value from 1 to 5 is used as a basis for correlation with ratings. As a final output, the vulnerability class and adequate ratings are calculated (Table 1).

Tab. 1 Classes of vulnerability according to **MVCRS**

Vulnerability class description	Rating
Very low (VL)	0-40
Low (L)	41-55
Middle (M)	56-70
High (H)	71-85
Very high (VH)	86-100

The obtained curves with regression formula are given in Figure 1.

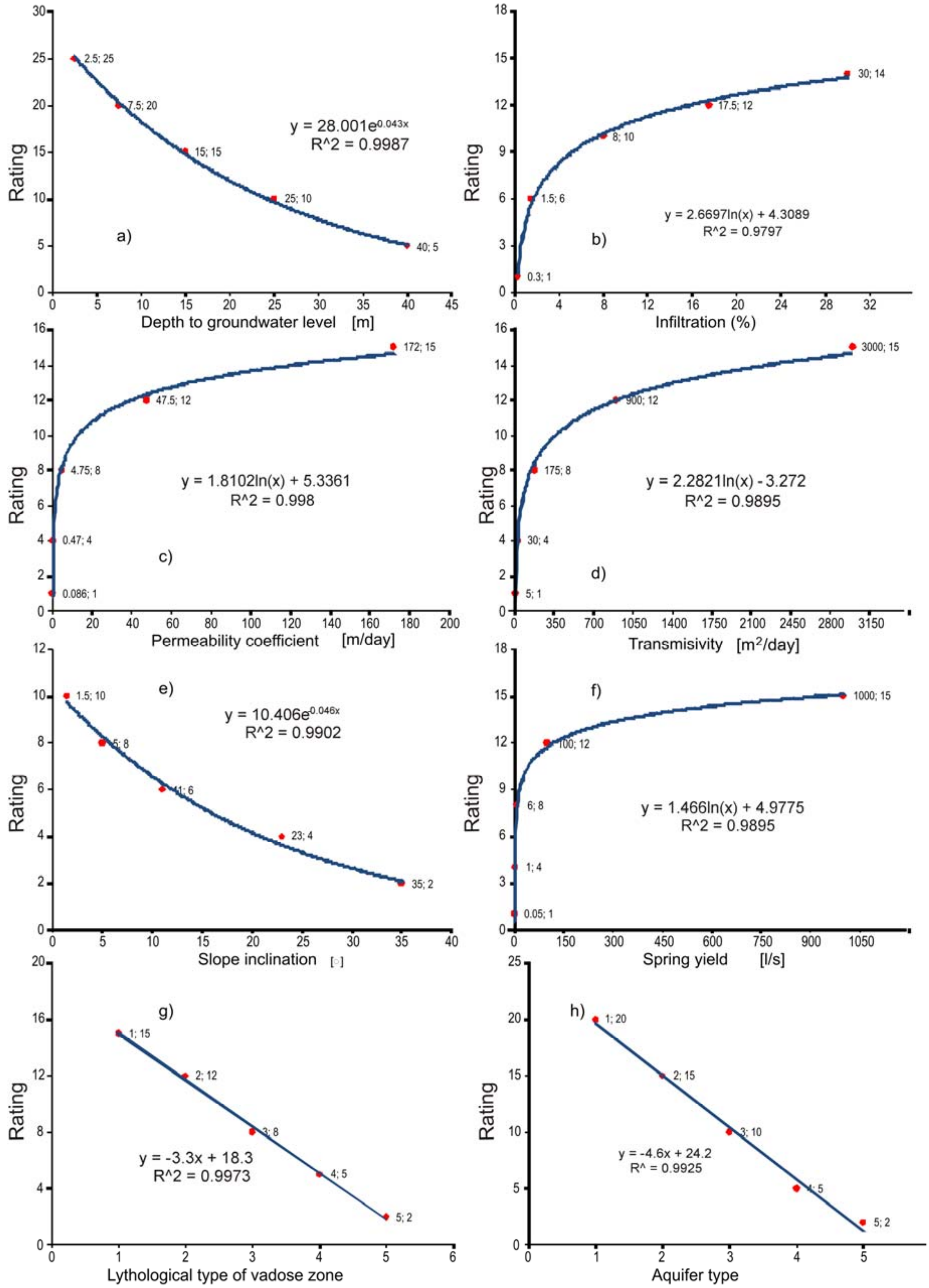

Fig. 1. Ratings for Classification parameters in MVCRS system and regression lines between ratings and a parameter value

Second step is to define most important Risk Hazard Evaluation Factors (RHEF). In our case, RHEF are connected with potential pollution sources divided in tree main categories:

- Point and area pollution sources;
- Land use for agriculture purposes;
- Distribution of population in the country;

Methodology of rating definition of individual evaluation factors is explained by Ilijovski [7], and here only one example in a form of map is given in Figure 2.

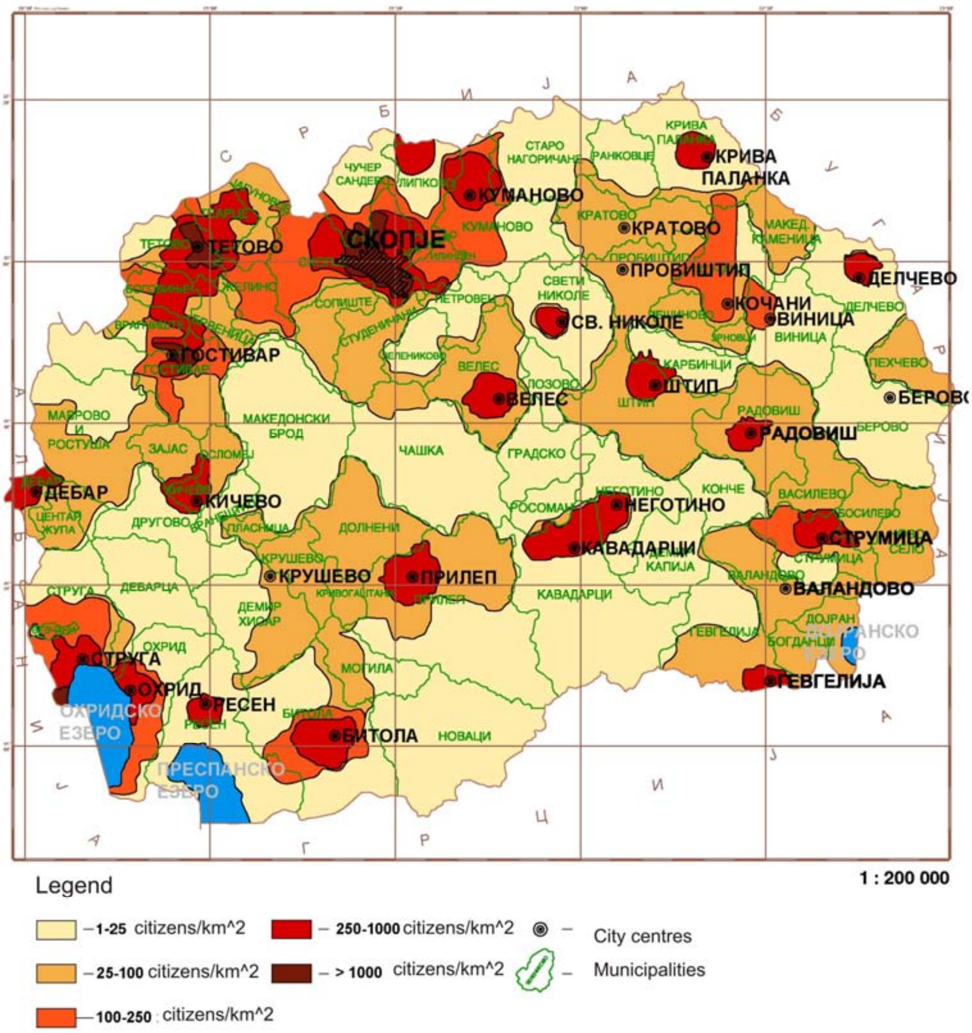

Fig. 2. Distribution of population in Republic of Macedonia

Similar maps with presentation of land use for agriculture, as well as with presentation of main pollution sources are given by Ilijovski [7].

3 RESULTS

The risk mapping includes preparation of geological, hydrological and other models in order to define contamination risk in reliable way. The application of the adequate mapping methodology allows visualisation of groundwater risk contamination and separation of areas with different level of risk for groundwater contamination. The authors methodology is based on separation of five (5) risk categories with ratings from 0 to 40 (Table 2).

Tab. 2 Risk categories (RC) and rating for each category

Risk category	Rating
Very low (Class 5)	0-10
Low (Class 4)	11-16
Middle (Class 3)	17-23
High (Class 2)	24-30
Very high (Class 1)	31-40

Graphical presentation of zones with different groundwater pollution risk is given in Figure 3.

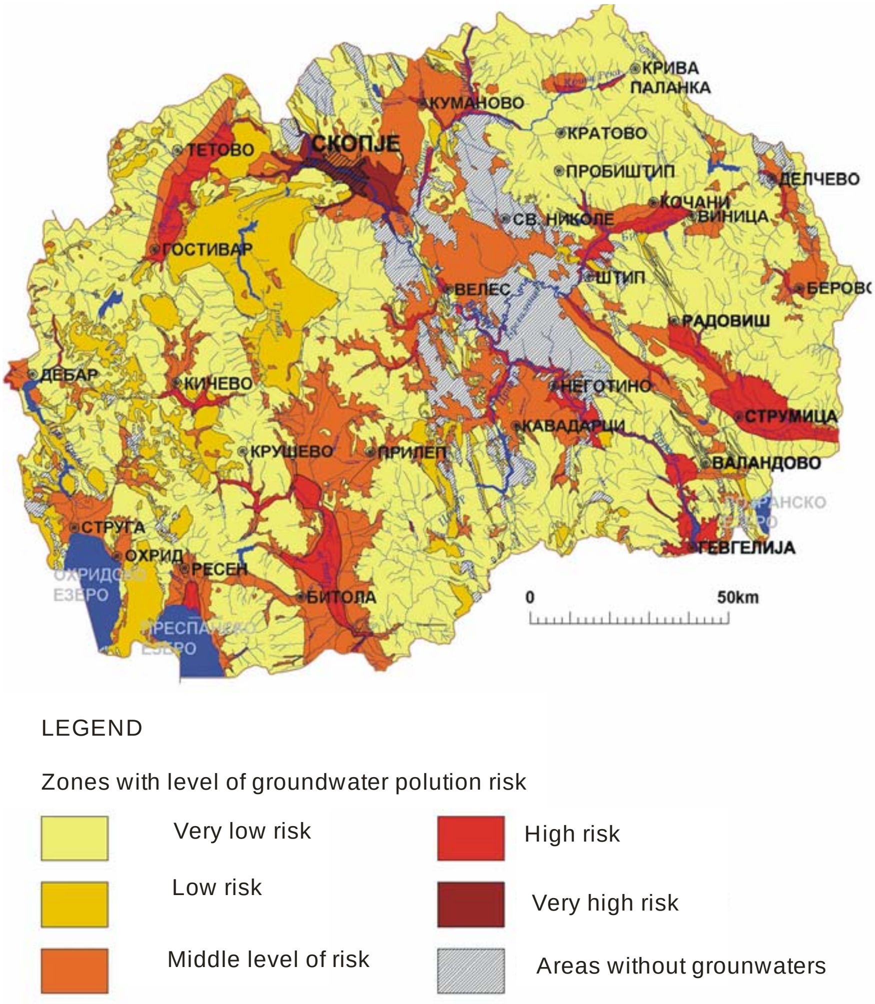

Fig.3. Example for groundwater risk map of R.Macedonia [7]

Some of the evaluation factors are periodic, thus determining various development stages for the groundwater pollution risk. The occurrence of a new stage of evolution presumes the influence of the preponderance of the evaluation factors, which increases the degree of complexity and makes the problem very difficult. From practical point of view it is necessary to find a connection between intrinsic vulnerability of the media, and risk classes. Here, the so-called Total Groundwater Risk Evaluation Factor (TGWREF) is suggested as an output from vulnerability and risk ratings (Figure 4).

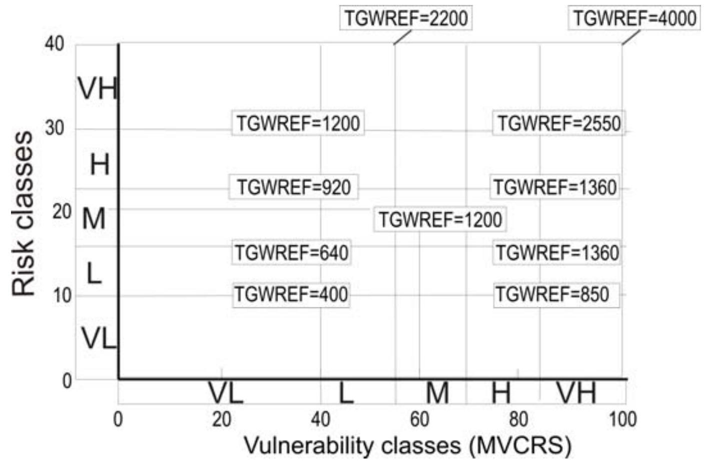

Fig.4. Diagram for connection between vulnerability and risk classes

Multiplying the values for vulnerability and risk ratings, it can be noted that the minimal values for zones without risks will have the value of TGWREF=0-400. The maximal value can be TGWREF=0-4000. Knowing the RHEF and MVCRS value, it is easy to calculate the TGWREF for any grid point at the map by predefined arithmetic calculations. This can be fundamental for successful design of protection zones for some important aqufer zone, and it is a field for future authors occupation.

4 CONCLUSION

The presented methodology is one of the possible ways for groundwater risk assessment, and it can be described as semi-empirical method based on certain level of experience. The philosophy of the methodology presumes dynamical, critical and development period, relatively it should be subjected to critical reviewing in time, and should be used in combination with other methods developed for this problem.

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INFLUENCE OF SOIL TYPE ON THE INTERACTION WITH THE GROUND WATER

B. Kandra[1] and M. Gomboš[2]

Abstract

Soil environment is part of the hydrological cycle. It is composed of two subsystems consisting of aeration and saturated zone. Their interface is the active area, which is determined by the position of ground water level. It is necessary to understand balance of the water in the aeration soil zone and quantify the interactions with subsystems of atmosphere, vegetation cover and ground water level. Integral impact of system components determines course of the soil water storage in the aeration zone throughout the whole vegetation period. It can be quantified either by a direct monitoring, or by a numerical simulation using soil water regime models. Overall dynamics of water in the aeration zone is significantly affected by the hydrophysical properties of soils. Their accurate determination allows a studying of the water regime in different soil types.

In this article was analysed the impact of selected soil types on the dynamics of soil water regime. The individual components entering to balance equations were simulated via numerical model GLOBAL. In this study, time courses and sums of the capillary water inflow and the drainage outflow through the aeration zone lower limit are presented, during the vegetation period 2004 in the Milhostov locality (Eastern Slovakia Lowland).

Keywords

Capillary inflow, drainage outflow, soil water storage, numerical simulation

1 INTRODUCTION

The soil is fixed place of plants, as well as their existential environment constitutes one of the subsystems in the hydrological cycle: atmosphere – vegetation cover – aeration zone of soil – ground water level (A-VC-AZ-GWL). Its quality is therefore partly linked to the nature and dynamics of water regime. Through the water storage the interaction between individual subsystems and aeration zone can be quantified. The overall dynamics of water storage in the

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aeration zone is at the bottom of the active layer directly affected by the position of ground water level and at the upper edge by the atmosphere and vegetation cover [1]. Taking into account this fact and based on the initial volume of water in the soil (W_0) at time (t_0), then at time (t), the volume of water will (W_t). Change in soil moisture condition ($\Delta W = W_t - W_0$) in the time period ($\Delta t = t - t_0$) can be characterized by a simple balance equation (1) [2].

$$(W_t - W_0) = F_c + P - E - T - F_d \quad (1)$$

where
W_0 - initial water content of the soil aeration zone,
W_t - water content of the considered term in the hydrological year,
F_c - capillary inflow,
P - water inflow from precipitation,
E - water outflow due to physical evaporation from the soil surface,
T - water vapour outflow from the plant cover, i.e. transpiration,
F_d - drain water from the zone of aeration via infiltration into the lower horizons, respectively into the ground water.

Dynamics of soil water regime in terms of time and spatial expression has its specific. In space are these specific determined by soils textural composition [3], [4]. During quantification of the water regime elements by the methods of numerical simulations the hydrophysical characteristics of soils are an essential part of the mathematical models. Within the soil types are such inputs the retention curves [5]. These can be obtained through analytical expression of the points measured in the laboratory or by using easily measurable indicator such as soil texture. Determination of the retention curves parameters and saturated hydraulic conductivity values from the soil texture is made by subsequent calculation through the pedotransfer functions. In this paper are quantified the selected components of the water regime by the numerical simulations. The amounts of capillary inflow and drainage outflow through the lower boundary of the active horizon of selected soil types were analysed. Using the model allow to analyse the influence of soil type and ground water level position on selected components of water regime under the same conditions. Also been described mutual interactions between other subsystems and soil aeration zone too.

2 MATERIAL AND METHODS

Analysis of selected balance equation elements was performed within four soil types. Those indicators characterize the interaction of groundwater level (GWL) with soil aeration zone (AZ). The interaction of these two subsystems within the system A-VC-AZ-GWL takes place at the bottom of the active soil horizon, i.e. on the interface of AZ and GWL. Selection of soil types was conducted by USDA triangular classification diagram, according to which the selected soil profiles fall into four groups: the clay, silty loam, loam and loamy sand. The values of the investigated parameters were obtained by calculation using the mathematical model GLOBAL [6]. Influence of soil hydrophysical differences were taken into account through the model inputs. Soil retention curve as a basic hydrophysical characteristic and its parameters were calculated using the RETC software from the soil texture composition [7]. Other inputs to the model are meteorological and phenological characteristics describing local conditions and the type of vegetation. For this purpose was chosen Milhostov locality, situated in the central part of the Eastern Slovakian Lowland (ESL). Necessary daily meteorological data and phenological characteristics along with weekly-course of GWL for the vegetation period 2004 were obtained from databases of SHMÚ. Choosing this period was based on a previous model verification. Verification was based on mutual comparison of calculated and measured soil water courses to a depth of 80 cm for a period of eight years (2000-2007). In the growing season 2004, the coefficient of linear regression demonstrated the highest dependence level of these values (R^2 = 0,7972). This paper analysed vegetation totals of capillary inflow (F_c) and drainage outflow (F_d) in different types of soil in relation to soil water storage (WS) to a depth of 100 cm. This depth include the highest proportion of root mass of most crop plants. These indicators were simultaneously compared with the average positions of GWL. With a view to preserving the real course it was 5 different positions below soil surface: 82 cm, 107 cm, 132 cm (real), 232 cm and 307 cm.

3 RESULTS

Chronoisolines in fig. 1 shows the distribution of moisture in the investigated soil types in the vertical direction and in time.

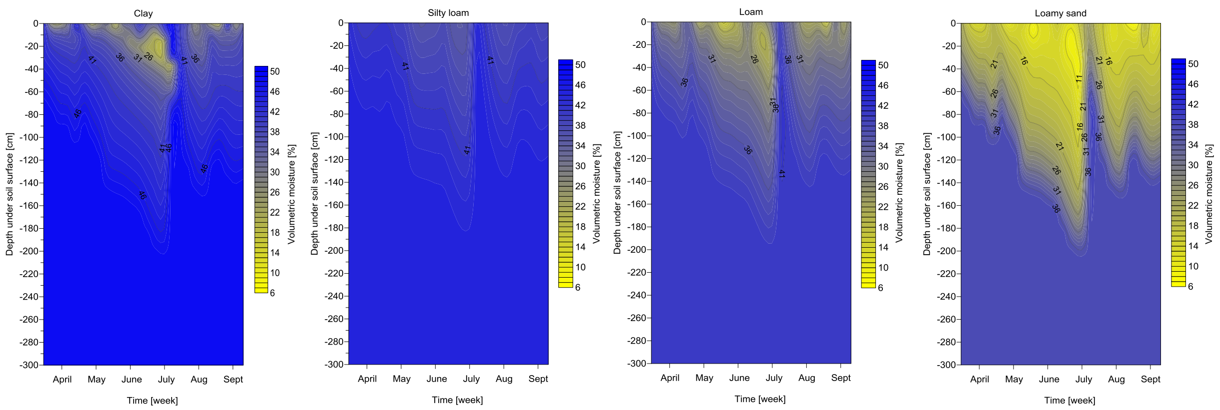

Fig. 1 Vertical and time distribution of volumetric moisture in the soil types

The values of volumetric moisture for vegetation period 2004 were calculated from the actual course of GWL. The driest is the profile of lightweight loamy sand soil. Volumetric moisture here ranges from 7,36 % to 37,34 %. Humid condition can be observed in the heavier silty loam soils, where values vary between 34,88 % to 44,46 %. Somewhere in the middle is clayey and loamy profile with values from 14,33 % to 48,52 %. The vegetation totals of F_c and F_d corresponds with the moisture courses (Fig. 2). The F_d predominates in lightweight loamy sand soil under real average position of GWL (132 cm). The F_c dominates in the other profiles, however the most in silty loam soil. The vegetation totals of F_c is going up with an increase in the average level of GWL (at 82 cm and 107 cm). Here is possible in lightweight loamy sand soil to observe a positive balance between F_c and F_d in favour of F_c. Conversely reducing the average depth of GWL at 232 cm, F_d begins to dominate in clayey and loamy sand soil. By decreasing at 307 cm similarly in loamy profile. In the silty loam profile remains even at maximum reduction GWL prevail F_c. Tab. 1 shows the balance between F_c and F_d ($\Delta F = F_c - F_d$) along with the average WS in the studied soils and GWL positions. Effect of soil hydrophysical differences in their interaction with GWL can be directly observed on WS. Fig. 3 illustrates the changes in average WS according to GWL.

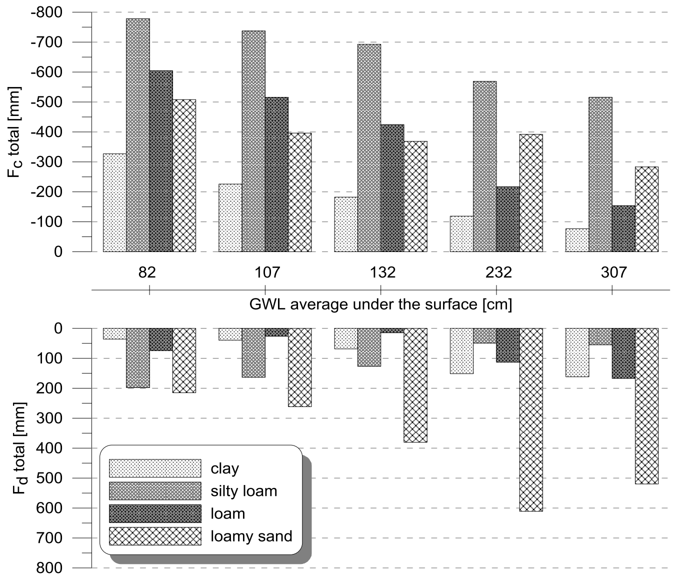

Fig. 2 Vegetation totals of F_c and F_d under average GWL positions in the soil types

Tab. 1 Balance between F_c(-) and F_d(+) with WS averages under GWL

GWL [cm]	clay		silty loam		loam		loamy sand	
	ΔF	WS	ΔF	WS	ΔF	WS	ΔF	WS
82	-291,23	429,35	-580,45	426,80	-530,59	366,18	-292,96	280,29
107	-186,64	410,39	-573,59	418,26	-488,80	346,74	-134,63	241,35
132 (real)	-113,84	393,93	-565,83	407,62	-409,84	326,58	11,86	206,23
232	32,55	352,41	-519,53	353,91	-104,54	260,38	219,11	138,05
307	84,90	345,25	-460,82	311,43	13,36	233,56	236,48	129,47

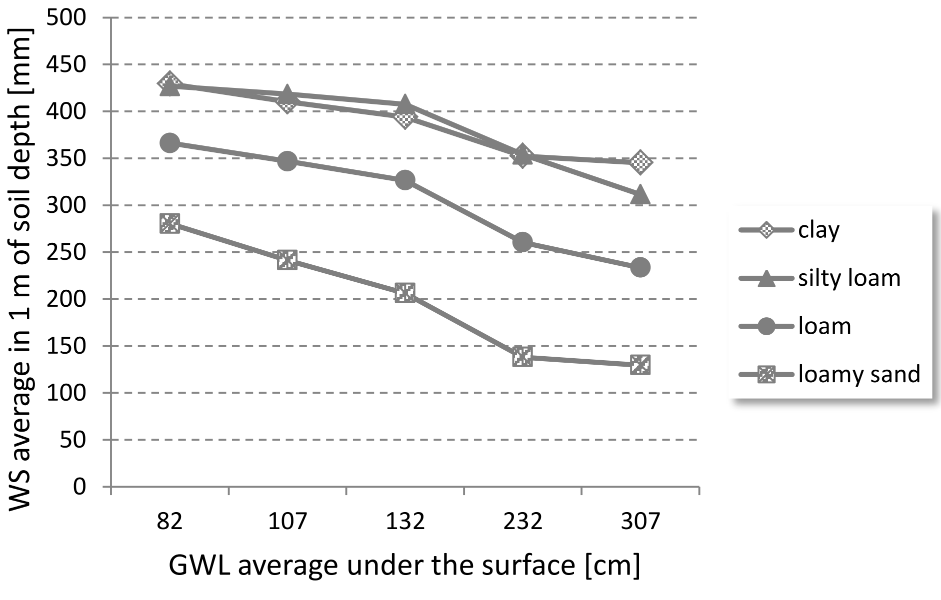

Fig. 3 Average values of WS under GWL during vegetation season 2004

The values of WS are closest in clayey and silty loam soils. Close relationship between the position of GWL and lots of capillary obtained water seen on the fig. 4.

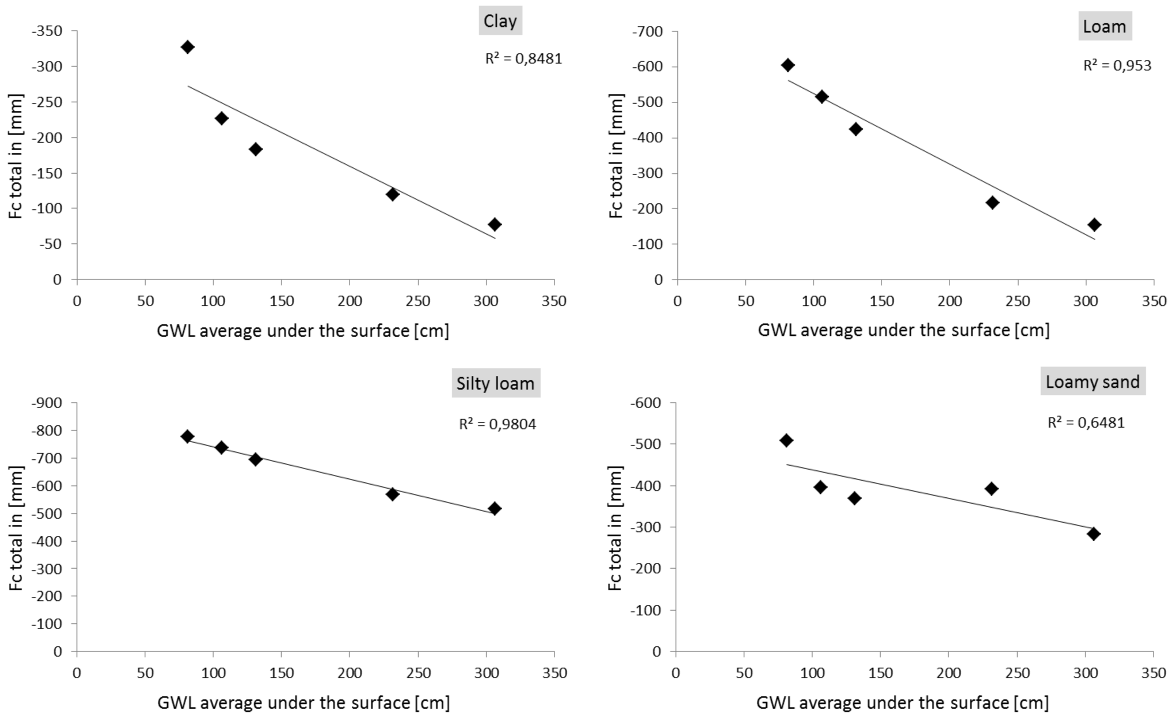

Fig. 4 Linear relationship between F_c and GWL

The strongest bond between them expresses linear regression coefficient in loam, silty loam and clay soils and conversely weakest in loamy sand soil.

4 DISCUSSION

In conditions of Milhostov locality the influence of soil type and GWL position on capillary inflow and drainage outflow through the lower boundary of aeration zone was studied. The values of these members of balance equation (1) were obtained by calculation using the model GLOBAL. From the difference of F_c and F_d follows that F_c dominates at all levels of GWL in the silty loam soil (tab. 1). By contrast, in lightweight loamy sand soil is predominant F_c observed after increase in average level of GWL at 107 cm. In the greatest extent the F_d exceeds the F_c in loamy sand soil. In the clay profile this occurs only after decreasing the average level of GWL at 232 cm and in loamy profile at 307 cm. On chronoisolines (fig. 1) can see the influence of soil environment on moisture distribution in space and time in the real course of GWL. Lightweight loamy sand profile in which the F_d dominates has the driest conditions and heavier silty loam profile with predominant F_c is conversely most humid.

5 CONCLUSION

When evaluating selected components of the soil water regime entering to the balance equations has been demonstrated the suitability of computational approach. Using the mathematical model GLOBAL was possible to take advantage of the repeatability of inputs characterizing meteorological and phenological conditions in the chosen location for the growing season 2004. Through other inputs describing hydrophysical characteristics of soils in the selected soil types were investigated vegetation totals of F_c and F_d relative to average values of WS to 1m. At the same time was also evaluated impact of GWL position on the development of the indicators through the lower boundary condition in the model. Position of GWL presents a bottom edge of the active horizon through which is realized mutual interaction with AZ. The water storage in AZ, within the balance of F_c and F_d is reliable indicator of dominance of one or the other member. Position of GWL in conjunction with soil types is crucial on the amount of capillary received water and hence the water storage in the balance zone of soil.

Acknowledgements

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LONG-TERM PERSPECTIVE FOR SALT WEDGE INTRUSION IN RIVER MIRNA

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Abstract

Numerical analysis of saline wedge intrusion in river Mirna for the periods 2007/2008 and 2019/2020 is presented in this paper. The 3D numerical model covers the area of Novigrad Bay, analyzing the fields of the sea currents, temperature and salinity in the unsteady conditions of the freshwater inflow of river Mirna. The model open boundary conditions for 2007/2008 are referenced on CTD measurements in front of the entrance of Novigrad Bay and the sea surface elevations. Analyses related to the future hypothetical condition in the period 2019/2020 are based on the same boundary conditions in terms of the sea temperature/salinity distribution at the model open boundary and atmosphere forcing at the air-sea interface. The influence of changes in the climate is taken into account through an increase in the mean sea level and a decrease in the mean annual discharge of the Mirna in relation to 2007/2008. The results of the analysis indicate significantly increased upstream salt water intrusion in the 2019/2020 simulation period in comparison to that of 2007/2008.

Keywords

3D numerical model, climate change, river discharge decrease, salt wedge intrusion, sea level increase

1 INTRODUCTION

The Croatian littoral and its karst hinterland have just a few surface water flows with a satisfactory water balance for its immediate water catchment. This is the outcome of a relatively poor hydrographic network and inadequate forming conditions for watertight reservoirs. Trends in natural processes such as a reduction in precipitation and surface flows, together with the lowering of ground water and a rise in the sea level, increase the problem of the sea and freshwater relationship in karst littoral aquifers [14]. The danger of freshwater

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salinization is not exclusively related to karst littoral springs and aquifers, but also to the river mouths and the lower courses of rivers. In that case, the salinization is more emphasized during drought periods when the freshwater discharge is low. Low discharge can also be caused by water consumption for irrigation or by other increases in water demand. In the case of small bed slopes, a low freshwater discharge and a high sea water level, more intensive intrusion of saline water in the surface freshwater system can be expected. Consequently, wedges of saline water penetrate more frequently and farther upstream [3], [16]. The recirculation of saline and fresh water is also detected upstream of the Mirna river mouth, especially during the low discharge period [14].

One of the key factors affecting the relationship of fresh and saline water in underground aquifers is the exploitation of underground fresh water itself. Drawing fresh water from a well causes the intrusion of saline water into the body of fresh water. Saline water forms a cone of ascension, and fresh water forms a cone of depression around the well. The intensity of the conical ascension of saline water depends on permeability of the ground, the inflow of fresh water from the hinterland and the depth of the well. The conical ascension of saline water is most noticeable during the dry season [2]. In the surrounding regions of the coastal Istrian towns of Rovinj, Poreč, Umag and Novigrad, there are significant agricultural areas with many, rather shallow, dug wells dating from the early 20th century. Deeper wells have been drilled for the needs of the Istrian water supply system. From the 1980s to the 2000s, a number of wells were drilled for irrigation needs and most of them are situated near the shoreline in low altitude terrain [14]. However, the influence of wells on the saline and fresh water relationship is not analyzed in this paper.

Changes in the water level and other processes in the Mediterranean and Adriatic Sea, alter the costal karst aquifers. The current climate changes are an additional problem for the management of costal karst aquifers [3]. Melting of the permanent ice cover and, consequently, a rise in the sea level will probably continue [11], [9]. This implies an imbalance in interface between fresh and saline water and more problems in the utilization of fresh water resources, especially those in karst media [7].

Besides the rise in the sea level, the expected long-term decrease of discharge in rivers, due to the reduction of precipitation will also stimulate the upstream intrusion of a saline water wedge [10], [15].

This paper analyses two key factors in the amplification of the salt wedge intrusion in the River Mirna: the rise in the sea level and the lowering of the Mirna discharge. Analyses are conducted for two periods: November 2007 - November 2008 and November 2019 - November 2020.

Statistical analysis of recorded sea levels from 1955 to 2009 at the measurement station Rovinj indicates a rise in the sea level of 0.45 mm per year. If the analyzed period is limited to 1993-2009, then the sea level rise is even more visible with a value of 0.91 mm per year. Taking into account the position of Novigrad Bay and the River Mirna estuary, a sea level rise of 1 mm per year is adopted within numerical simulations. This implies a 12 mm sea level rise from 2008 to 2020. Mean annual discharge decrease is defined by linear regression equation gained by analyzing data collected from “Portonski most” measurement station in the period

1955-2008 [15]. The mean annual discharge of the Mirna in 2008 was 5.4 m^3/s. It is estimated that the discharge will decrease by 14% by 2020 and will fall to 4.6 m^3/s. Changes in sea temperature and salinity, along with changes in the air temperature, were not taken into account in this numerical simulation.

The numerical modeling setup and the used modeling approach is presented in the next section along with a description of the initial and boundary conditions. The results of numerical modeling are presented and discussed in the third section, while concluding remarks are given in the fourth section.

2 IMPLEMENTATION OF THE NUMERICAL MODEL

Model Mike 3 [13] is used with two modules: a 3D hydrodynamic (HD) module for the computation of the sea level dynamics and currents, also including heat exchange with the atmosphere, and an advective-dispersive (AD) module for the computation of scalar-field transport.

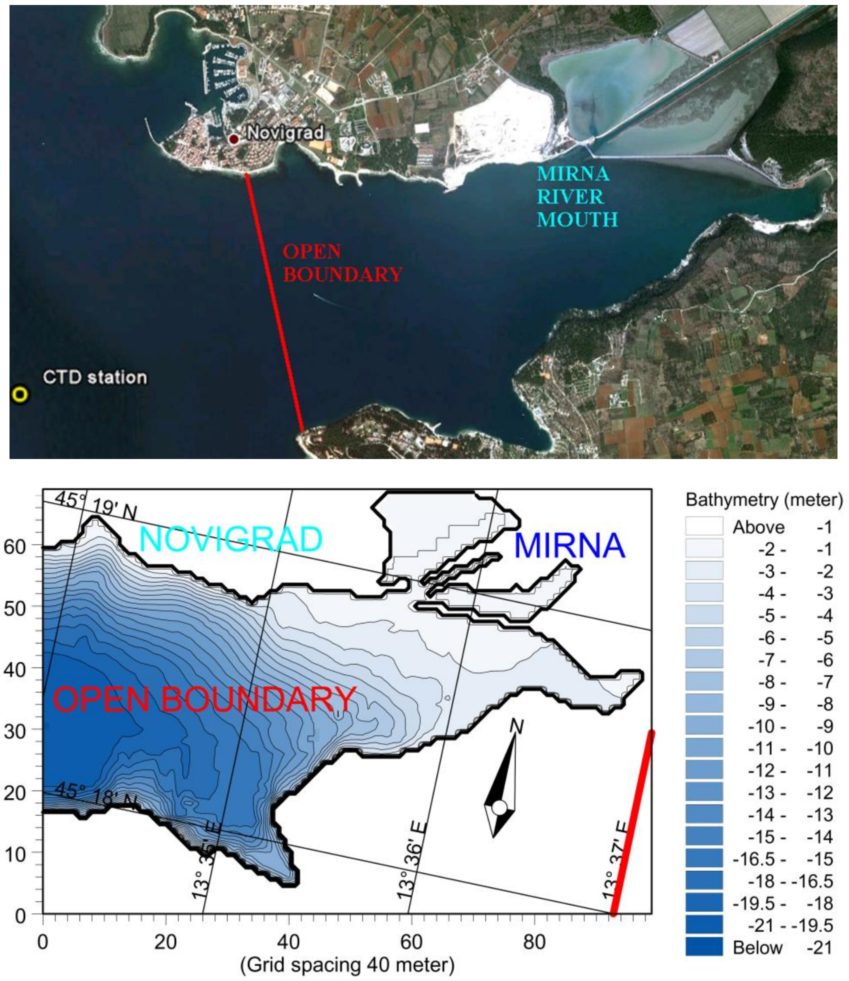

Fig. 1 Satellite photograph of the Novigrad area (above) and spatial domain of the Novigrad Bay area with bathymetry isolines used in the numerical models (below)

The mathematical foundation of Mike 3 is the mass conservation equation and the Reynolds-averaged Navier-Stokes equations including the effect of turbulence and variable density, together with the conservation equations for salinity and temperature. The hydrodynamic module of Mike 3 uses the so-called Alternating Direction Implicit (ADI) technique to integrate equations for mass and momentum conservation in the space-time domain. The equation matrices were resolved by a Double Sweep (DS) algorithm, discretized on an Arakawa C-grid aiming at second order accuracy on all terms. All simulations were executed using a non-hydrostatic option, a k-ε turbulence closure formulation in the vertical direction and the Smagorinsky concept in the horizontal direction. To analyze the transported scalar fields, a 3D Quickest-Sharp scheme was used.

Two separate model domains with two different spatial discretizations were implemented. The course equidistant discretization grid $\Delta x = \Delta y = 40$ m and $\Delta z = 1$ m was used on the larger model domain which included the area of Novigrad Bay and the River Mirna 400 m upstream of the river mouth (Fig. 1). The result values of the sea level, temperature and salinity at the river mouth cross section were used as boundary conditions for the second model domain. That domain included the segment of the River Mirna from the river mouth to a cross section 9000 m upstream, utilizing the finer discretization grid $\Delta x = \Delta y = 10$ m and $\Delta z = 0.2$ m. The channel bed was approximated with a constant slope. The channel bed elevation values were -1.8 m at the river mouth, and -1.0 m at the upstream end of the modeled segment.

No flooding outside the river banks was assumed during the entire simulation period. Fig. 2 shows the adopted Mirna hydrograph during the simulation period of November 2007 - November 2008. Fig. 2 also shows the hypothetical hydrograph used in the simulation for November 2019 - November 2020, where the discharge values are reduced by 14% in relation to those from November 2007 - November 2008.

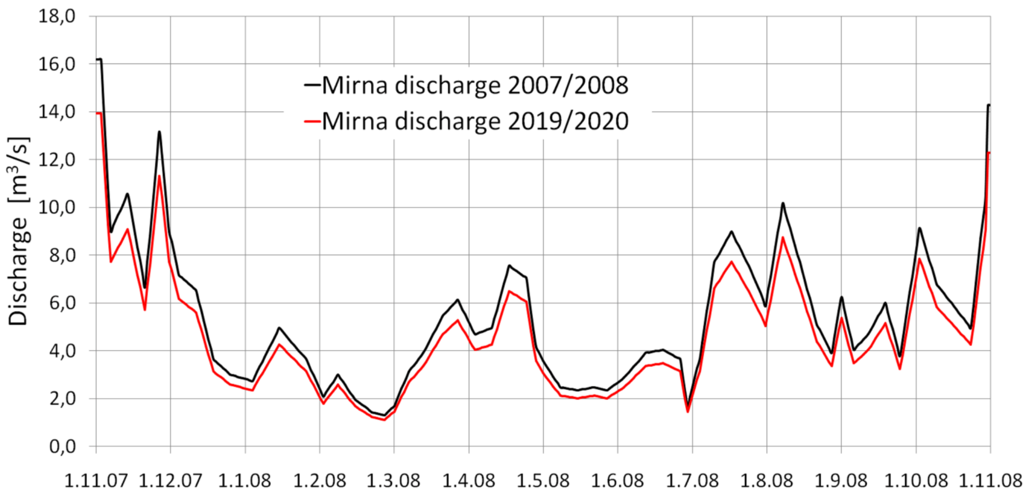

Fig. 2 Mirna hydrograph during the simulation period 1/11/2007 - 1/11/2008 along with the hypothetical discharge for the simulation period of 1/11/2019 - 1/11/2020

The calculation time step used in the numerical integration was set to $\Delta t = 10$ s (spatial domain with coarse discretization) and $\Delta t = 1$ s (spatial domain with fine discretization).

Numerical calculations covered the periods from 1/11/2007 to 1/11/2008 and from 1/11/2019 to 1/11/2020. The model output data sets include sea levels, sea currents (*u*, *v* and *w* components), sea temperature (*T*) and salinity (*S*).

The available data for vertical *T* and *S* distribution were measured at the CTD site situated in the vicinity of the model open boundary (Fig. 3). Measurements were carried out on 29/10/2007, 13/3/2008, 26/5/2008, 27/6/2008, 17/7/2008, 20/8/2008 and 18/9/2008 (Fig. 3) in the scope of the *Adriatic Sea Monitoring Program* [1]. The measured *T* and *S* values were used directly for boundary forcing. The time variability in *T* and *S* at the open boundary during the simulated period was gained using linear interpolation between measurements. Initial and open boundary *T* and *S* 3D fields were defined on the basis of the *T* and *S* profiles measured on 29/10/2007 (Fig. 3). Furthermore, the *T* and *S* fields were unified in the horizontal direction across the whole model domain. The sea level dynamics at the open boundary were synthesized using seven major tidal constituents M2, S2, K2, N2, K1, O1 and P1 (Fig. 4). The only difference between the sea level dynamics for the 2007/2008 simulation and the 2019/2020 simulation is related to the 12 mm increase in the mean sea level.

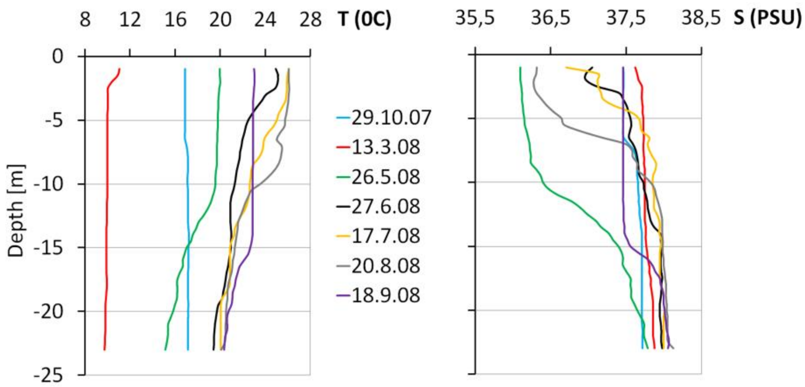

Fig. 3 Measured temperature (T) and salinity (S) values at the CTD station situated in the front of the entrance of Novigrad Bay (see Fig. 1)

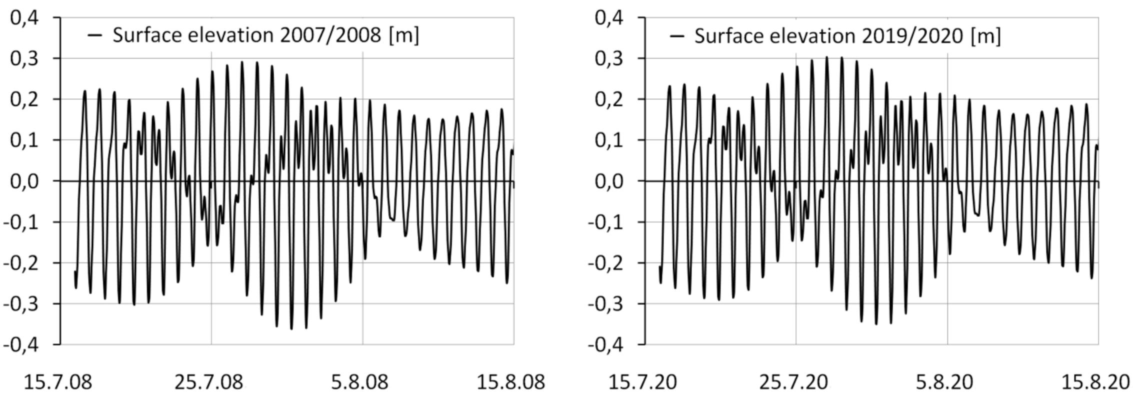

Fig. 4 Sequence of the surface elevation time series used for the model boundary forcing with one hour temporal resolution (left – simulation 2007/2008; right – simulation 2019/2020)

Information on wind velocities and directions for the analyzed period from November 2007 to November 2008 is available from simulations made with the Aladin-HR atmospheric numerical model used by the Croatian Meteorological Service [8], [6], [4], [5], [12]. The version of the Aladin-HR model used is run on a domain which covers the Adriatic region, together with the orography of the surrounding Alps, the Dinaric Alps and the Apennines, with a horizontal resolution of 8 km and a time interval of 3 h. The time series of wind speeds and directions for the second half of July and the first half of August 2008 on the basis of the Aladin-HR model results are shown in Fig. 5. The time series of air temperature, air humidity and cloudiness were also obtained by means of the Aladin-HR model (Fig. 6). The same data sets were used for both of the simulation periods: 2007/2008 and 2019/2020.

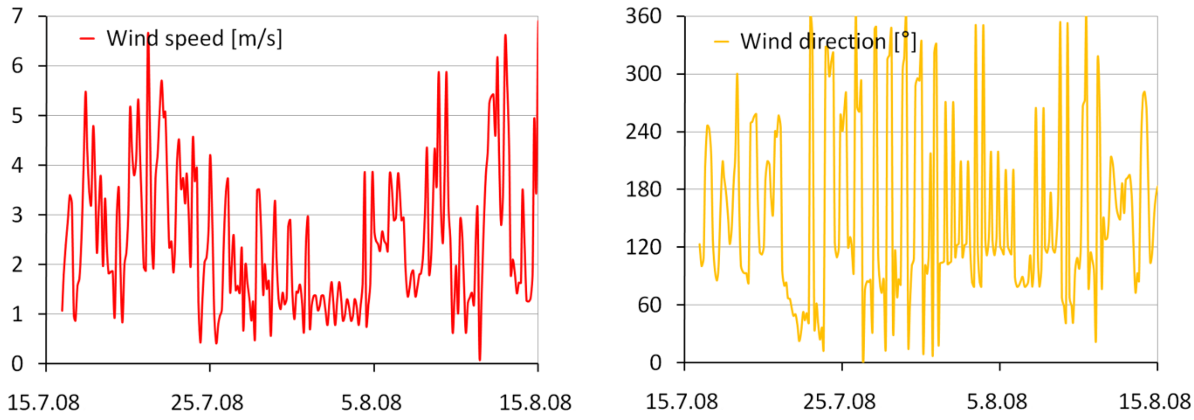

Fig. 5 Sequence of wind speed (left) and wind direction (right) time series (10 m above sea level) with a three hour temporal resolution (extracted from the Aladin-HR model results)

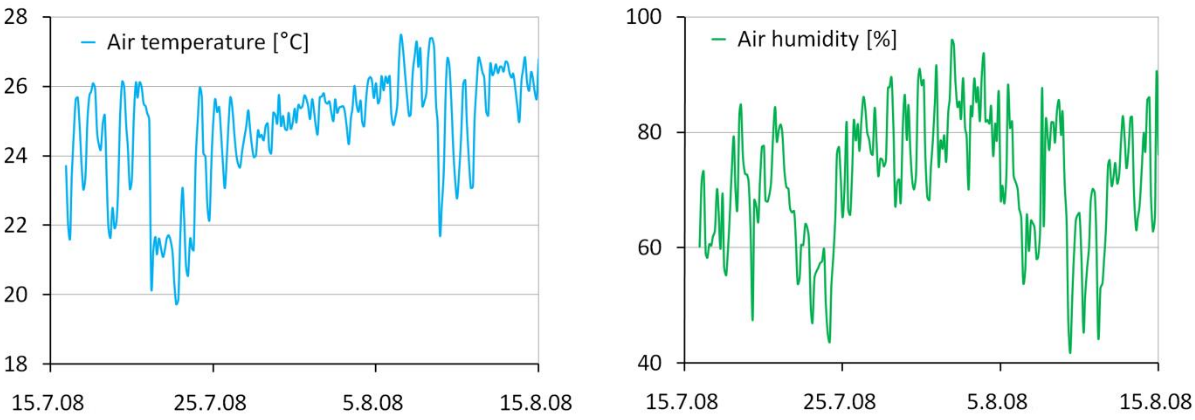

Fig. 6 Time series of air temperature and air humidity (2 m above sea level) with a three hour temporal resolution (extracted from the Aladin-HR model results)

The dispersion coefficients (Prandtl's number) for the scalar *T* and *S* fields were defined with a proportionality factor 0.09 in the vertical, and 0.85 in the horizontal direction with respect to scaled eddy viscosity. The proportionality factors for the dispersion coefficients of turbulent kinetic energy *(TKE)* and dissipation (ε) were used with the values 1 for the *TKE* and 1.3 for ε in the horizontal and vertical directions. Roughness and Smagorinsky coefficients were set as spatially and temporally constant values of 0.01 and 0.2, respectively. The value of 0.00123 was used for the wind friction coefficient. The relations for global radiation and insolation were defined according to Ångström's law. The correlation coefficients *a* and *b* in Ångström's law were defined according to the global mean radiation per decade in the period 1981 - 2000: in this case, the constants for July were a = 0.25 and b = 0.52. A wind constant of 0.5 and an evaporation coefficient of 0.9 were used in Dalton's law. The heat flux absorption profile in the short-wave radiation is described by a modified version of Beer's law. The values used were 0.2 for the energy absorption coefficient in the surface layer and 0.092 for the light decay coefficient in the vertical direction.

3 RESULTS

Figs. 7 and 8 show the hourly averaged current fields for depths of 1 m and 10 m in situations with high Mirna discharges on 3/11/2007 (Fig. 7) and low Mirna discharges on 2/2/2008 (Fig. 8). Fig. 9 shows a time series of hourly averaged S values at the Mirna river mouth at a 1 m depth for one part of the simulation period (i.e. February 2008[2020] and March2008[2020]) obtained with the coarse discretization model (Δx = Δy = 40 m; Δz = 1 m). The shown temporal dynamic of S is used as a boundary condition at the open boundary of the saline wedge intrusion model using fine spatial discretization (Δx = Δy = 10 m; Δz = 0.2 m). Fig. 10 shows a comparison of the salinity fields in the vertical cross section averaged throughout the simulation periods of 2007/2008 and 2019/2020. Fig. 11 shows the salinity time series at the cross sections 1000 m, 3000 m, and 5000 m upstream of the river mouth for the model bottom layer.

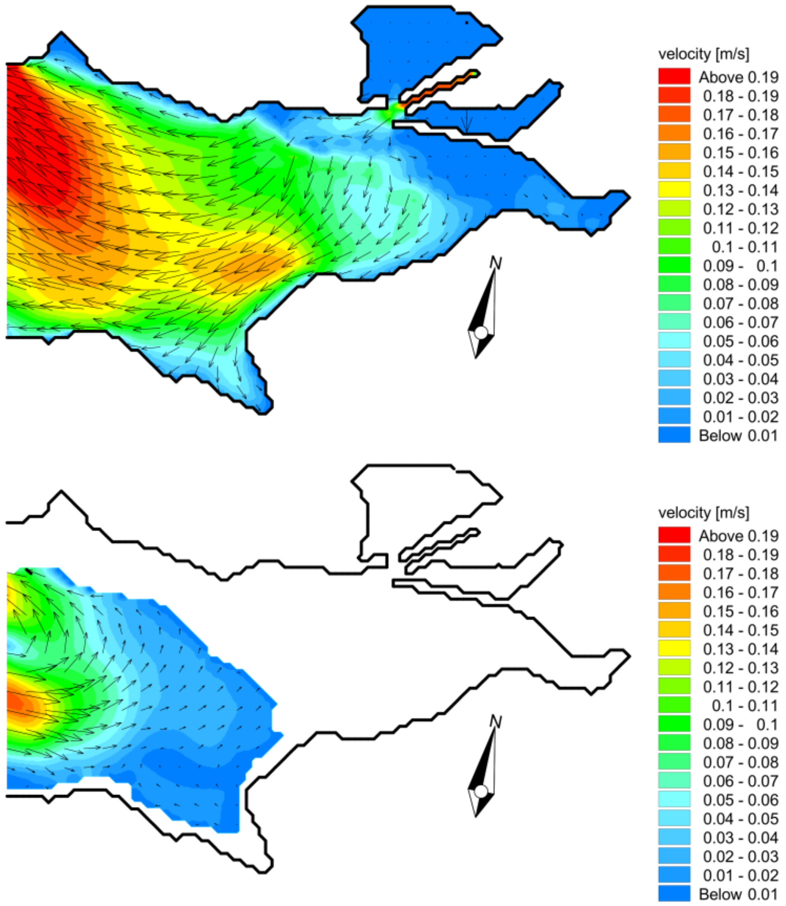

Fig. 7 Hourly averaged current fields for depths of 1m (above) and 10m (below) during high Mirna discharge (3/11/2007)

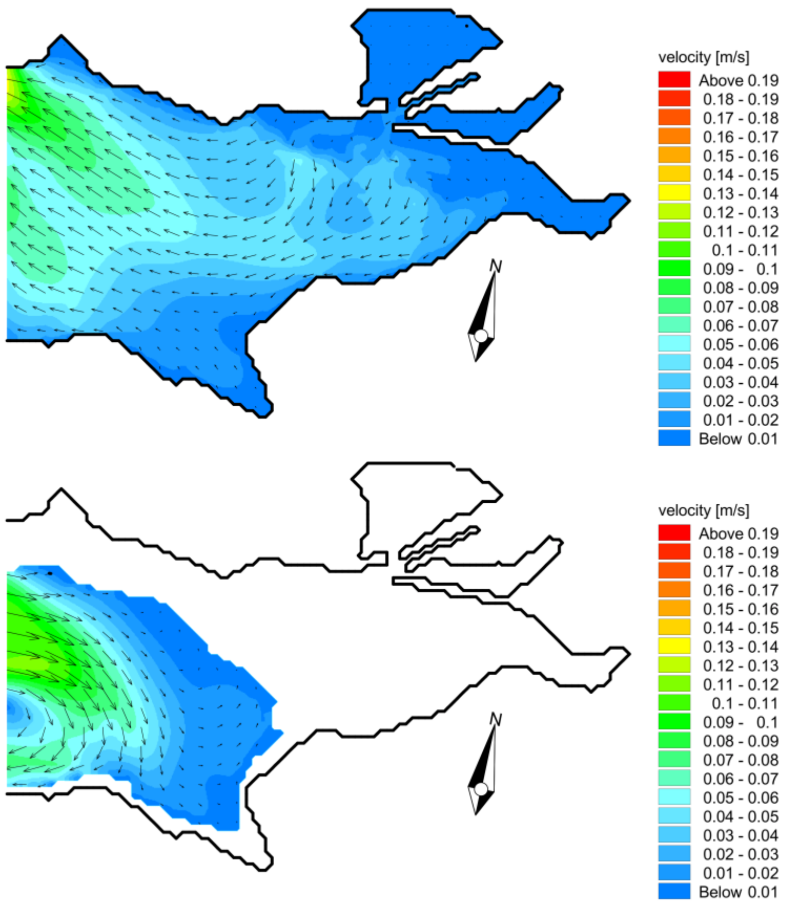

Fig. 8 Hourly averaged current fields for depths of 1m (above) and 10m (below) during low Mirna discharge (2/2/2008)

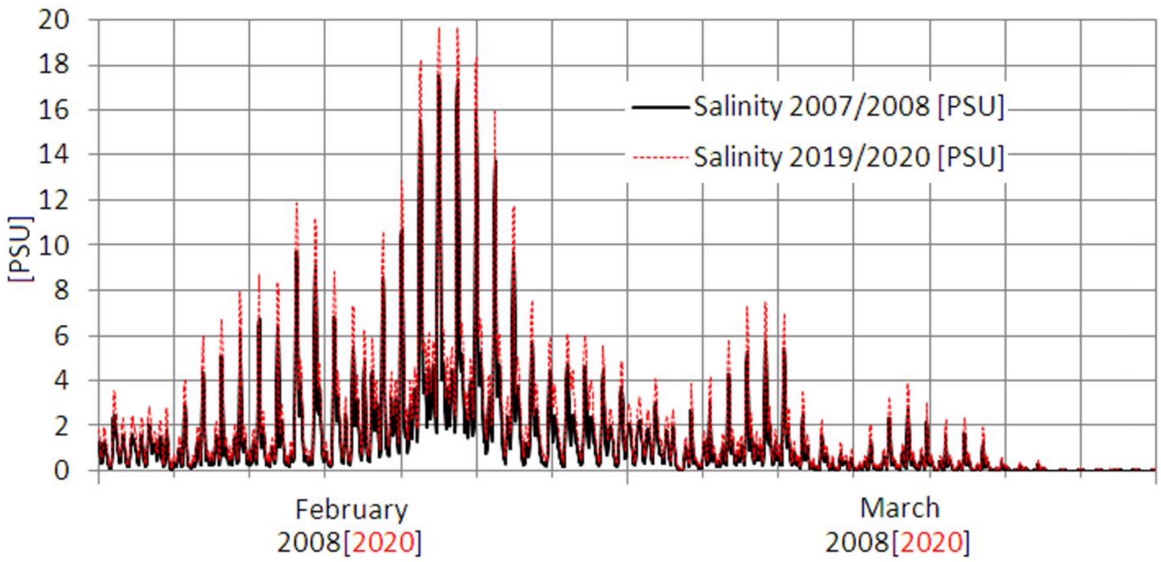

Fig. 9 Time series of hourly averaged salinity at the Mirna river mouth during the 1/2/2008[2020] - 31/3/2008[2020] period, used as boundary condition for model domain with finer spatial resolution

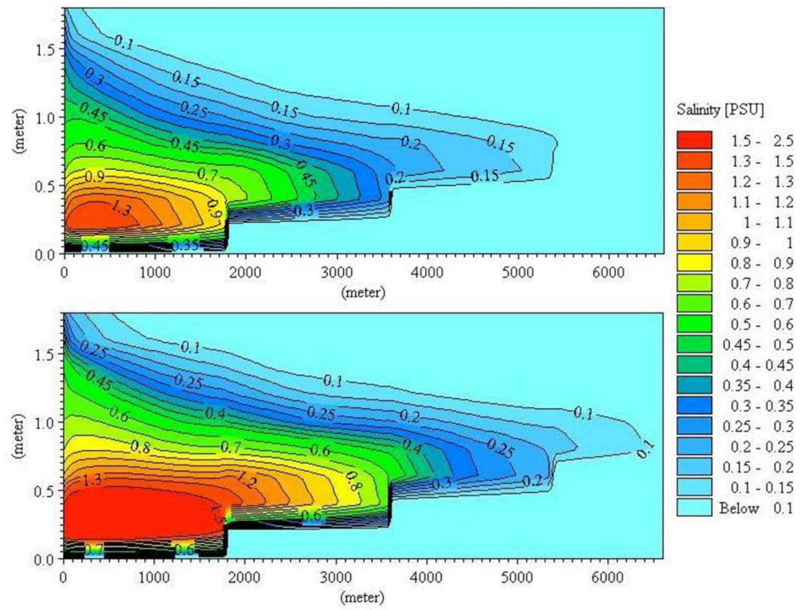

Fig. 10 Mean vertical salinity distributions in the River Mirna channel during the simulation periods 2007/2008 (above) and 2019/2020 (below)

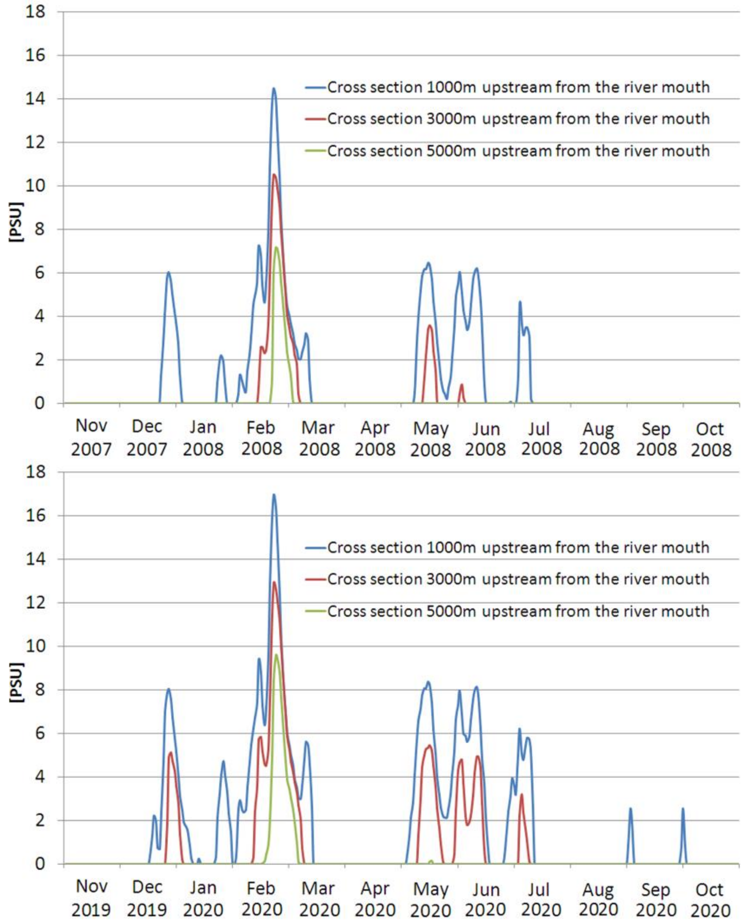

Fig. 11 Comparison of salinity (S) time variations in the cross sections 1000 m, 3000 m, and 5000 m upstream of the river mouth in the near-bottom layer during the simulation periods 2007/2008 (above) and 2019/2020 (below)

4 DISCUSSION

The current fields (Figs. 7 and 8) show that total water movement in Novigrad Bay is highly influenced by the Mirna discharge. Sea surface velocity vectors have practically the same directions but different sizes regarding the value of the Mirna discharge. The vertical compensation flow is registered between layers at depths of 1 m and 10 m (Figs. 7 and 8). Fig. 9 shows that the salinity values at the Mirna river mouth gained with the coarse discretization model for the simulation period 2007/2008 are, as expected, lower than those for the 2019/2020 simulation period. Mean salinity at the river mouth in the 2019/2020 simulation period is about 40% higher than the mean salinity in the 2007/2008 simulation period. Maximum intrusion of the saline wedge front with a mean value of 0.1 PSU in the River Mirna channel is about 5.5 km upstream of the river mouth in the conditions of 2007/2008. In the expected conditions of 2019/2020, maximum intrusion is increased by about 1 km (Fig. 10). The intrusion of the saline wedge through the control cross sections is recorded more often and with higher salinity values during the 2019/2020 period than in the 2007/2008 period (Fig. 11). The same conclusion is found in the recent literature referred to in the introduction.

5 CONCLUSION

The impact of ongoing climate changes on salt water intrusion in the River Mirna channel has been analyzed. A statistical analysis of the measured data shows a trend of rise in sea level and decrease in river discharges in the North Adriatic area. Two 3D finite difference hydrodynamic models were implemented in the North Adriatic area and also in the segment of the River Mirna ranging from the river mouth to the cross section 9000 m upstream of the river mouth. One of the models analyses the November 2007 – November 2008 period and the other one simulates November 2019 – November 2020 period. Only the sea level and River Mirna discharge values are changed using linear extrapolation based on statistical analysis of the collected data sets. All the other values such as sea salinity and temperature, wind and current fields have the same value in both simulation periods. The goal of this paper has been to show the influence of the rise in sea level and the decrease of river discharge to salt water intrusion since they are recognized as key factors in causing this problem. Modelling results show that, in the future, expected changes in the sea level and river discharge will cause a significant increase in saline wedge intrusion in the River Mirna channel. Intrusion will be increased both in the frequency of its occurrence and in its maximum reach. The expected increased salinization of fresh water aquifers and of the soil has negative effects that will be manifested in the shortage of drinking water and in a reduction of agricultural production. It is very important that these problems are anticipated and quantified. Only then can adequate measures for solving them or mitigating their effect be implemented.

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ANALYSIS OF THE RIVER DRAVA AT GAUGING STATION BELIŠĆE

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Abstract

The paper presents results of analysis of the Drava River low-flows from the gauging station at the Belišće [located at 53+800 of Drava's rkm] in period 1962 to 2012. Analysis of stochastic processes of stream low flows presented by the paper takes into account all essential components of the aforementioned process like deficit, duration, time of low-flows appearance, in addition to number, maximum deficit and maximum duration of low-flows in the determined time interval. Considering the river low-flow is of great importance when water quality and receiving ability of streams are discussed, in the paper also has been analyzed the quality of the Drava River at Belišće in periods of low-flow, respectively drought periods.

Keywords

discharge, Drava River, low-stream flow, truncation level, water quality

1 INTRODUCTION

The Drava River springs in South Tyrol close to Dobiaca (Italy) and runs, for almost 750 km, through five countries (Italy, Austria, Slovenia, Croatia and Hungary). It enters Croatia at the Slovenian border near Ormož, 325 km away from its well. Seventy-five km downstream the Mura River flows into Drava and, from that point till Donji Miholjac, it forms the Croatian-Hungarian border line. After flowing through Croatia for 305 km it flows into the Danube near Aljmaš. Its watershed (over 40 000 square kilometres) constitutes about 5% of Danube's whole catchment area.

River Drava has a pluvio - glacial hydrological regime (uniform discharge throughout the whole year, but maximal in the summer - May, and minimal in the winter - January, February).

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Drava River is a source of water for many purposes, but it (as and its tributaries) is also a natural waste water recipient. Simultaneously, Drava and its tributaries are getting a great deal of polluted farmland waters.

Knowledge of the stramflow and low streamflow characteristics is necessary for adequate planning and design of water use and control schemes such as water supply, irrigation control works and licensing of effluent discharges, and for environmental assessment. Increased environmental awareness has led to increased demands for better protection of water resources. The characteristic of low-flow is to have the quantities of water in streamflow i.e. in recipient the minimum and thereby the possibilities for dilution of pollution brought in are also the minimum. Low-flow regulation is conditioned by the precipitation distribution under the year, actually it is conditioned by drought forming while streamflows are being recharged from groundwater. [1]

The stochastic low-flow analysis of the River Drava at Belišće gauging station [53+800 of Drava's rkm] is given in this article. The probabilities of minimum flow phenomenon of different durations of low-flow periods are analysed as also the stochastic low-flow processes where the method takes all the important components of the above mentioned process into account: deficit, duration and time of low-flow phenomenon and also the highest deficit and the highest low-flow duration at the given time unit.

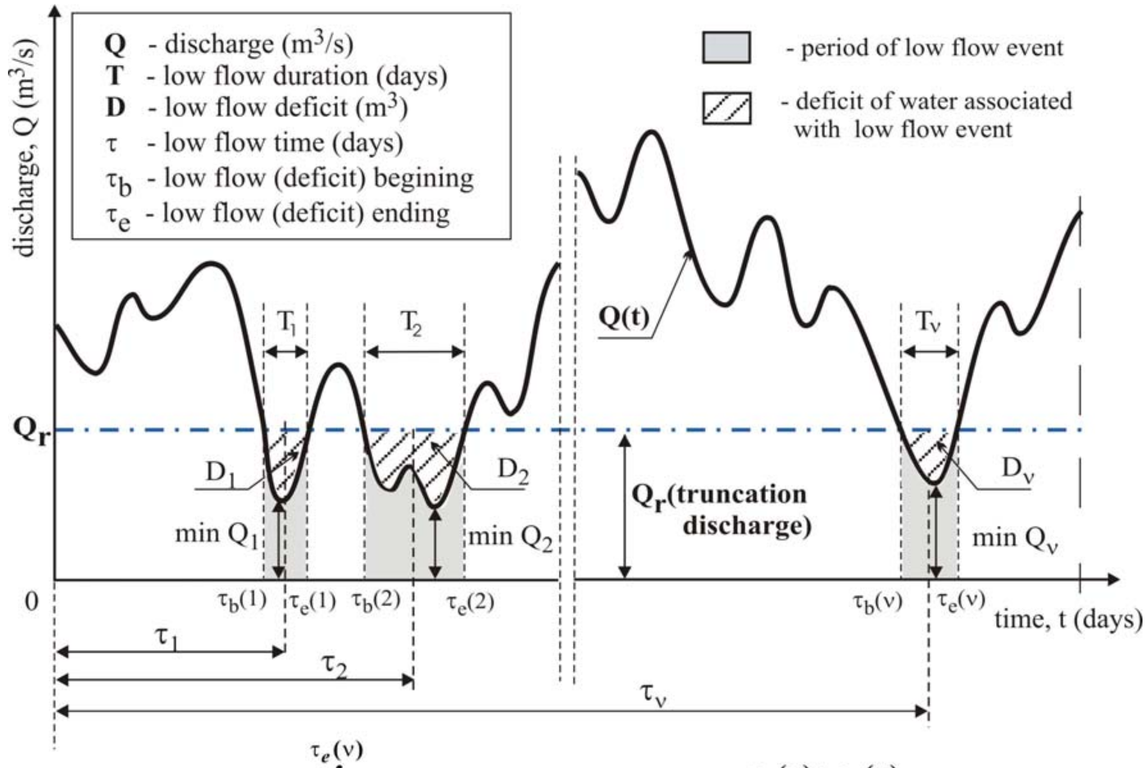
2 APPLIED MODEL AND APPROXIMATIONS

The truncation method is nowadays applied in drought and low-stramflow analysis. Taking this into consideration, it is possible to accept the definition of drought, or low-flow as a period when the analysed characteristic is either equal to or lower than the truncation level of drought.[2] In this paper also applied this method. The stochastic low-flow process analysis is given according to the method which for the chosen location on the river provides the approximate solution for the low-flow calculation as two-dimensional random variable (D_i, T_i), while analysing only the flows equal to or less than the chosen truncation discharge $Q<(Q_r=Q_{95\%})$. (Fig. 1) The obtained solution hasn't been achieved by determining the function of distribution of the above mentioned two-dimensional variable but by testing the approximate equality of phenomenon probability of its deficit and duration as also by the detailed stochastic analysis of all important components of the mentioned process. The low-flow phenomenon, duration and deficit are "estimated" in relation to the beforehand accepted truncation discharge Q_r.

The method treats low-flow as random number of random variables in the time unit [0,t], where both the number of low-flow η (t) appeared under some referential value Q_r in the given time unit [0,t], and the magnitude of their duration are random variables.

The method also enables the defining of computational i.e. artificial 10-, 50- and 100-year low-flows.

For the truncation discharge Q_r, by which the original hydrograph is traversed, the flow of 95 % of low-flow is accepted from the flow duration curve of the mean daily flows - so on the lower extremes zone are approximately separated from flows which belong to the broader mean values zone and the flood flow zone. The attention is concentrated only to the partial duration series so-called the deficit developed by the adequate low-flow phenomenon.



$$D_\nu = Q_t T_\nu - \int_{\tau_b(\nu)}^{\tau_e(\nu)} Q_t dt \quad \text{za } Q_t < Q_r \qquad \tau(\nu) = \frac{\tau_b(\nu) + \tau_e(\nu)}{2}$$

Fig. 1 Main parameters of low-flow event series - hydrograph of instantaneous river discharge on the assorted location in the time unit [0,t] [4]

The theoretical consideration of this method were given by Zelenhasić (1986) [3].
Some simplifications have been performed as to the method application:
- all the observed very small deficits D_i, i=1,2,... which satisfy the condition D_i<0,005 max D_{rec} are neglected as insignificant and are not taken into consideration in this analysis. On one hand they are insignificant compared to the deficit of mean value or rare high deficits;
- the other hypothesis is referred to those deficits which are kept in the analysis i.e. to those higher than the mentioned limits - when the flow Q_t of the examined river and its location surpasses the truncation Q_r during a very short time unit $\Delta T_{\nu,\ \nu+1}$ and thus it artificially separates practically one low-flow into two parts i.e. events. If it is the case of short time unit $\Delta T_{\nu,\ \nu+1}$, in relation to time T_ν respectively $T_{\nu+1}$ and if the volumes $\Delta T_{\nu,\ \nu+1}$ are small in relation to the deficit D_ν or in relation to the deficit $D_{\nu+1,}$ than the events E_ν, and $E_{\nu+1}$ are practically referred to one low-flow. $\Delta T_{\nu,\ \nu+1} = 7$ days is accepted as the time-limit. If the time $\Delta T_{\nu,\ \nu+1}$ is high in relation to the time T_ν namely $T_{\nu+1}$ then the deficit D_ν and $D_{\nu+1}$ are considered reciprocally independent.
- when the time unit [0,t] is taken as the whole calendar year, some low-flow can happen to start in one and finish next year. If it is the case, the time for this low-flow is calculated by using the formula $\tau = \frac{1}{2}(\tau_b + \tau_e)$ This low-flow is not divided into two years as an event but its whole deficit and its whole duration are put into one year, that one of time τ.

3 LOW-FLOW STOCHASTIC ANALYSIS OF THE RIVER DRAVA NEAR BELIŠĆE

3.1. BASIC DATA

The low-flow stochastic analysis of the River Drava near Belišće was made on the basis of the sample of mean daily flows from the beginning of the gauging station work i.e. 1962. to 2012. The data for the year 1992 are not completed (no water level data) because of the war and hence they haven't been taken into consideration. The discharge date over the period 1993-2002 are estimated from flow curves for prior / subsequent period.

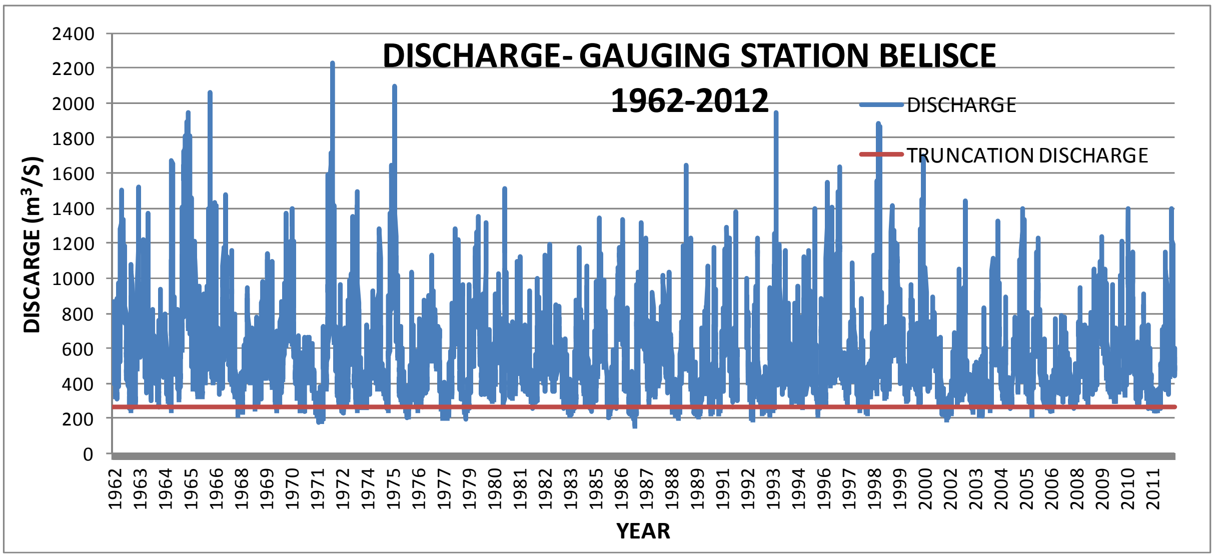


Fig. 2 Disharges at gauging station Belišće in period 1962-2012

The mean annual flow of sample is Q_{sr} = 540 m^3/s, the maximum observed flow is Q_{max} = 2232 m^3/s, and minimum flow is Q_{min}= 163 m^3/s.
The value of Qr = 264 m^3/s is taken as the truncation discharge which represents the flow of 95 % of low-flow from the flow duration curve of mean daily flows. Figure 2 shows daily discharge over the period 1962-2012 and 95% truncation discharge. 61 low-flows have been observed during 51 years from these hydrological data. There has been a period of 22 years without low-flows.
The auto-correlation analysis has been performed for the 61 observed deficits to get the total given sample. The auto-correlation coefficient of the first series indicates that the linear dependence of veneer production in successive periods (interval, lagging, lag of one deficit) has been positive according to the direction and the intensity has been of very inferior quality which is $\rho(0) = \mathbf{-0{,}032735}$**.** The other auto-correlation coefficients are equal to zero. This leads to a conclusion that the deficit phenomena are independent and non-correlated.

3.2. LOW-FLOW NUMBER DISTRIBUTION

The one-year variation of low-flow phenomenon of the Drava River at Belišće gauging station according to seasons is presented in Figure 3. It can be seen that only in May the low-flow hasn't been observed. The low-flow most often appears during the winter months from December to February and hardly ever during May, June and July.

According to [3] and [5], the expected number of low-flows in the time unit [0,t] represents the time-dependent Poisson process (this distribution we assume for partial duration series approach, and the peaks-over-a-truncation approach). Poisson's distribution of low-flow number for the interval of 365 days has been calculated and also the test of goodness-of-fit of theoretical distribution to observed has been performed. Poisson's function distribution parameter has been evaluated through statistics and equal to λ_1 = 1,196. Chi-squared test provides satisfactory results at the level of probability of α=5%, thus the theoretical Poisson's distribution can be accepted for the low-flow distribution. (Figure 4.)

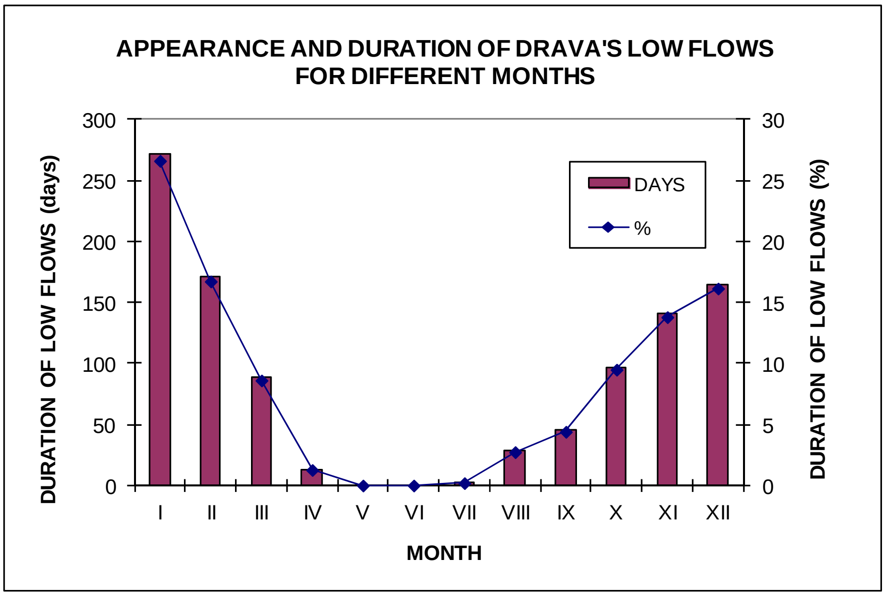


Fig. 3 Monthly variations of appearance and duration of Drava low-flows close to Belišće ($Qr=264\ m^3/s$; 1962-2012)

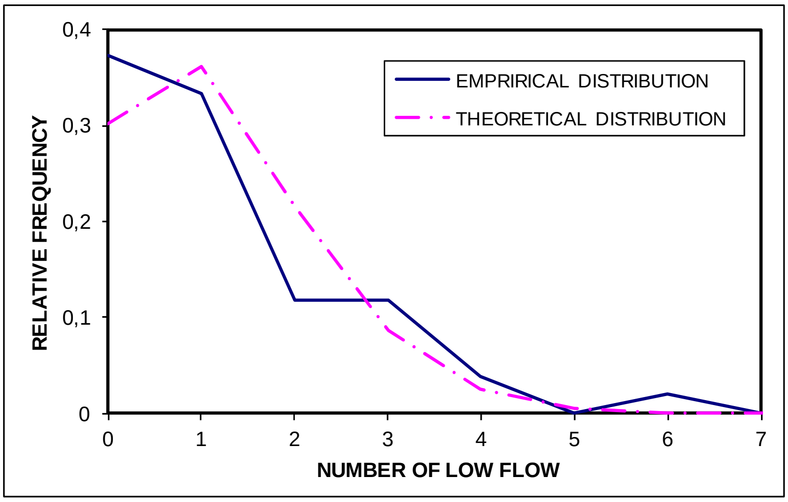


Fig. 4 Empirical and appropriate theoretical (Poisson) distribution of low flows' number in the time interval of 365 days

3.3. DEFICIT DISTRIBUTION

As to determining the deficits distribution, i.e. their magnitudes during the time unit [0,t], the whole year has been taken into consideration. The sample has contained 61 data covering the period of 29 years. Basic statistic characteristics of the obtained deficit sample are:

- MAX DEFICIT 173.923.200 m^3
- MIN DEFICIT 950.400 m^3
- AVERAGE DEFICIT 21.680.917 m^3

It is known that if one event series in time is occurred according to Poisson's probability distribution, then the phenomenon time of k-event from this series follows Gamma distribution. The observed and appropriate theoretical distribution functions are presented in Figure 5. from which it can be seen that the exponential distribution (this distribution is assumed according to [3,5]) is very well adjusted to the observed one i.e. the empirical deficit distribution. The coefficient λ_2 has been obtained through statistic and is equal to λ_2=3,0157 $*10^{-8}$. The theoretical function is indicated by the equation:

$$H(x) = 1 - e^{-3,0157*10^{-8} x}, \text{ for } x \geq 0, \qquad (1)$$

In this equation x represents the deficit and is measured in m^3. The goodness-of-fit test according to Kolmogorov-Smirnov has proved the good coincidence i.e. the closeness of two distributions shown in Figure 5. Figure 6 shows values of 61 streamflow deficits with associated trend which is lightly positive.

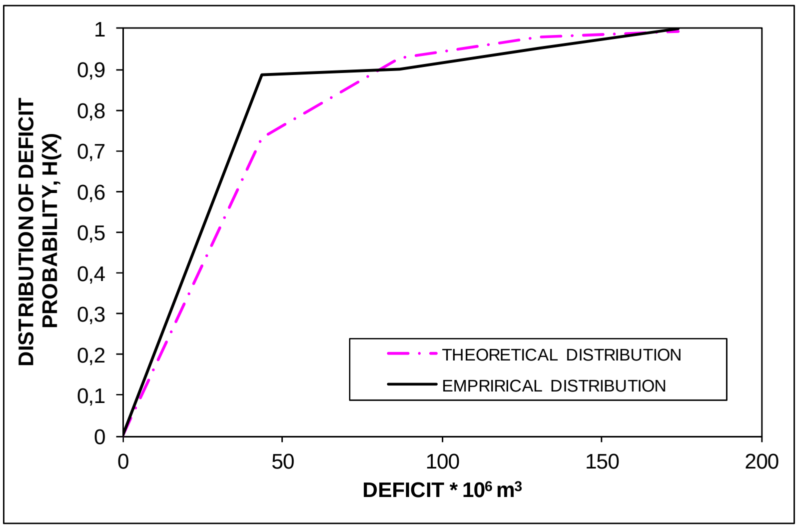


Fig. 5 Theoretical and empirical distribution of deficit (low river flows)

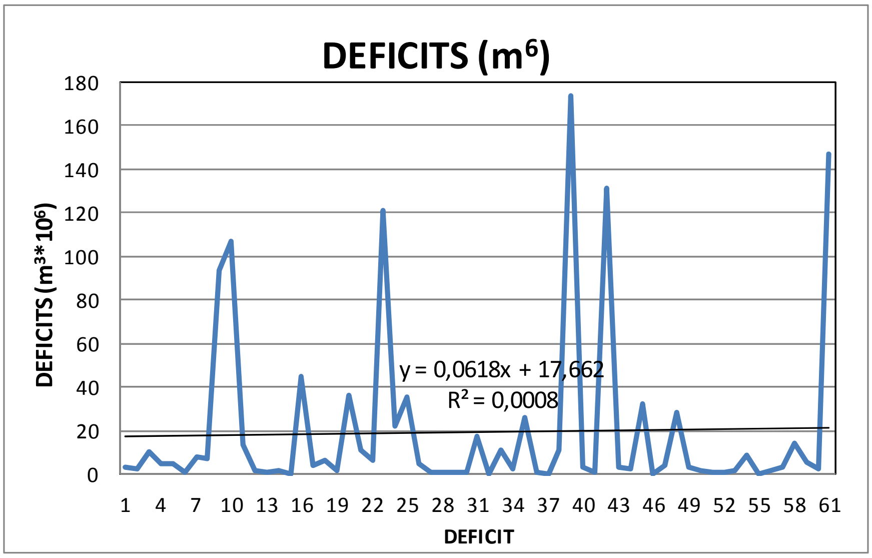


Fig. 6 Values of streamflow deficits (61 deficit) with associated trend

3.4. LOW-FLOW DURATION DISTRIBUTION

A time unit [0,t] of one year is chosen for low-stramflow duration distribution. The sample has contained 61 data about the low-flow duration whose arithmetic mean and standard deviation may be stated as follows:

$\overline{T}$ = 16,75 days,

σ (T) = 20,79 days.

The maximum observed low-flow duration during 51 years of available data of the flow observation is max T_{os}= 100 days (1986).

The observed and suitable theoretical functions of low-flow duration distribution for the river Drava near Belišće are shown in Figure 7. It can be very clearly seen that the exponential distribution function

$$G(u) = 1 - e^{-\lambda 3 u} = 1 - e^{-0,053816\,u} \qquad (4)$$

represents the observed i.e. empirical distribution. In this equation u represents duration and its measured in days. Kolmogorov-Smirnov test for the level of accuracy of α=5% has shown that there is a good coincidence of the two distributions represented in Fig. 7.

Figure 8 shows duration of 61 streamflow deficits with associated trend which is positive.

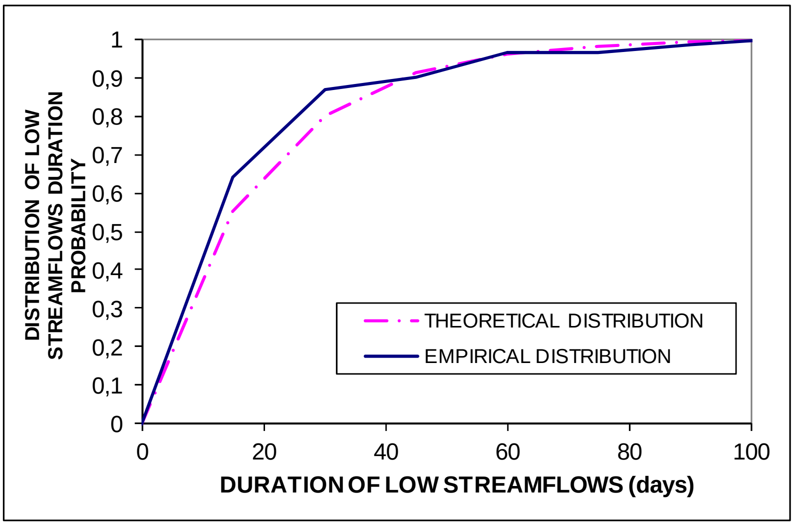


Fig. 7. Theoretical and empirical function of low river flow duration probability

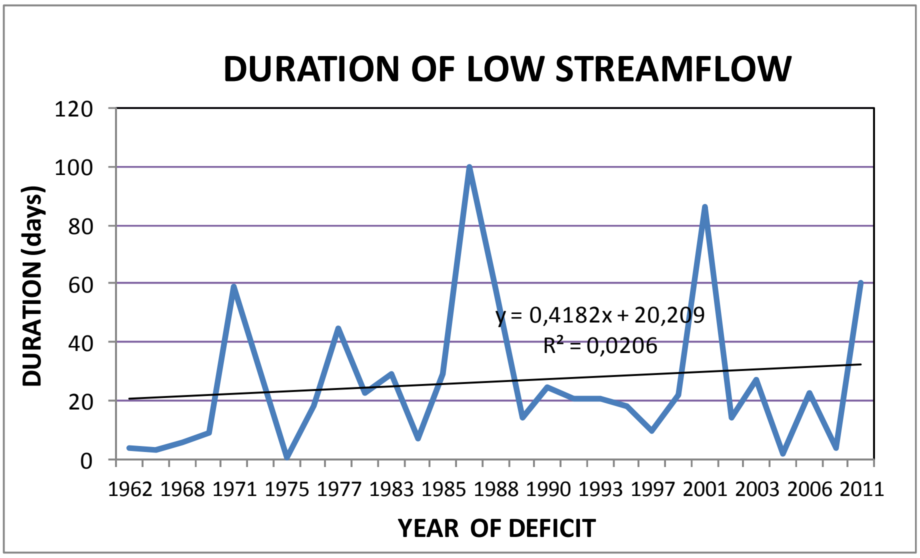


Fig. 8 Duration of 61 - streamflow deficits with associated trend

3.5. DEFICIT RANKS AND LOW-FLOW DURATION

In order to examine the possibility of performing the computational 10-, 20-, 50- or 100-year low streamflow, it has been decided to check the hypothesis about (at least approximate) the equality of phenomenon probability if rare i.e. extreme deficits and low-flows durations for the river Drava near Belišće. Twenty most extreme observed low-flows have been ranked in decreasing series seen Tab. 1, and then the appropriate rank m_D. Since the low-flow as hydrological magnitude is two-dimensional random variable which is characterized by its deficit and duration, in the cases where the condition about probability equality of phenomenon of maximum deficits and low-flow duration is at least approximately satisfactory, it is possible to use deficit magnitudes and duration of the same recurrence interval for determining the computational low-flow of this recurrence interval. [6,4].

The computational deficits as also low-flow durations of 10-, 20-, 50-, 100- and 1000-year recurrence interval have been calculated in equations (3) and (6), on the basis of phenomenon probability of the highest deficit, namely the highest duration, and are presented in Tab. 2.

Tab. 1 Rank of deficits and durations Drava's low-flow for 20 maximum observed deficit

DEFICIT No.	DATE OF LOW STREAMFLOW	DEFICIT D m^3	DURATION T days	RANK OF DEFICIT m_D	RANK OF DURATION m_T
1	05.11.1986.	173.923.200	100	1	1
2	06.12.2001.	146.583.985	86	2	2
3	31.12.1988.	130.896.000	58	3	5
4	18.12.1977.	121.305.600	45	4	7
5	14.12.1971	106.790.400	59	5	4
6	06.10.1971.	93.484.800	57	6	6
7	31.01.1976.	45.100.800	18	7	19
8	13.10.1977.	36.028.800	34	8	8
9	22.11.1978.	35.251.200	23	9	14
10	05.8.2003.	33.350.400	27	10	12
11	03.01.1990.	32.659.200	25	11	13
12	30.12.2011.	31.276.800	60	12	3
13	14.09.1992.	27.993.600	21	13	17
14	15.10.1985.	25.747.200	29	14	10
15	08.02.1978.	21.772.800	18	15	19
16	10.03.2002.	19.326.660	14	16	23
17	22.11.1983.	17.193.600	29	17	10
18	20.09..2003.	14.515.200	16	18	22
19	14.03.1998.	14.439.938	22	19	16
20	01.01.1973.	13.392.000	30	20	9

Tab. 2 Calculated Drava's low-flow and duration for different returning period

RETURNING PERIOD (years)	CALCULATED LOW-FLOW (10^6 m^3)	CALCULATED DURATION OF LOW-FLOW (days)
10	80,9	45
20	104,5	58
50	135,3	76
100	158,5	89
1000	235	132

4 WATER QUALITY

The low-streamflows are especially important when the different aspects of the water quality are discussed. The characteristic of low-flow is to have the quantities of water in stream i.e. in recipient the minimum and thereby the possibilities for dilution of pollution brought in are also the minimum. Figures 9 and 10 show water quality of Drava River at Belišće (Bistrinci) in period of droughts which are the second/tenth in drought ranking. Figures shows that the parameters of water quality - BOD_5 and nitrate (in second – summer drought nitrate are not problematic) are significantly exceed the maximum allowed value. It might be expected. Considering that on the water quality have impact not only discharge already and other parameters, should be carried out a more detailed analysis of correlation between low-streamflow and water quality.

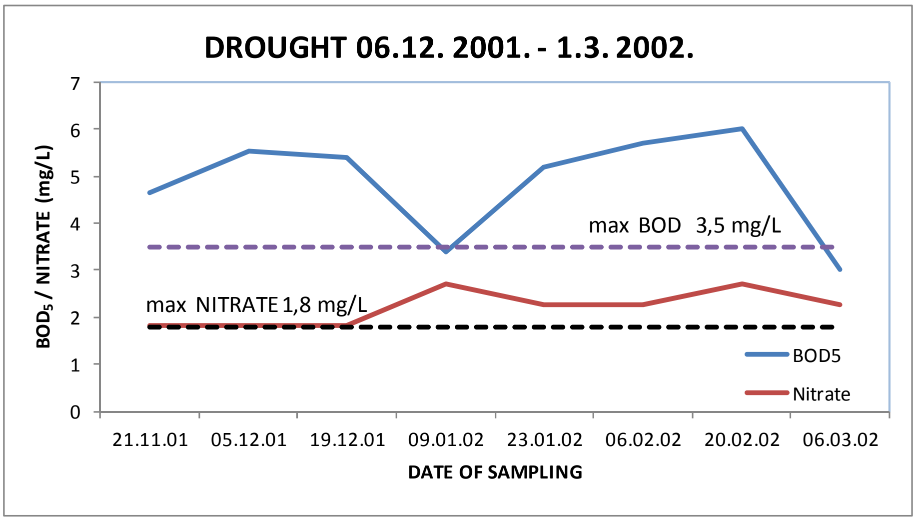


Fig. 9 Parameters of water quality in one winter drought (second in ranking)

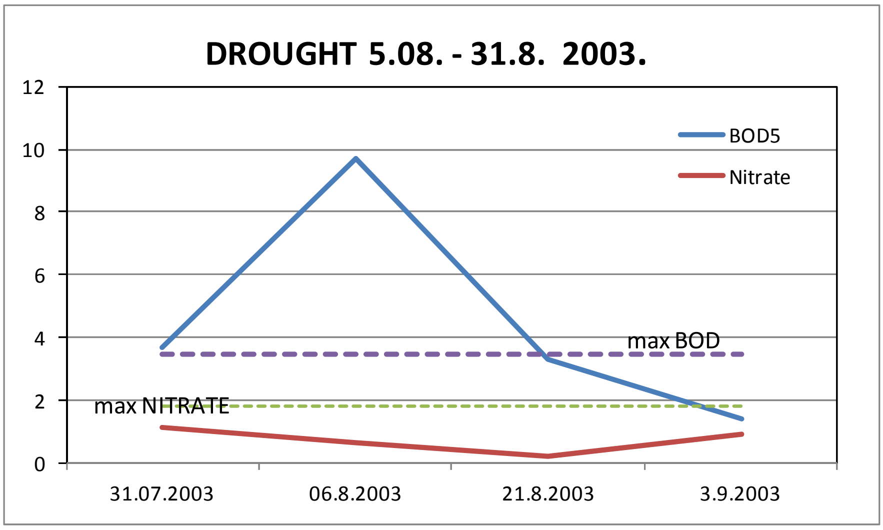


Fig. 10 Parameters of water quality in one summer drought (tenth in ranking)

5 CONCLUSION

Flows at Belišće gauging station from 1962 to 2012 were analysed. The conclusion can be made on the basis of the data:

- mean yearly flow is Q_{sr} = 540 m^3/s, maximum observed flow is Q_{max}=2232 m^3/s, and minimum is Q_{min} = 163 m^3/s;
- only the deficit less than accepted truncation discharge have been stochastically analysed so the attention has been concentrated only to the lower extreme zone, that is to the partial duration series. As to the truncation discharge the flow of 95 % of low-flow has been accepted, from the flow duration curve of the mean daily flows, Q_r =264 m^3/s;
- 61 low-flows have been observed within 29 years;
- there has been a period of 22 years without low-stramflows;
- low-stramflows haven't been observed in May and June whereas in July namely 2 days with low-flows have been recorded;
- days with maximum observed low-flows were in January - a total of 272 days, in December – 165 days, and in February – 171 days;

- droughts occurs less frequently (14 maximum deficit have occurred before more than 20 years), but they have the longer duration (duration trend is positive);
- the highest probability of having only one (k=1) low-flow, i.e.
  P(k=1) = 0,362, afterwards as follows:
  P(k=0) = 0,302
  P(k=2) = 0,216
  P(k=3) = 0,086, etc.
- on the basis of equations (3) and (6), as well as on the basis of Table 1 and the discussion in chapter 3.5., the possibility of defining computational low-flows of any desired recurrence interval can be examined, e.g. 10,20, 50 or 100 years.
- the parameters of water quality - BOD_5 and nitrate (in second-summer drought nitrate are not problematic) significantly exceeds the maximum allowed value that could be expected.

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HOW CAN WE QUANTIFY THE BENEFITS FROM FLOOD WARNING SYSTEMS

Hans Peter Nachtnebel[1]

Abstract

The EU flood risk directive emphasizes non-structural measures, such as flood forecasting and warning systems, to mitigate flood damages and human losses. The objective of this paper is to assess the reliability and the efficiency of early flood warning systems. The reliability expresses the hydrological quality of the forecasts for different lead times. The efficiency evaluates the socio-economic benefits from a forecasting system characterised by the ratio of reduced damages dependent on lead time and the costs of the early warning system. These warning systems are based on several sources of information, like ground observations, meteorological forecasts and hydrological models utilising physical characteristics of the catchment. Obviously, data, models and human behaviour introduce uncertainties and reduce the reliability and the efficiency of the Early Warning System.

Keywords

Hydrological forecasting, reliability, risk assessment, flood risk management, efficiency measures

1 INTRODUCTION

Early warning systems (EWS) are gaining increasing attention and constitute an important element of comprehensive flood risk management strategies [1].

The purpose of EWS is to provide information on expected flows and water levels prior to the actual occurrence of a flood peak and to generate alerts in order to take preventive measures for avoiding damages. A good example refers to two flood events in the Rhine basin which occurred in winter in 1993 and 1995. The reported flood damages in the German part of the basin added up to 615 Mio € 1,2 in 1993 while the respective damages in 1995 were 255 Mio €. Both floods were of similar magnitude, corresponding to about a 100 years flood event, and both occurred both in winter. Although the 1995 flood event exhibited even a larger flood peak in Cologne and downstream the total damages were only about 50 % of the previous

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event [2]. The lower damages are explained by flood protection measures taken between the two floods but more important by the improved flood forecasting and the awareness of the population. For instance, more than 100 accidential oil spills originating from leaking oil heating systems were reported in 1993 while only 6 were listed in 1995.
EWS can be evaluated from a pure hydrological perspective by comparing forecasts with observations by a root mean square error, or by comparing the flood peak with the respective forecasted discharge.
In this paper EWS will be assessed by two important indicators: the reliability of the warning and the damage prevented by the warning. The latter is expressed by the efficiency with respect to the economic benefit of the forecast and costs for implementing and operating such a forecasting system. Obviously, we have to consider the benefits of successful warning as well losses originating from false alerts. The benefits from an EWS are in general the reduced damages and the timely evacuation of people from flood prone areas.

2 RISK ASSESSMENT

This chapter describes the methodological approach for estimation of flood damages and of reduced flood damages due to EWS under consideration of various sources of uncertainties. Based on this information the reliability and efficiency of EWS can be assessed.
Risk is generally defined as a product of hazard and vulnerability. Flood hazard is defined as the probability of potentially damaging flood situations in a given area and within a specified period of time, along with the intensity of the process, the most prevalent indicator for the intensity of a flood being the inundation depth. Vulnerability is composed of two elements, exposure and susceptibility. Whereas exposure analysis answers the question “Who or what will be affected by floods?”, analysis of susceptibility focuses on the question “How will the affected elements be damaged?” [3].
Here, we are interested in the estimating the damages of at least two flood management alternatives. Alternative one refers to a catchment without an EWS while in alternative 2 the benefits of EWS is analysed.

2.1 Hazard Analysis

Hazard analysis is based on flood frequency analysis of time series of gauging stations. An extreme value distribution is fitted to the sub sample, either based on annual maxima or on partial duration series. Then, certain design floods can be estimated. The EU directive states only that frequent events, a flood with a 100 years return period and a rare extreme event should be selected for risk analysis. In Austria, floods with return periods of 30, 100 and 300 years have to be chosen [4].

2.2 Vulnerability Analysis and damage potential

Vulnerability analysis is based on two elements: exposure and susceptibility. The estimation of these two quantities depends on the scale of the analysis. Risk assessment in regional studies [5] can utilise NUTS3 data [6] which provide uniform breakdown of territorial units for the production of regional statistics for the European Union.
Based on inundation and land use maps, which are available together with the delineated APSFR regions [4, 7] by the end of 2013 for EU country, the exposed residential areas can be identified and number of people in the flood plain can be estimated. For a detailed analysis

digital terrain information, land use maps and cadastral information have to be combined with 2-D hydraulic model to identify inundated areas [8].
The number of exposed residents can be estimated on the basis of the residential registry at the district level by estimating the percentage of inundated residential area. The number of active persons per company has additionally to be determined. To avoid double counting of residents the number of commuting people in a village has to be estimated. Usually these figures can be obtained from the municipal office because some local taxes are allocated per employee to the municipality.

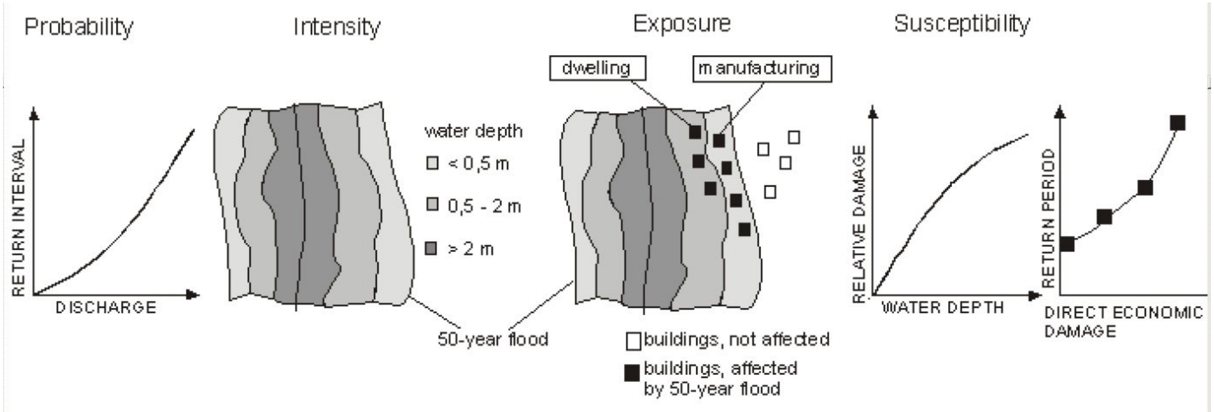

Fig. 1. General structure of risk analysis

The types of damages (Table 1) can be grouped into direct and indirect losses which are either expressed in monetary units (tangible) or by an ordinal scale (intangible), according to [9, 10].

Table 1: Types of flood damages

		MEASUREMENT TANGIBLE	INTANGIBLE
TYPES OF DAMAGES	DIRECT	physical damage to assests buildings contents infrastructure losses in agriculture	losses of life health effects losses of cultural heritage loss of ecological goods
	INDIRECT	production losses traffic losses emergency costs	inconvenience of post-flood recovery increased vulnerability of survivors

The calculation of direct economic damages (right graph in Figure1) makes use of relative flood damage functions that transform the inundation depth at an object into a monetary damage for each object and for each return period. Several existing sets of damage functions are available from various risk assessments [11-16].

Besides the direct damages (Figure 1) of objects business disruption due to inundation is also quite essential. There is an ongoing debate if these damages should be included or not, dependant upon the perspective of the decision maker. With a regional perspective, value added may be included because it is lost to the region whereas on a national level value added may be excluded because it is only shifted to other regions of the nation [6,26]. Secondly business disruption occurs in case of a flood warning when employees of a company stop their productive work and turn to any kind of damage mitigation actions. These costs have to be considerd in the analysis of economic efficiency of EWS. GVA is calculated on the basis of national data. Gross value added per activity is divided by active persons per activity on NUTS2 level [5]. The annual values are broken down to daily values on the assumption that average working days per year.

3 RELIABILITY OF EARLY WARNING SYSTEMS

The predictive uncertainty [17, 18] expressed in terms of a probability distribution function that describes the variation and the expected value of the predicted variable. Todini [18] illustrates how this distribution function can be inferred from the comparison of predicted and observed values using a Bayesian approach. However, practical approaches to determine the distribution function accounting for all sources of predictive uncertainty are still under evaluation [19, 20]. The relative prediction error $\varepsilon_{i,\tau}$ is defined according to (1) by comparing observed runoff $Qobs_i$ with forecasted $Qsim_i$ at lead time τ.

$$\varepsilon_{i,\tau} = \frac{\left|Qsim_{i,\tau} - Qobs_i\right|}{Qobs_i} \qquad (1)$$

Assuming that the errors stem from the same population an overall probability density function (PDF) or a cumulative distribution function (CDF) can be derived. Based on the CDF an estimate of the magnitude of the prediction error can be derived for any confidence probability quantile. In [15] the error at the 85% percentile of the CDF ($\varepsilon_{i,\tau}$ at 85%) was selected to provide an appropriate estimate of forecast reliability. As reliability is usually defined on a scale between zero (unreliable) and one (reliable) it is defined as

$$FR = 1 - \varepsilon_{\iota,\tau}\,(85\%) \qquad (2)$$

This procedure can be repeated for several lead times and as a consequence the reliability function of the forecasts can be established. Obviously, the reliability will decrease with increasing lead time (Figure 2)

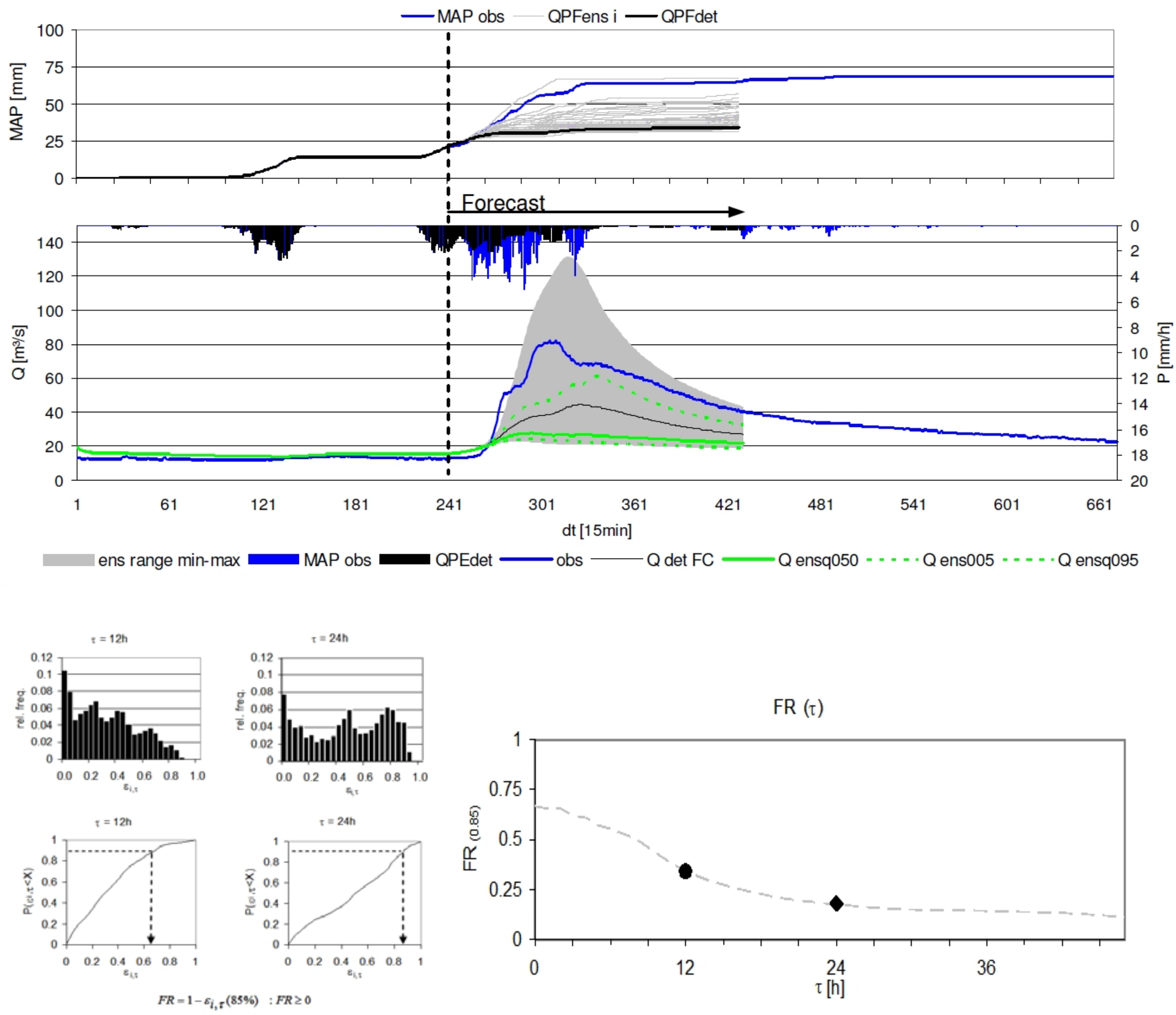

Fig. 2 Ensemble forecast for rainfall and runoff, histograms of relative forecast errors for two different lead times t and reliability function FR(τ) (from [21])

4 EFFICIENCY ANALYSIS OF EARLY WARNING SYSTEMS

The benefits from EWS depend on the reliability of forecasts and the respective lead time. Additionally they depend on the awareness and preparedness of the population and of the responsible managers in companies.

Benefits originate from reduced damages due to temporary flood protection measures, either by mobile walls to protect larger areas or by local flood proving of objects. Further, according to Table 1 the contents of objects could as well as cars etc. could be removed from the flood plain and susceptible people could be transferred to saver places. The mitigation potential can be estimated similar to the damage potential [5]. This figure gives an upper bound for the benefits of EWS that may be reached under optimal conditions, such as precise long term forecasts are at hand and well prepared people will take action. Interviews with local emergency managers will substantiate the estimate of mitigation potential. Additionally, conclusions could be drawn with respect to required lead time for mitigation measures.

The costs of EWS include the implementation of the EWS together with all the communication network to collect data and to release warnings. Additionally, costs for the required input data including the monitoring system have to be considered. Further costs elements are in staff costs and training, software development and maintenance. As already mentioned costs from business disruption have to be also considered when employees take protective measures, either in the company or preparing for the flood event somewhere else. These costs have are especially relevant when a false alert is released.

The total investment in EWS must successfully pass an event-dependent as well as an event-independent evaluation for showing its viability. Usually there is a greater probability for successfully passing the event-dependent evaluation. An event dependant evaluation focuses on the flood event, leaving all issues which do not directly relate to the flood aside. Therefore an event dependant evaluation does not include investments etc. In economic terms, investments are treated as sunk costs, i.a. costs which cannot be recovered by a decision about an alert [5].

Based on this information a graph could be established relating reduced damages and total costs with reliability function. Due to the fact that all elements are subjected to uncertainty a range of benefits can be derived for each lead time (Figure 3)

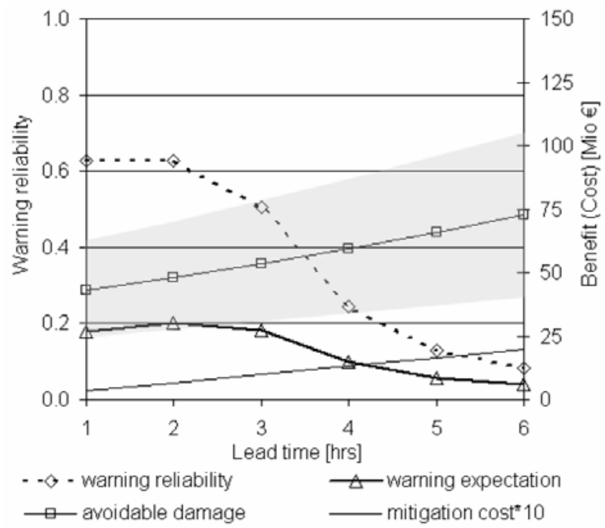

Fig. 3 Range of warning expectations (from [5])

5. APPLICATION TO THE TRAISEN CATCHMENT (AUSTRIA)

5.1 Physical description of the catchment

The Traisen basin is located approx. 50 km West of Vienna and discharges to the Donau river The size of the catchment is 921 km² and elevations range from 200 m in the North to 1800 m a.s.l. in the South [22,23]. Mean annual precipitation ranges from 600 to 1500 mm/a with maximum precipitations in summer mostly in the alpine part. The flow regime is characterized as pluvio-nival. In spring snow driven floods occur often in combination with rainfall. In summer flood events usually are caused by heavy rainfall. In particular the summer flood events are characterized by response times of 8 to 24 hours. However, depending on the spatial characteristics and the location of rainfall within the basin flash-floods may occur especially in the mountainous sub-basins in the southern part.

5.2 Description of implemented early warning system

The operational warning system for the Traisen is based on the continuous hydrological model COSERO [21] incorporates QPF from the INCA-System (Integrated Nowcasting through Comprehensive Analysis [24], which is made available by the Central Institute for Meteorology and Geodynamics (ZAMG). For flood warning additional information is provided by the hydrological services' observation network which consists of ten rain gauges and seven online river gauges. The outcomes of the flood forecasting system are passed on to the Warning Alert Centre (LWZ) which decides about an alert on the basis of predefined warning levels corresponding to thresholds of discharge values.

The data provided for this study comprise the deterministic QPF and ensemble QPF with 50 members for a maximum lead time of τ_{max} = +48 h. The forecasted precipitation fields have a spatial resolution of 10 km², a temporal resolution of 15 minutes and are updated every hour [17]. The warning reliability of the EWS for events, different models, and inputs is displayed in Figure 4.

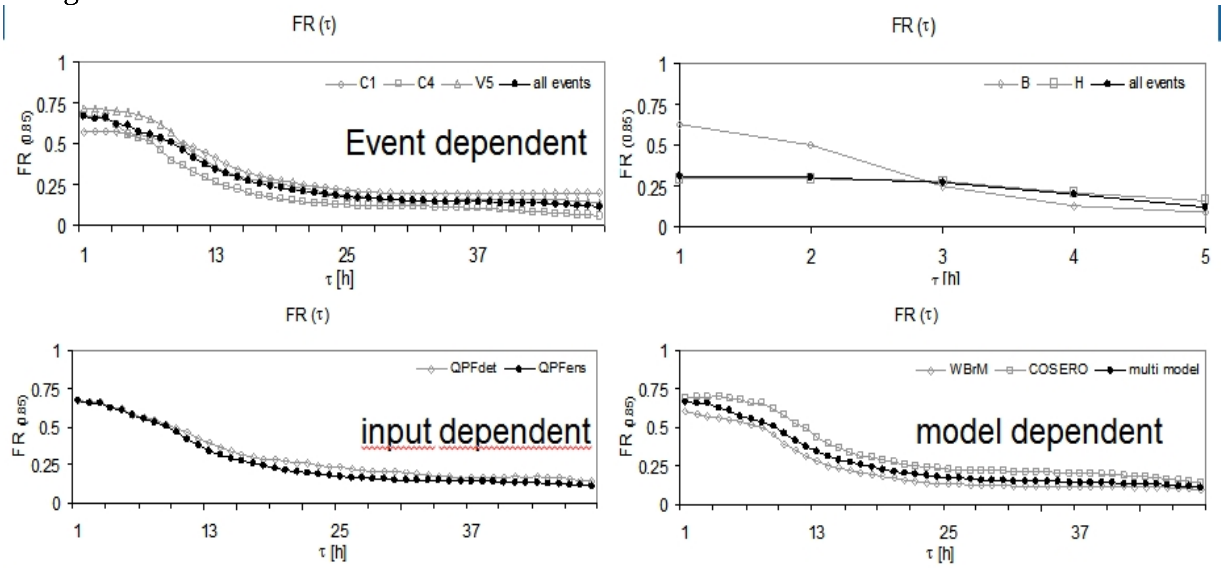

Fig. 4 Reliability of the EWS in the Traisen catchment [from [21])

5.3 Flood risk

The water depths and the floodplains for the Traisen catchment in Lower Austria were taken from the risk zoning system HORA that was completed in 2005. HORA is a joint project of the Austrian Federal Ministry of Agriculture, Forestry, Environment and Water Management and the Austrian insurance sector [25]. The simulation runs were carried out for return periods T=30, 100 and 200 years. The HORA floodplains represent inundations that would occur without any flood protection measures. Inundation depths at the objects within these flood plains were determined on the basis of the HORA results and a digital elevation model.

Especially the lower part of the Traisen catchment is intensively populated and industrialized. A standardised set of damage functions based on [11-16] was established to differentiate among different economic sectors and about ten categories of residential buildings [5] distinguished by parameters such as type of building, age and basement use.

Table 2 Potential damage and risk in the study basins [5]

Return period (years)	30	100	200
Damage (mn €)	563	877	1017

The expected risk has been estimated at 25,89 mn€/a. Obviously, these figures are quite high. The main reason for the high Austrian risk is the neglect of the existing structural flood protection measures in the HORA study. Another factor could be in the derived damage functions.
The largest share of damage potential comes from manufacturing industry with 56%, as calculated with the ICPR-functions. On the second place private dwelling is found with 33%. The third place goes to trade with shares of 5%. These three activities account for more than 90% of the risk, leaving very small shares to the other activities in the basins.

5.4 Mitigation potential

To estimate the mitigation potential originating from an EWS a questionnaire was distributed among the 20 companies from which 8 responded. Additionally the number of active persons was obtained from additional 22 companies. According to [26] this indicator could be used as indicators of value if a reliable relation between capital stock and employees exists. For determining the ability of the companies to effectively react in case of a flood warning, a questionnaire based survey was carried out. The most important question for assessing the benefit of an alert was “supposed you receive an alert some time before a flash flood, how far could you reduce flood damage”? Respondents were asked to tick their estimate on a matrix with rows for a given lead time and columns indicating the damage reduction.
Based on these answers the damage reduction function presented in Figure 5 was fitted. The answers to the questions are presented as black triangles. The size of the triangles is a measure for the frequency of a certain answer, ranging between 4 and 1.

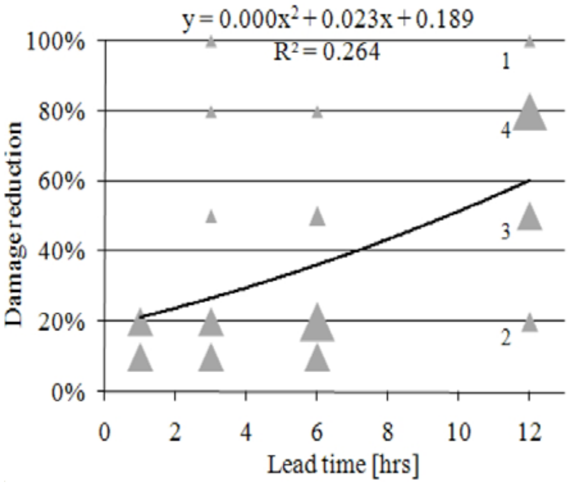

Fig. 5: Damage reduction as a function of lead time.

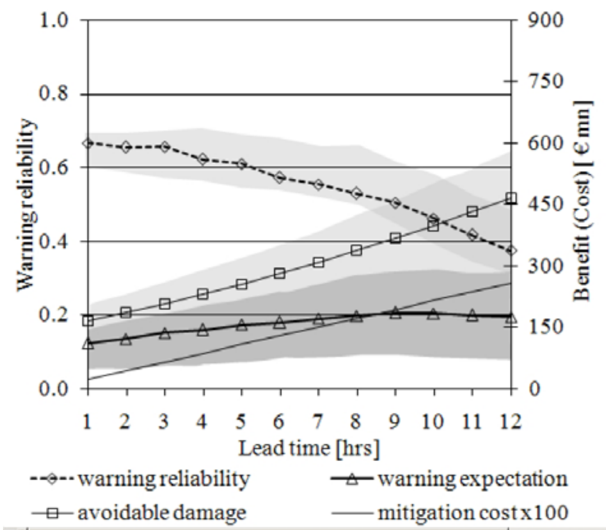

Fig. 6 Warning expectation of the EWS In the Traisen bansin

The damage reduction function allows for calculation of an event-dependent benefit, because the potential damage for the event is known from the risk assessment. The benefit of the warning is the portion of damage avoided as indicated by the respondents.

5.5 Efficiency of EWS

In Figure 8 the benefits of EWS are displayed together with the uncertainties of the elements as a function of lead time. The warning reliability of the EWS decreases with increased lead time while the damage mitigation (benefits) would increase. The expected benefit are obtained by multiplying the reliability (a probability measure) with the avoidable damage (Figure 6). The maximum of the warning expectation could be reached at a lead time of about 9-10 hours. Although the avoidable damage potential is increasing with larger lead times, the large uncertainty (unreliability) in the forecast dominates and forces a decrease of the expected benefits.
Additionally, the mitigation costs are included in Figure 6 as a linear function increasing with time because the people would not be available for production while they are taking counter measures to reduce flood damages.
The result for the Traisen basin is tremendous and should be treated very carefully therefore. Average benefits and costs translate into a benefit cost ratio of 11.70 and a net present value of € 28.62 mn, even at a benefit reduced by 80%. Even an extreme increase of costs by 20% and decrease of benefit by 40% would not force the benefit cost ratio into shiftiness. It still maintains a value of 5.9 which means a net benefit of € 15.6 mn.
It has to be questioned if the used damage functions appropriately reflect the conditions in the study basins.
The grey shaded areas indicate the range of the various uncertainties.

6 SUMMARY AND CONCLUSIONS

Early Warning systems are gaining increasing attention. The objective of this paper was to assess the hydrological reliability and the economic efficiency of such flood risk management tools. An analysis of an EWS system indicates that these tools are economically highly attractive, compared to other flood damage mitigation measures, such as technical systems. Particularly in the economic sectors, which contribute about 70% to the risk, high potential benefits can be realised.
In the light of current knowledge no flood protection strategy appears to offer higher efficiency than the combination of local protection measures and early warning, given high levels of preparedness are maintained in the affected population.

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ESTIMATION OF CRITICAL DEPTH OF GROUNDWATER LEVELS IN DIFFERENT GROWING SEASONS

I. PÁLEŠOVÁ[1] and D. PAVELKOVÁ [2]

Abstract

Accurate estimation of critical depth of groundwater is extremely important for proper management of groundwater systems. Critical depth means the highest depth of ground water that still influence root zone of vegetation. With the decline of this water below a critical depth, the transfer of water from the water table to the root zone of the soil profile stops.

In this paper, we modelled shifts of groundwater level during two growing seasons chosen from the period 1970-2011 in order to estimate the critical depth of groundwater level. We had chosen year 2000 which was defined as an average year in terms of volume of water within 1 meter of the soil and year 2007 which was defined as below-average year. The survey has been made in the conditions of Eastern Lowland in Slovakia.

Keywords

critical depth of groundwater level, root zone of vegetation

1 INTRODUCTION

Due to climate change, issues related to quantifying their possible consequences on the hydrological cycle become the main subject of hydrological research [5]. Hydrological cycle is understood as a rugged unit composed of several interdependent linkages. Between the unsaturated zone and other surrounding subsystems take place different interaction processes, whose intensity depends on hydrophysical characteristics of soils [9]. Groundwater level limits unsaturated zone from the bottom. As a result of interaction processes between the unsaturated zone and groundwater, the level varies in space and time. At high states of groundwater level, the soil profile may be interfered. The effect of these interaction processes may significantly affect water supply in soil and its availability for plant cover [1]. With the

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decline of groundwater level below a certain critical level, the transfer of water from water table to the root zone is interrupted and humidity conditions in the root zone are then affected only by precipitation and evaporation [6].
In the present work we have applied numerical simulation of groundwater level shifts with a mathematical model GLOBAL in two soil profiles differing in soil texture. The aim of this work was to determine the critical groundwater level, when the hydraulic connection and also interactions between groundwater level and root zone is interrupted.

2 MATERIAL AND METHODS

The experimental area of the Research Institute of Agroecology in Michalovce, experimental base Milhostov was chosen for solving of the given issue. This is located in central part of Eastern lowland, about 5 km north of Trebišov (Fig. 1), at an altitude of 101 meters above sea level and is characterized by continental climate. Long-term average air temperature was 8,9°C, during the growing season 16°C. Long-term annual average rainfall was 559 mm, during the growing season 348 mm [4]. The average values of meteorological variables in the years 2000 and 2007 are shown in Tab. 1. Soil type on the experimental site is gley-fluvisol, mainly clayey soil containing particles of I. category in amount of 62,71%. These soils are hardly permeable throughout whole profile and are also harder cultivated due to high percentage of clay fractions.

Fig. 1. Location of experimental site Milhostov within the Slovak Republic

Tab. 1. Average, cumulative, minimal and maximal values of meteorological factors during vegetation periods (April-September) in the years 2000 and 2007

	daily temperature			total precipitation	average wind speed	pressure of water vapor	average moisture (wetness) of air	length of sunlight
	average	maximum	minimum					
Year	[°C]	[°C]	[°C]	[mm]	[$m.s^{-1}$]	[kPa]	[%]	[hour]
2000	**17,3**	**36,5**	**-3**	**417,9**	**2,1**	**1,4**	**70,9**	**1534,6**
2007	**17,9**	**38,2**	**-2,2**	**328,6**	**2,3**	**1,2**	**63,9**	**1566,6**

The impact of groundwater level on water supply in soil was evaluated in two soil types. In heavy soils (occurring directly on the site Milhostov) and medium-heavy soils (modelled data). Hydrophysical properties of these soil types (Tab. 2) which were needed as inputs into the model were determined in the physical laboratory of Department of lowland hydrology IH SAS Michalovce.

Tab. 2. Initial hydrophysical characteristics of soils

Type of soils	sand	silt	clay	theta r	theta s	alpha	n	m	Ks	FWC	TP	WP
	[%]	[%]	[%]	[-]	[-]	[-]	[-]	[-]	[cm/day]			
Medium-heavy soils sandy-loam	**70**	**15**	**15**	0,0517	0,3806	0,0319	1,3957	0,2835	33,77	0,1508	0,0735	0,0330
Heavy soils silty-clay-loam	**18**	**51**	**31**	0,0847	0,4578	0,0079	1,5102	0,3378	12,27	0,2660	0,1115	0,0399

Impact of groundwater level was evaluated in two selected years (2000 and 2007) which were chosen from the period of years 1970-2011. The year 2000 was chosen because of the fact that it was an average year regarding to volume of water supply in root zone from the period of years 1970–2011. The year 2007 was chosen because the value of the water supply in the root zone was the lowest.
Numerical simulation with model Global [3] has been used for examination of the solved issues. Global is one-dimensional numerical model of water movement in the soil, allowing the calculation of the distribution of moisture potential or soil moisture. Model is based on nonlinear partial differential Richards's equation [8]:

$$\frac{\partial h_w}{\partial t} = \frac{1}{c(h_w)}\frac{\partial}{\partial z}\left[k(h_w)\left(\frac{\partial h_w}{\partial z}+1\right)\right] - \frac{S(z,t)}{c(h_w)} \quad (1)$$

$$c(h_w) = \frac{\partial \theta}{\partial h_w} \quad (2)$$

k(hw) unsaturated hydraulic conductivity of soil [cm.s^{-1}],
S(z,t) intensity of water abstraction by plants roots per unit volume of soil per unit time,
c(hw) specific water capacity [cm^{-1}],
θ volumetric soil moisture [cm^3.cm^{-3}],
hw humidity soil potential,
z vertical coordinate [cm],
t time [s].

Numerical simulation was carried out with the daily calculation step. For simulation, homogenous profile up to a depth of 3 meters has been chosen. Water supply in soil influenced by underground water level was evaluated to a depth of 1 meter below the surface. This layer is a crucial part of the root zone of field plants [2].
Hydropedological inputs into the model were determined for each soil type. As a vegetation cover the current crop was used. In 2000 it was clover mixture and in 2007 winter wheat.
Hydrometeorological inputs into a mathematical model and bottom boundary conditions were used from the growing season of the years 2000 and 2007 from Milhostov station.
An average groundwater levels were for the each year moved by 40 cm above long-term average and 40, 60, 80 and 100 cm below its average value in order to determine the depth when the effect of groundwater level on water supply in root zone is negligible.

3 RESULTS AND DISCUSSION

Effect of soil environment

In Fig. 2 it is possible to see a different rate of underground water level influence on the water supply in a different soil types. This means sensitivity of the impact on hydrophysical characteristics of porous environment. Input hydrophysical characteristics of chosen soil types are shown in Tab. 2. High degree of influence of hydrophysical properties of soils on water supply in root zone was demonstrated.

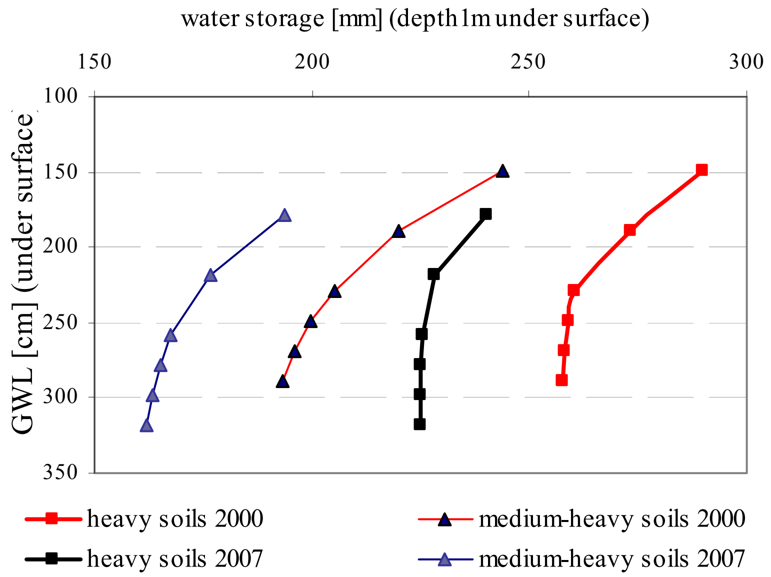

Fig. 2. Dependence of water supply on groundwater level for heavy and medium soils in the years 2000 and 2007

Effect of hydrometeorological characteristics

From the period of years 1970 to 2011, years 2000 and 2007 were chosen. Year 2000 was chosen as an average regarding to water supply in depth up to 1 meter and year 2007 as the year with the lowest water supply. All other inputs (boundary conditions, phenology and soil hydro-physic) remained unchanged. Rainfalls in the growing season during the year 2000 were up to 20% above normal and in 2007 were 6% below normal. Because of changes in meteorological inputs for each year, there is distinct different amount of water supply in depth up to 1 meter with the change in groundwater level (in 2000 there were higher values achieved than in 2007; in both years there were same levels of groundwater level used). It was caused mainly because of changed amount of rainfalls. Dependency curve of water supply and groundwater level did not change fundamentally, which means that meteorological factors did not affect the critical level of groundwater level.

Effect of groundwater level movement to water supply in root zone of soil

Calculations have shown (Tab. 3) that dependency of water supply in root zone on groundwater level is more pronounced in heavy soils than in medium heavy soils. In heavy soils the critical groundwater level occurs in depth around 250 cm below the surface. In medium heavy soils it occurs more than 300 cm below the surface. Decrease of groundwater level below this depth (300 cm) means negligible (0,1 mm WS / 20 cm GWL) impact of groundwater level on supply of root zone by groundwater.

Tab. 3. Values of Δ WS [mm] (water supply up to 1 meter depth) for different groundwater levels in the years 2000 and 2007

	year 2000		year 2007	
	Heavy soils	*Medium-heavy soils*	*Heavy soils*	*Medium-heavy soils*
Δ WS (GWL about 40cm higher)	-	-	-	-
Δ WS (GWL actual)	- 16,60	-23,89	-12,14	-16,64
Δ WS (GWL about 40cm lower)	-12,79	-15,00	-2,62	-9,42
Δ WS (GWL about 60cm lower)	-1,63	-5,46	-0,50	-2,40
Δ WS (GWL about 80cm lower)	-1,03	-3,69	-0,03	-1,78
Δ WS (GWL about 100cm lower)	-0,40	-2,69	-0,15	-1,14

From the results it can be inferred about the use of regulatory effect of groundwater level on water supply in root zone of soil profile [7]. The results will help to easier estimation of water supply based on the location of groundwater level.

4 CONCLUSION

Paper deals with issue of water-table and its influence on water supply in root zone of vegetation (defined by depth of 1 meter). For identification of the objectives, experimental site named Milhostov have been chosen. Two years from interval 1970–2011were chosen from available data set of water supply in root zone of vegetation. Year 2000 was chosen as an average regarding to water supply and year 2007 was chosen as the driest. Influence of water supply on root zone of vegetation was examined on two soil types. One of them was heavy soil (actually occurring on the experimental site) and other one was moderate soil, which hydrophysical properties were modelled. Water-table level was moved compared to the actual state by 40 cm higher and 40, 60, 80 and 100 cm below. It was found that changes of meteorological factors have no significant impact on critical depth of water-table level. But different hydrophysical properties of soil move water-table to different levels. Further, it was found that the critical of water-table level for heavy soils is located approximately at a depth of 250 cm and critical depth for medium soils lie below 300 cm below the surface.

Acknowledgements

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MODELLING THE RISK OF CONTAMINATION OF THE SIHOŤ ISLAND WATER RESOURCE

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Abstract

The aim of the study is to provide qualified and objective basis for quantification of the risks of contamination of the Sihoť Island Water Resource and to ensure its lasting quality protection as one of the most important resources of drinking water supplying not only a large part of Bratislava City but also its surroundings. The study includes the results of fieldwork, sampling, sediment quality analysis, complementary measurements and groundwater modelling. There were identified and quantified potential negative effects on groundwater quality, evaluation of interactions among groundwater, surface water and sediments in relation to pollution, processing documents (levels, regimes, water table contours) for solving the penetration of substances through the unsaturated zone, performance evaluation of contaminants and soil properties, the modelling of penetration of pollutants through the unsaturated zone in "worst case" regime (update the existing hydraulic model in 3D and its transformation and refinement in MODFLOW program, hydraulic calculations for different scenarios of transport and spread of pollutants during different groundwater levels and operational situations (maximal, average, minimal groundwater levels, different amounts of water extractions), modelling the spread of pollution normal conditions and "worst case" in MT3D program. The range of contamination and its impact on the quality of drinking water in the saturated and unsaturated zones was determined after the partial flooding of the island by infiltration either from the whole area or significant points in the most vulnerable areas. The results of transport modelling are the isolines of contamination, time graphs of contamination in wells, evaluation of the hazards of individual wells defining the areas significantly compromising the water quality pumped from wells,

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assessment of sediments quality in the Karloveské rameno River Arm and their impact on groundwater quality.

Keywords
groundwater modelling, MT3D program, Sihoť Island Water Resource, scenarios of transport and spread of pollutants

1 INTRODUCTION

This study contains the results of modeling the spread of various pollutants in groundwater. The aim of the calculations was to accurately quantify the negative impact of leakage of these substances on groundwater pumped from wells in Sihoť Island Water Resource (hereafter Sihoť WR), which is one of the most important sources of drinking water supplying not only a large part of Bratislava, but also its surroundings. Present study provides a comprehensive modeling of the risk of pollution in Sihoť WR, which is based on the operating results of the monitoring carried by BVS (Bratislavská vodárenská spoločnosť, a.s., in English: Bratislava Water Company, Inc.). The evaluation is processed in accordance with the legislation in force and based on all available evidence, which often provided by the client, BVS, and more were added from other existing literary sources, own and public databases, data from various survey works from the archives and data from monitoring of environmental components. Dangerous substances and scenarios of their release were chosen to be contained in them all reasonably possible situations that can have a negative impact on groundwater quality. In solution the intersection of major pollutants through the unsaturated zone is considered the normal conditions and worst real possibility ("worst case"). Possible penetration of pollutants into the water resource wells were assessed by the method of numerical modelling of groundwater flow and transport of substances in the aquifer in the selected area. There was used a 3D model of groundwater flow which has developed in 1997 by NuSi Ltd. (Pospišil & Kovács, 1997 and Pospišil P. et al., 1999). At that time, the model was used to determine the usable groundwater resources in the area of Sihoť WR and to determine the resource protection zones. The hydraulic model from 1997 was modified and supplemented by the transport section so that it is possible to calculate the transport of substances from source places in the whole island and its surrounding to water resource wells. Hydraulic calculations were performed for different scenarios of transport of substances in different water levels and operational situations in Sihoť WR (high, average, low status of ground water levels, average extraction of 472 $l.s^{-1}$ and maximum equal to 868 $l.s^{-1}$). There have been simulated different situations and types of contamination sources through the unsaturated zone, the impact of contamination after the partial flooding of the island surface area by the areal infitration and also spreading through local depressions (dead branches), the extent of contamination in the saturated and unsaturated zone aquifer groundwater flooding the entire Sihoť Island, scope for significant point release contamination in the most vulnerable areas of the Sihoť Island and their impact on the quality of drinking water exploited by extraction objects of water resource. Results of the study are:

1. Assessment of real and potential impacts of various anthropogenic activities and activities, including the planned cycling routes lead through Sihoť WR on the quality of drinking water in Sihoť WR.

2. Processing risk analysis of potential impacts on groundwater quality on the Sihoť Island in terms of potential pollution.
3. Recommendations and guidance documents to ensure the protection of Sihoť WR for its sustainable and safe use.

Investigator of the study was a team of experts from WRI and cooperating company NuSi Ltd., which implemented the model solution.

The study area is the Sihoť Island, which forms VZ Sihoť through which the proposed cycling route in Bratislava City from Karlova Ves Ward to Devín Ward (Fig. 1.1). The Sihoť Island is located on the left bank of Danube River, in the city of Bratislava - Karlova Ves Ward between river 1876-1871 river kilometers.

Fig. 1.1 Study area of the Sihoť Island Water Resource

2 MODELLING OF GROUNDWATER FLOW

The hydraulic model from 1997 (Pospišil & Kovács, 1997) was used for the modeling of groundwater flow in the Sihoť WR. It was made in the program MODFLOW (McDonald & Harbaugh, 1988). It addresses three-dimensional steady-state groundwater flow in detail in the water resource and its surroundings. It was designed for precise knowledge of groundwater flow to the well source at different hydraulic situations. There can be displayed:

- the geometric complexity of the construction watered environment,
- Danube River and the Danube arm as strong boundary condition III. type variable with its incomplete terminations in the bottom sediments and areal extent dependent on the level of the Danube, while the low stocks boundary condition in the shoulder there,
- groundwater extraction from a relatively dense network of wells with limited perforations,
- spatial variability coefficient of filtration.

Fig. 2.1 shows the finite differential computing network hydraulic model in detail water source. Vertical territory is broken down to cover a layer and a layer of gravel. Overburden layer is capped by the terrain, bottom base layer of overburden and is implanted into the model as the first model layer. Under the overburden layer is a layer of gravel, which is bounded from below bedrock. Gravels in the model solutions are divided into four layers (Fig. 2.2).

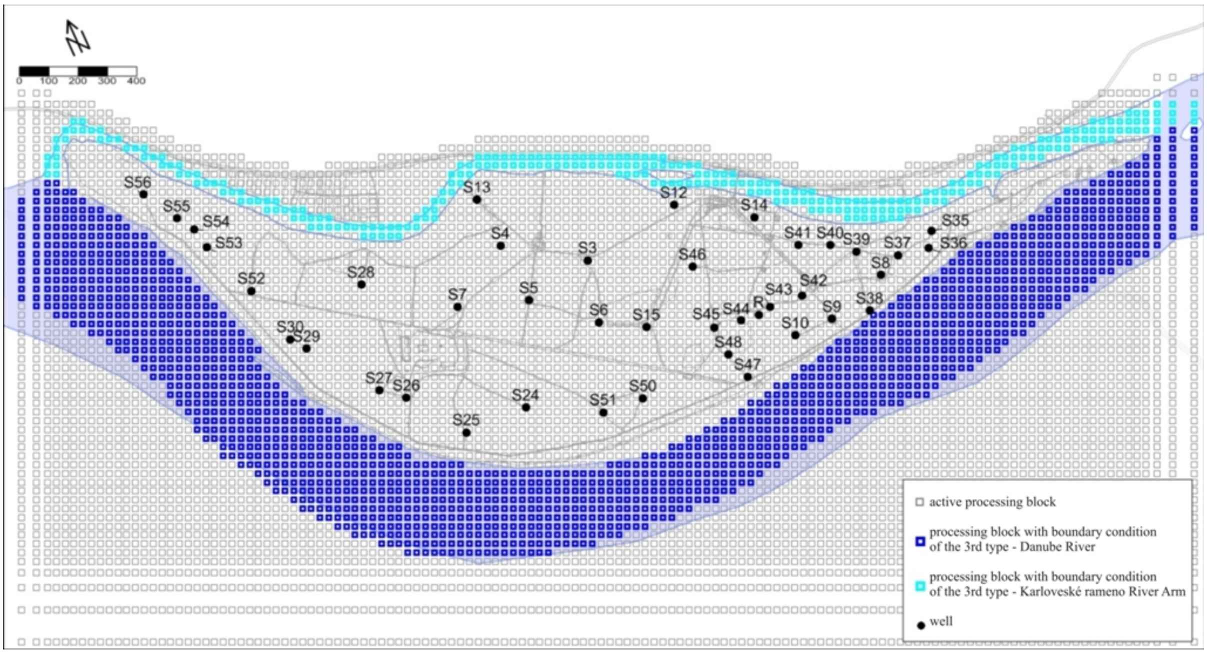

Fig. 2.1 Computing network in the model

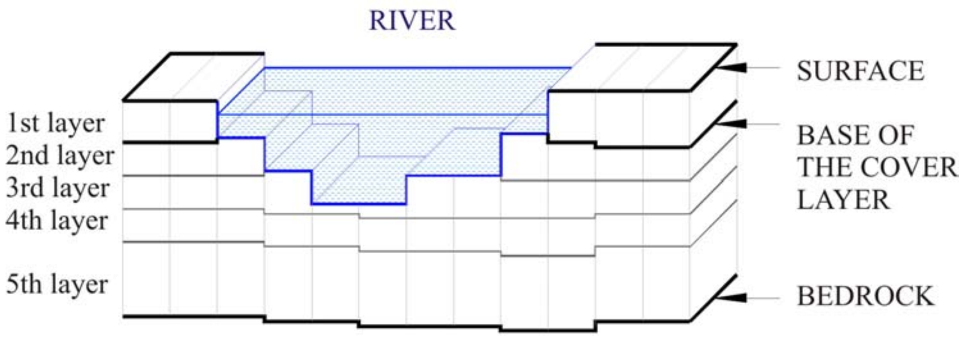

Fig. 2.2 Vertical scheme of the aquifer

Determining influence on groundwater flow in the area of interest is the Danube River. The water level in the Danube River is related to the water level in Karloveské rameno River Arm of the Danube River, which also affects the hydraulic conditions. Both, Danube River and its arm, have incomplete, unstable slaughtering river bed to the sediments, they are expressed as boundary conditions of the 3rd type. The model automatically determines the bank line of the Danube and the extent of flooding river arm by level in the Danube River. Karloveské rameno River Arm is dry only a few days in a year if water level in Devín gauging station is 135.54 m a. s. l. or less. (Pospišil & Kovacs, 1997). This value of water level (average value during hydrological years 1995 - 2010) was used to model the transport of contamination. The transport rate in groundwater is the fastest in this state and it was used for all cases of spread of pollutants leaked either on the Sihoť Island or location near the Danube River. The model shown that if the arm is not dry, and thus performs as a boundary condition of the 3rd type, pollutants leaked behind the arm in the cottage area can not directly penetrate into the island. Thus, arm protects the island before penetration. The worst case situation was therefore simulated during drought, when arm is dry and groundwater flow can penetrate from the cottage area into the island's water supply.

Transport of pollutants was simulated for conditions of groundwater flow during the mean (472 $l.s^{-1}$) and maximum (868 $l.s^{-1}$) water extractions. Fig. 2.3 and 2.4 shows the model hydroisohypses during these extractions. In both cases, the whole island except its marginal parts is in a great depression caused by water pumping from wells. All water inflowing into this area either from the Danube River or from the Karloveské rameno River Arm flows to the wells filters. From a 3D view water flows into the filters, it does not flow away above them. There flows 82 $l.s^{-1}$ of water from the arm to the wells during the mean extraction and 390 $l.s^{-1}$ of water from the Danube River. During the maximum extraction amounts of water are much higher, 252 $l.s^{-1}$ from the arm and 616 $l.s^{-1}$ from the Danube River. It means that the higher extraction the higher ratio of water supply from the arm.

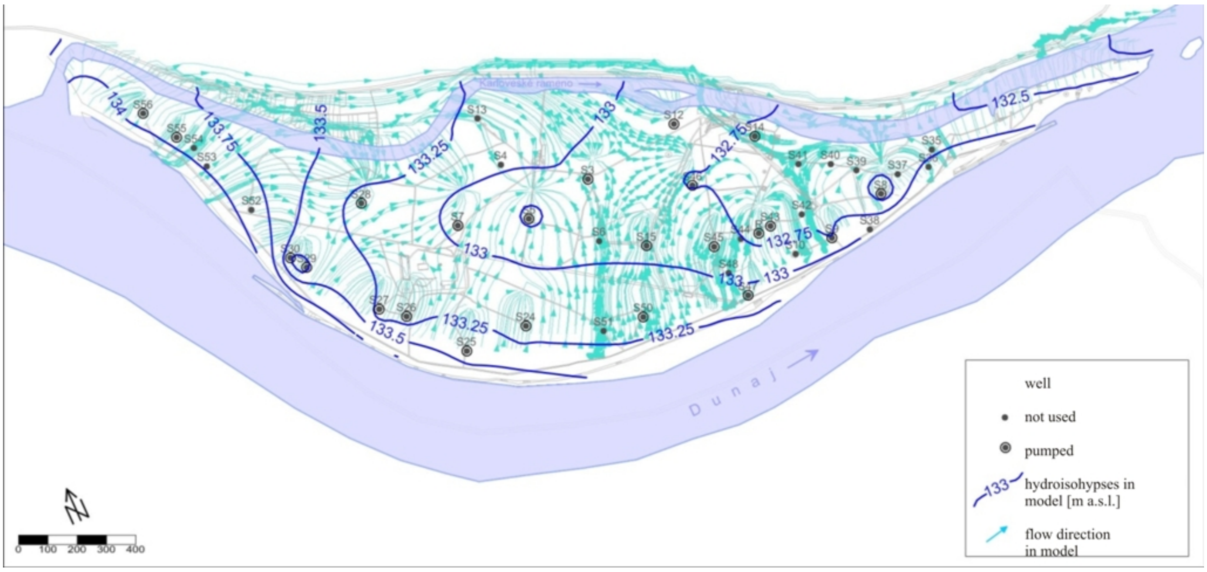

Fig. 2.3 Model streamlines of groundwater during the mean extraction (472 $l.s^{-1}$)

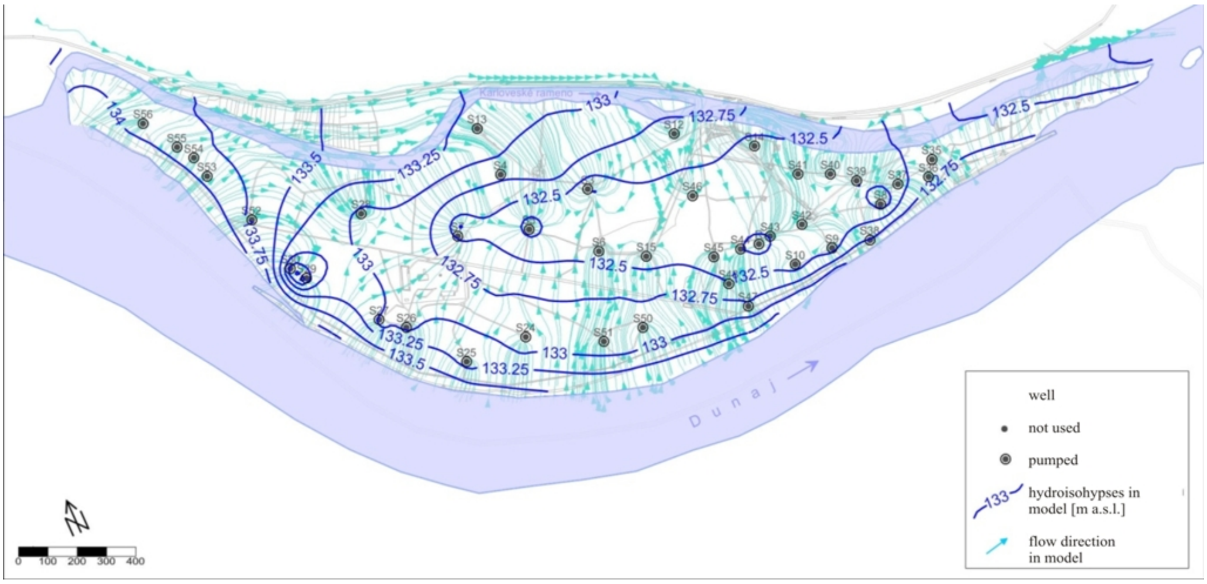

Fig. 2.4 Model streamlines of groundwater during the maximum extraction (868 $l.s^{-1}$)

3 MODELLING OF CONTAMINATION TRANSPORT INTO THE WELLS OF SIHOŤ ISLAND WATER RESOURCE

Some pollution, which would be penetrated from the island and its surroundings to groundwater flow, would be further spread in the aquifer by flowing groundwater. Velocity field of groundwater flow was determined by hydraulic model. Transport of the model for the spread of dissolved contamination was prepared using the MT3D program. Transport ground parameters were determined on the base of changes in water temperature in the ground and also in the Danube River in our previous works (Pospišil & Kovács, 1997). Our obtained values are shown in the Tab. 3.1, they were used in the presented calculations. These values are consistent with the common, rather conservative values for gravel-sand rocks in literature.

Tab. 3.1 Transport parameters in aquifer

parameter	values
Average hydraulic conductivity [m.s^{-1}]	0.0045
Total porosity [-]	0.31
Porous media density [kg.m^{-3}]	1750
Longitudinal dispersivity [m]	3
Diagonal dispersivity [m]	0.6
Vertical dispersivity [m]	0.3

The sorption and degradation properties of pollutants during their transport in groundwater depends on specific conditions by complex way. They may also in the same geological environment take values from a relatively wide range. The selected values of transport parameters can be considered much more conservative but they are adequate in a view of the risk to contamination of drinking water resource. Sorption process was incorporated into the model through the distribution coefficient which indicates the ratio between the dissolved concentration in groundwater and sorbed concentration on rock during the steady state. Loss of the pollutant as a result of chemical reactions was entered by DT50 value in the model (Tab. 3.2).

Tab. 3.2 Transport parameters of dangerous pollutants

parameter	Pollutant						
	METALAXYL-M	SDS-46851	CLOPYRALID	NITRATES	BACTERIES	COPPER	BENZENE
DT50 [days]	38.70	103.00	36.00	1386	15	-	693.00
Distribution coef. [ml.g^{-1}]	1.80	0.08	0.06	0	0	9189	0.02
Retardation factor* [-]	11.16	1.44	1.32	1	1	51875	1.13
Density [kg.m^{-3}]							878.60

* calculated from values in Tab. 3.1 (porous media density and total porosity)

Eight cases of dangerous contamination leakage into the drinking water resource wells were simulated and reviewed two calculations of transport with different regime of water extraction (mean and maximum). In this article are described only cases where a significant risk of contamination occurred: Continuous penetration of pesticides and nitrates in the cottage area, continuous penetration of copper deposits from arm and leakage of petroleum products.

3.1 Modelling the Contamination Risk of Drinking Water Resource by Bacteria Penetration

In this case bacteria were penetrated for 7 days throughout the whole island during the flood and penetration into the wells from the 30 m wide strip along the planned cycling routes by rainfall.

In the first calculation 3.66×10^7 bacterial colony forming units (CFU) were penetrated for 7 days throughout the whole island during the flood. Fig. 3.1 shows possible distribution of surface bacterial concentrations 10 days after the beginning of the flood. Despite the rapid disappearance of bacteria (DT50 = 15 days), bacteria penetrated to the wells.

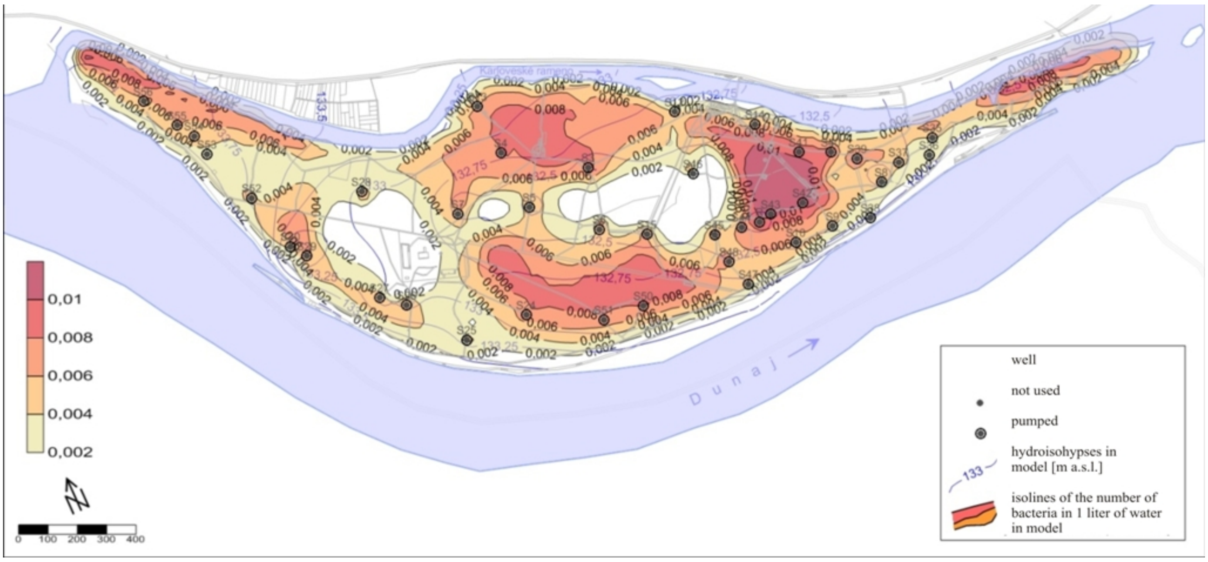

Fig. 3.1 Isolines of the bacterial conventrations in groundwater 10 days after the beginning of the flood [$CFU.l^{-1}$]

Surface bacterial penetration into the aquifer around the island during the flood increases number of CFU of bacteria over the limit standard. At the maximum extraction were simulated higher concentrations than at the average one but with the same amounts at overall.

In the second calculation the bacteria also penetrated into the wells even from the 30 m wide strip along the planned cycling routes by rainfall (Fig. 3.2). Higher concentrations between wells S-14 and S-42 occur due to the slow groundwater flow on the flow boundaries into several wells.

Fig. 3.2 Steady isolines bacteria concentrations in groundwater [CFU.l^{-1}]

3.2 Modelling the Contamination Risk of Drinking Water Resource by Single Point Release of Petroleum Products

Both, the level of the contamination risk of water quality in wells by petroleum products releases in some points on the island that happen in a short time (e.g. from truck tank) and an ecological accident on the Danube River (e.g. from ship tank) were investigated. In the case there was an assumption of leakage of 50 liters of petroleum, which means about 0.5 liters of benzene, on the island. The calculation was performed for 5 selected points on the island, 3 points along the planned routes and 2 vulnerable points on the surface of the island (Fig. 3.3). The leakage would imply highly exceeding the limit standard concentrations of benzene in 6 wells (S-8, S-14, S-15, S-27, S-37 and S-45). This increase in the concentration of benzene in wells can take (according to the location of the point of intersection) tens, even hundreds of days.

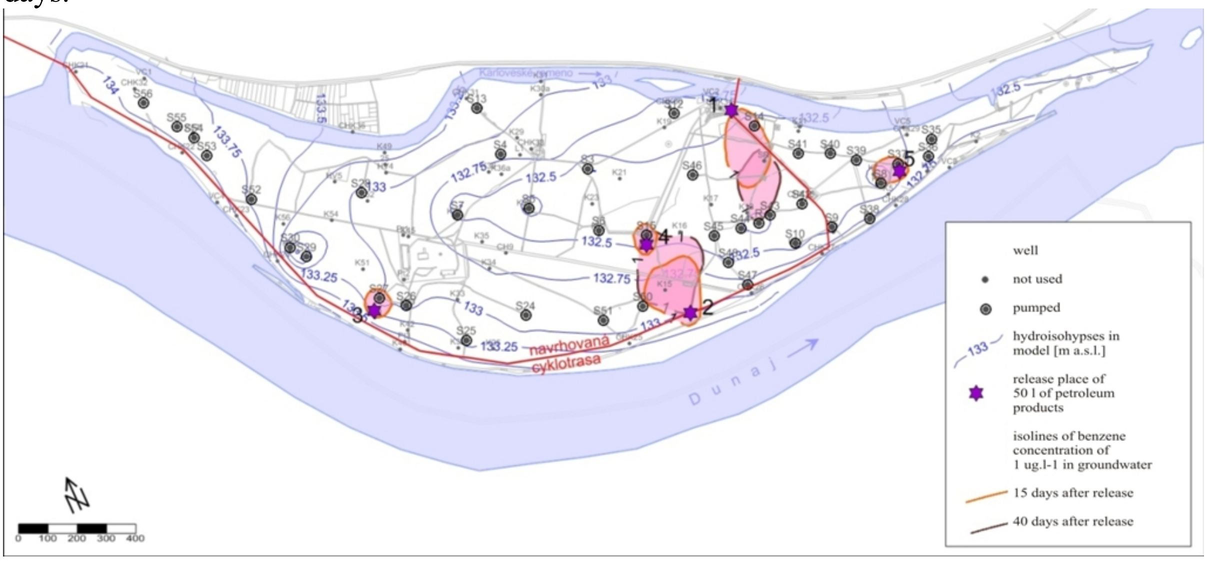

Fig. 3.3 Isolines of benzene concentrations in groundwater 15 and 40 days after the single point release [ug.l^{-1}]

Vulnerability map of the island (Fig. 3.4) shows that 0.5 liters of benzene release into groundwater from almost any point on the island would depreciate groundwater quality. Size of the contaminated area would depend on the release point, wells yields and distribution of wells nearby release. Almost the entire area of the island is in contamination risk except remote unused wells with their surroundings. This modeling showed that the island is extremely vulnerable to accidental or sabotage releases of pollutants. Risk to the island by the sudden release of benzene is at a maximum extraction higher than the average one.

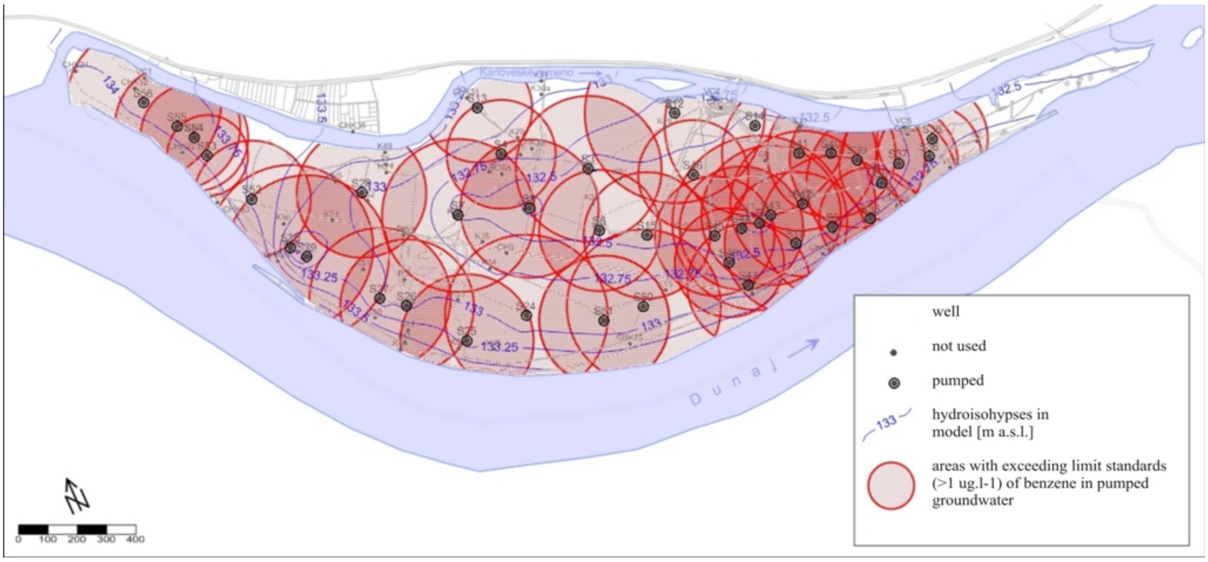

Fig. 3.4 Areas around wells, in which the release of 0.5 liters of benzene is an increase of benzene concentrations in wells in excess of the limit standards for drinking water > 1 ug.l^{-1}

3.3 Modelling the Contamination Risk of Drinking Water Resource by Single Point Release of Pesticides

The calculation confirmed that the immediate point release of the Clopyralid pesticide is very dangerous for groundwater quality. After release of real 20 g of this substance into 3 observation wells (K-50, CHK-25 and CHK-34) there was documented its higher concentration in wells than limit standard (S-14, S-27 > 0.5 ug.l^{-1}) for several tens of days (Fig. 3.5).

The immediate point release of 20 g of the Clopyralid pesticide almost in any point of the island would increase its concentration in wells over the limit standard for drinking water. Vulnerability map shows (Fig. 3.6) that much larger area is in risk at the maximum extraction. This fact is logical because of increasing number of pumped wells and also total extraction is larger.

Fig. 3.5 Isolines of Clopyralid concentration in groundwater 15 days later since single point release [ug.l^{-1}]

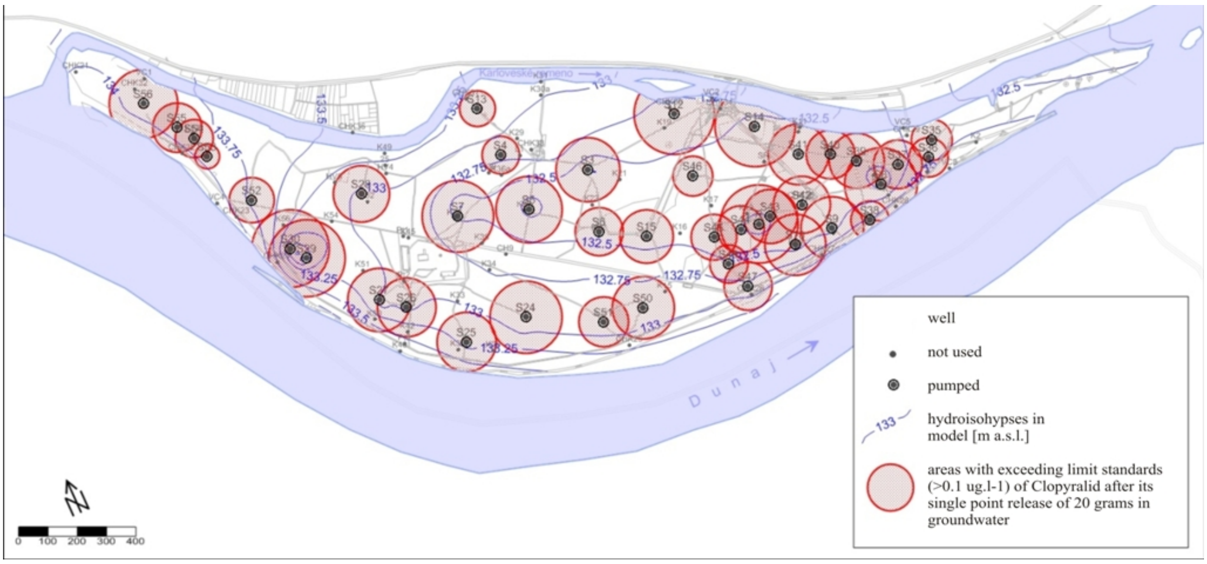

Fig. 3.6 Areas around wells, in which the release of 20 g of Clopyralid pesticide means an increase of its concentration in wells over the limit standard for drinking water > 0.1 ug.l^{-1}

4 CONCLUSION

The purpose of this study was to identify possible contamination risk to drinking water resource in the Sihoť Island for different cases of penetration of dangerous pollutants into groundwater. For this purpose, the method of mathematical modelling of groundwater flow and transport of dissolved substances in groundwater.

Transport of substances in aquifer is a very complicated process. In terms of the risk of contamination spread, the conservative values of necessary parameters were decided in the case of uncertainties. It means that the worst value of each parameter from the possible interval was used in the modelling. Transport of substances was simulated in terms of contamination risk of drinking water resource in adverse hydraulic conditions. The calculations for all cases of pollution penetration in groundwater were performed at an average and maximum water extraction from wells.

Groundwater flow model showed the following facts:

- In both types of extractions, practically the whole area of the island is covered by groundwater table depression cone of groundwater flow into the filter wells,
- Groundwater does not flow above the wells and it is pumped throughout the whole profile in aquifer trapped in filter wells,
- Dominant part of the inflowing groundwater in wells is formed from the Danube River. Filled Karloveské rameno River Arm contributes to wells approx. 17% at the average extraction and approx. 28% at the maximum one,
- Filled river arm forms hydraulic boundary condition which significantly prevents direct groundwater flow under arm to the island.

In modelling the transport of pollutants in groundwater were detected penetration possibilities into the wells filters. Pollutants concentrations will depend on their transport properties (sorption and degradation), the location of the penetration to groundwater, pollutants intensity and the hydrological conditions in groundwater.

Basic results of modeling the transport of pollutants in groundwater can be summarized as follows:

- Contamination of groundwater from outside area behind the river arm does not penetrate into the island when is filled because the river arm forms a hydraulic barrier. Under the arm groundwater can not flow into the island. In the worst case pollutants in groundwater outside the island penetrate into the arm itself, where are diluted with water, and then so diluted they infiltrate in the island. If leakages of pollutants outside the island are not extremely high and river arm is filled, groundwater can not be at risk on the island. Therefore, the implementation of permanently filled river arm would be very useful for protecting the water resource.
- Penetrating pesticides and nitrates in the cottage area at higher defined intensities and active substances can not increase the concentrations of these substances in wells at the level of drinking water standards.
- Bacteria originating from the increased production of faeces (e.g. a proposed cycling route) may infiltrate at periodic floods, but also can be transported to the wells by the gradual leaching from the unsaturated zone by rainfall.
- Copper trapped in sediment in the river arm can not threat water quality in wells due to its large retardation factor.
- Suddenly, whether accidental or sabotage point releases of pollutants pouring directly to the observation wells or leakage into the exposed gravel e.g. after cutting down trees, are extremely dangerous for groundwater quality. Vulnerability maps of 0.5 liters of benzene (Fig. 3.4) and 20 g of Clopyralid pesticide (Fig. 3.5) penetrations demonstrates that the sudden release of small quantity of pollutants almost anywhere on the island can cause significant long-term damage to water quality in large parts of the island. The only way it would be possible to eliminate the consequences of such accidents, it would be a long-term intensive pumping of contaminated water. The costs of such utilization as well as financial losses due to power extraction of drinking water would be extremely high.
- Leakage of petroleum products into the Danube River in the amount of conventional fuel tank vessels can not significantly threat the groundwater quality in wells. However, it is necessary to ensure that the petroleum products as soon as possible flow away from the

island and so that they were captured in the form of phase on the Danube River banks. It would be ideal in this case, if contaminated water can not flow into river arm.

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CURRENT STATE OF TRANSBOUNDARY TECTONIC LAKES-CASE STUDY OHRID, PRESPA AND DOJRAN LAKES

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Abstract

The three tectonic lakes, Ohrid, Prespa and Dojran, situated on the south-eastern part of the Balkan Peninsula are transboundary and have significant natural and cultural heritage. All three lakes in the past decade have faced dramatically water level declinations which were not clearly identified yet. How much the water storage decrease of the lakes is impacted by the natural causes, for example of climate change effect, and how much by human activities is the question that is not easily to be answered. This paper deals with statistical trend analysis of the observed water level oscillations. Data on average, minimum and maximum annual water levels have been collected for the period 1951-2010. Linear and non linear regression analyses have been applied by dividing the entire period of the observation into sub-periods. Conclusions and simplified climate change impact assessments are included.

Keywords

tectonic lakes, water level, statistical trend, regression analysis

1. INTRODUCTION

Three tectonic lakes of natural and cultural treasure situated on the south-eastern part of the Balkan Peninsula in the past decade have faced dramatically water level oscillations which were not clearly identified yet. How much the observed state of the lakes is impacted by

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nature and how much by human activities is the question that should be not answered easily having in mind that all three lakes are transboundary.

The largest one is Ohrid Lake formed in the Tertiary period between 3.5 and 4 million years ego. Because of its biodiversity and unique cultural heritage, UNESCO declared this lake a World Cultural and Heritage Site in 1980. The shape of the lake is regular elliptical. The free water surface of Ohrid Lake is 358 km^2 with 251 km^2 belonging to Macedonia and 107 km^2 to Albania. The average altitude of the lake is 1109 m a.s.l. The volume of the lake is estimated to 50.7 km^3. The surface outflow from the lake is at Struga to the river Crn Drim that has been regulated by a gate.

The Prespa Lake lies to the southeast of Ohrid Lake. Its watershed and water surface are shared between Macedonia, Albania and Greece. The lake is with irregular shape and is located at higher altitude than the Ohrid Lake for about 155 m. To the south, on Greek territory, the narrow sandy isthmus Gladno Polje separates the Macro and Micro Prespa lakes. The Macro Lake water surface area is 253.6 km^2, and that of Micro Lake is only 47.4 km^2. The surface inflow to Prespa Lake is mainly from north and east part of the watershed. The Lake does not have surface outflow. The waters from the lake outflow through karst underground conduits into Ohrid Lake.

The Dojran Lake is a tectonic lake situated in southeastern part of Balkan Peninsula and is shared by Macedonia and Greece. The lake is elliptic shape with 7.1 km width and 8.9 km length on north-south direction. The water surface area of the lake at normal elevation is 42.2 km^2 out of which 27.1 km^2 (63%) belongs to Macedonia. Total watershed area is 271.8 km^2 out of which 92.1 km^2 (32%) belongs to Macedonia. The volume of the lake at normal water level is 262 million m^3. Recharge of the lake is from direct runoff, small rivers and groundwater. This lake doesn’t have surface outflow. The only outflow is by evaporation from the lake water surface.

Water level oscillation

Basic parameter in hydrological state and vulnerability assessment of tectonic lakes is the water level oscillation. Time series data of recorded water levels for the three above described lakes are analyzed. Data for Ohrid Lake are collected for the period 1951-2010 at the hydrological station in Ohrid. Long-term average water levels with linear trend lines are presented in Figure 1 and with nonlinear (polynomial) regression analysis in Figure 2. Maximum water level is observed in 1963 with amplitude of 142 cm above the water level gauge zero (694.59 m asl). Minimum water level is observed in 1990 with amplitude of –17 cm bellow the water level gauge zero. The computed regression coefficients for the entire observed period 1951-2010 in both cases are R=0.46 for linear trend line and R=0.60 for nonlinear trend line.

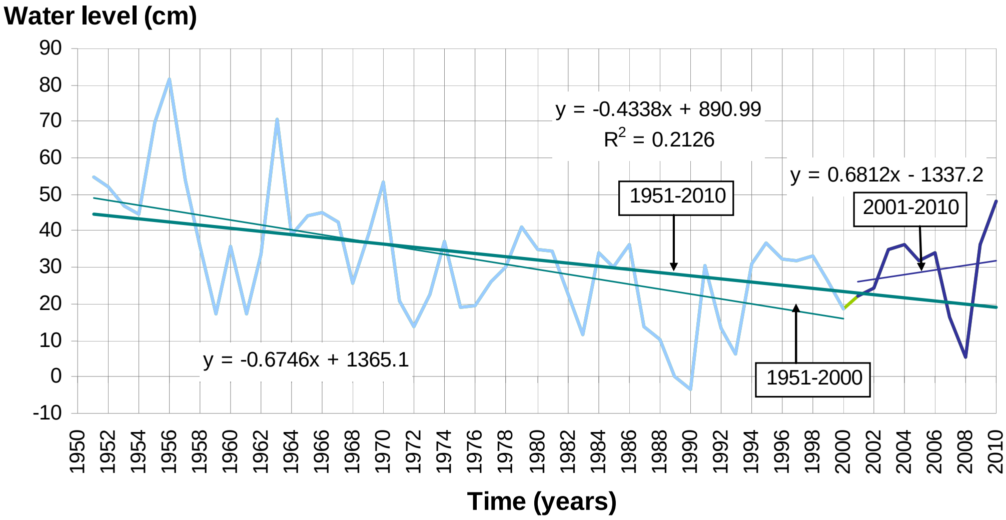

Figure 1 Average annual water levels of Ohrid Lake with linear regression analysis

Analyzing the long-term monthly water levels it is observed that lowest water levels are recorded in October, while maximum ones are recorded in May and June. The water level oscillation of Ohrid Lake for the observed period is with rather small amplitudes, especially after 1963 due to the construction of control gate at inflow of Crn Drim in Struga.

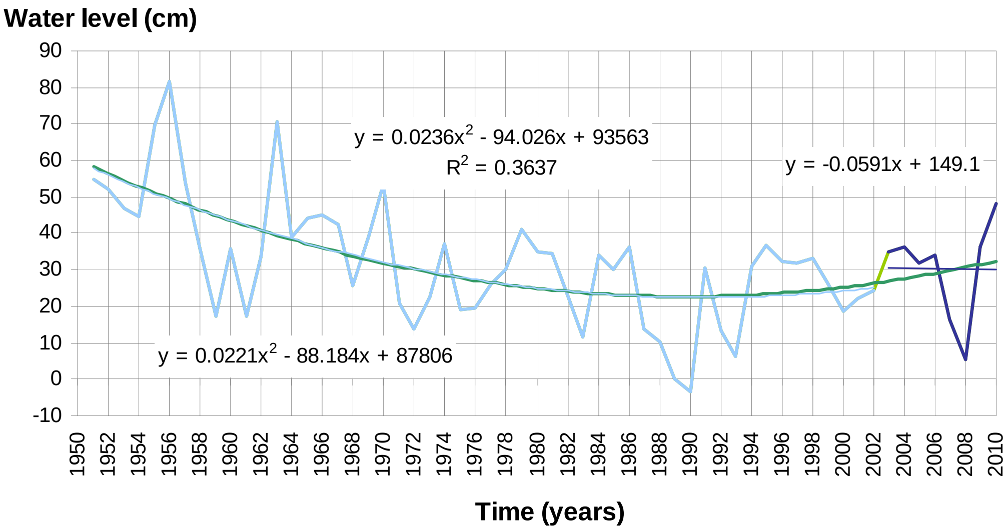

Figure 2 Average annual water levels of Ohrid Lake with nonlinear regression analysis

Prespa Lake water level data are collected for the same period 1951-2010 at hydrological station in Stenje. Long term average annual water levels with linear trend lines are presented in Figure 3. Significant water level oscillations are observed with maximum amplitudes of 415 cm and – 493 cm above and bellow the water level gauge zero, respectively. Serious water level declination is observed after 1987 and two sub-periods may be selected for trend analysis, 1951-1987 and 1987-2008. The long-term average annual water levels with nonlinear (polynomial) regression analysis are presented in Figure 4. The regression coefficients for the entire observed period 1951-2010 in both cases are very high, R=0.88 for linear trend line and R=0.94 for polynomial trend line. Concerning the long-term monthly water levels it is observed that the minimum water levels are recorded in November and maximum ones in June. The observed water level oscillations show very complex hydrogeological characteristic of this lake. Namely, the serious water volume loss can be explained by the outflow through karst underground conduits of Galicica Mountain from Prespa Lake into Ohrid Lake. It is obvious that natural and anthropogenic impacts in the watershed should be identified and investigated. Therefore, from hydrological and hydrogeological aspects, both lakes can not be analyzed separately (Popovska, Bonacci, 2007).

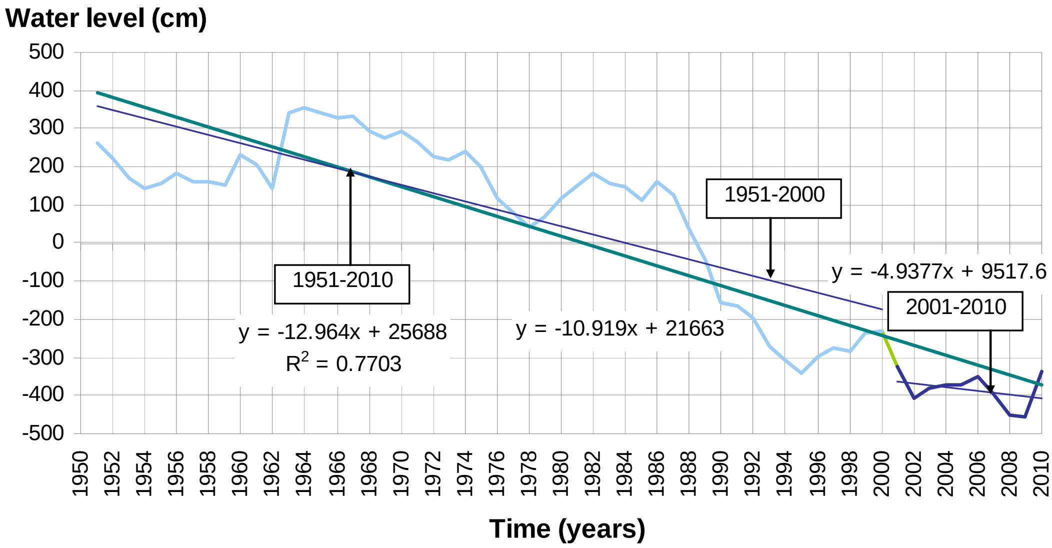

Figure 3 Average annual water levels of Prespa Lake with linear trend analysis

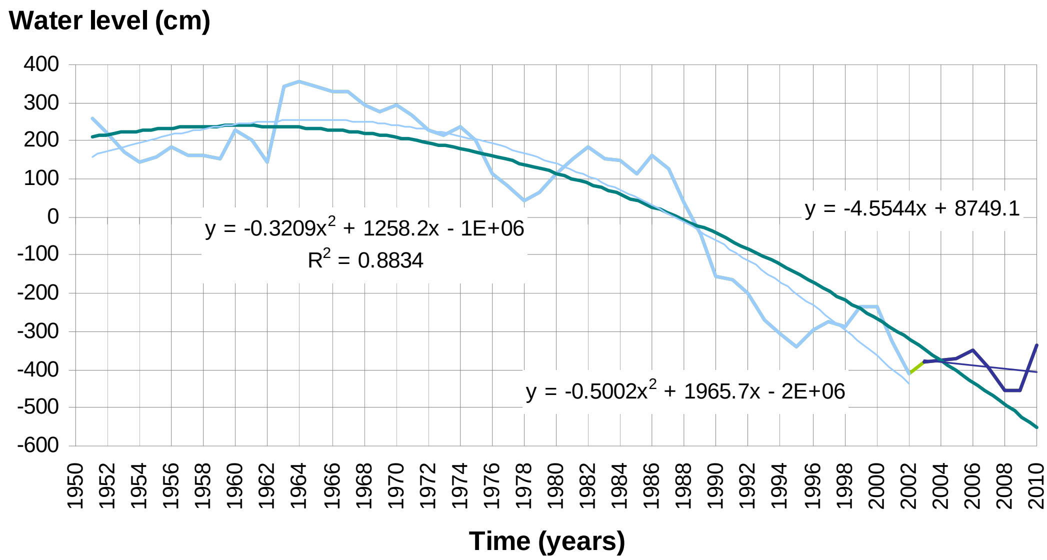

Figure 4 Average annual water levels of Prespa Lake with nonlinear trend analysis

The smallest Dojran Lake is analyzed with recorded water level data at Nov Dojran for the period 1951-2010. Long term average annual water levels with linear trend lines are presented in Figure 5. Significant water level oscillations are observed with maximum amplitudes of 311 cm above the water level gauge zero (144.93 m asl) recorded in 1956 and –388 cm bellow the water level gauge zero recorded in 2002. Serious water level declination is observed after 1987 and three sub-periods may be selected for trend analysis, 1951-1987 and 1987-2002 and 2002-2010. The long term average annual water levels with nonlinear trend line are presented in Figure. Analyzing the long-term monthly water levels it is observed that minimum one usually occur in October and maximum one in May. The total amplitude of water level oscillation is 692 cm. This significant water volume loss has initiated serious biodiversity vanishing and economic impacts in the region.

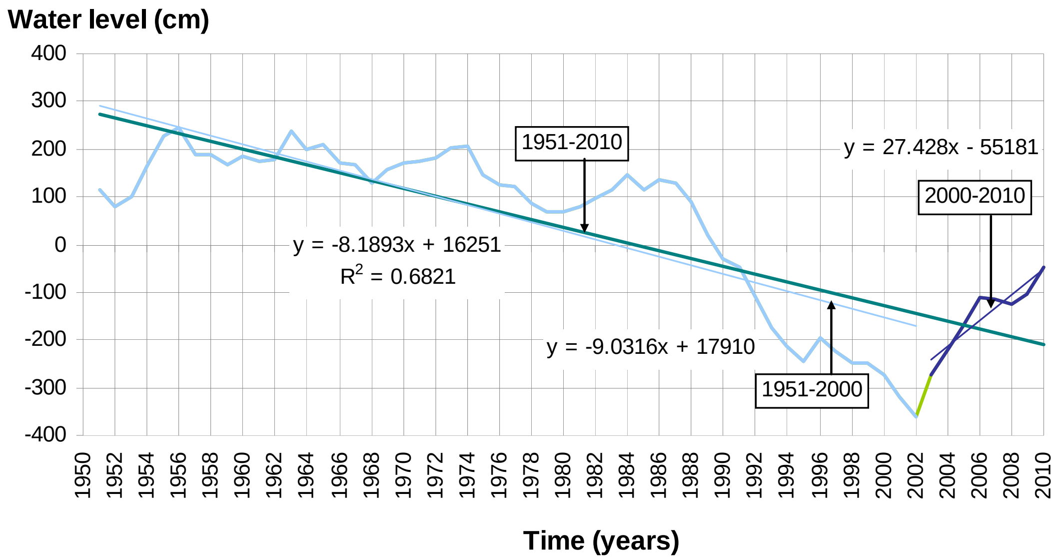

Figure 5 Average annual water level of Dojran Lake with linear trend analysis

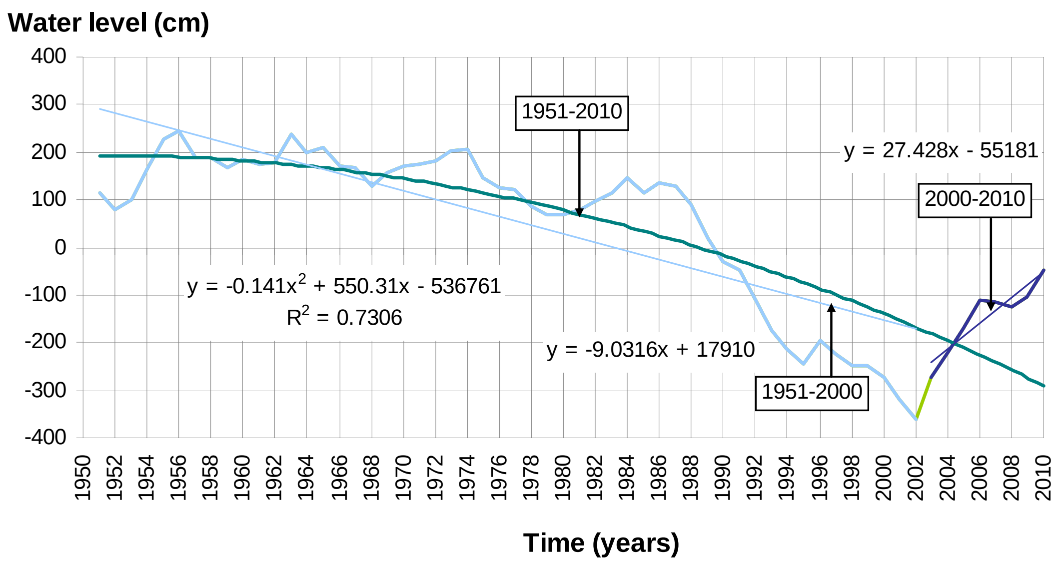

Figure 6 Average annual water levels of Dojran Lake with nonlinear trend analysis

Conclusions

All three tectonic lakes are situated in the south borders of the Republic of Macedonia with Albania and Greece. They have been surrounded by high mountains and are characterized with reach biodiversity. All three lakes are significant water resources for drinking water, irrigation and tourism. Due to climatic and anthropogenic impact the lakes' water volume has started to decrease in the middle of nineties. For the deepest one Ohrid it is not observed significant water level declination as it was recorded for Prespa and Dojran. In 2002 Dojran Lake has faced the volume loss of over 200 million m^3 due to over water extraction from its watershed and the lake. Prespa Lake is hydrogeological interesting karstic phenomenon and their volume loss is closely related to underground conveyance with Ohrid Lake. Last decade the region is in very convenient hydrological regime. There have been observed rainfall increase and rainfall redistribution, stream flow increase, and groundwater table rising. Due to these climatic variations the water volume in all three lakes significantly increased.

Trend analysis of the registered lakes' water level for the period 1951-2010 has been performed. To get more confidence id the obtained results the entire period was divided into two sub-periods 1951-2000 and 2000-2010. Statistical trend analysis is typically associated with regression analysis. Creating a trend line and calculating its coefficients allows for the quantitative analysis of the underlying data and the ability to both interpolate and extrapolate the data for the forecast purposes. The obtained regression coefficients are between 0.461 and 0.879 for linear function and 0.603 and 0.939 for applied nonlinear function.

Excel includes multiple functions for regression analysis such as linear, exponential, logarithmic, power or polynomial. Extrapolation is when the value of a variable is estimated at times which have not yet been observed. This estimate may be reasonably reliable for short times into the future, but for longer times, the estimate is likely to become less accurate, even dangerous. Linear and nonlinear trend lines were obtained to the observed time series data for the period 1951-2010. Extrapolation by polynomial trend lines to future longer period in case of climate change projections, for example to 2015 or 2050, would be less reliable in comparing to the linear ones although the regression coefficients of polynomial regression in some cases are very high.

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PARAMETRIZATION OF A MULTILINEAR FLOOD ROUTING MODEL FOR RIVERS WITH VARIABLE TRAVEL-TIME OF FLOOD PEAKS

J. Szolgay[1], M. Danáčová[2], and P. Šúrek[3]

Abstract

The discrete state space representation of the Kalinin-Miljukov model was used as the basis for a multilinear discrete cascade flood routing model. The time distribution scheme of model inputs was employed in the setup of the multilinear model and the travel-time parameter of the model was allowed to vary with discharge. The performance of the alternative models for different flood wave shapes is evaluated through comparison of model efficiency with the results of the optimal linear models on Torysa river. The results of the presented case study shows that without sacrificing the simplicity of linear model, the inclusion of additional information on the variability of the travel time parameter into the multilinear model enables better prediction of the flood propagation process.

Keywords

Multilinear model, flood routing, travel-time, discharge, river reach

1 INTRODUCTION

The application of hydrologic flood routing methods, in which the river is described as a lumped system, still remains a rational alternative (under certain hydraulic conditions), when the use of hydraulic models, due to their complexity and data intensity, is not reasonable. In this study a nonlinear hydrologic routing model is proposed. It is based on the discrete state space representation of the cascade of linear reservoirs (the Kalinin-Miljukov-Nash flow routing model) and belongs to the class of lumped hydrologic flow routing models, which are

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based on the storage-discharge relationship of the modelled river reach. An empirical flood wave speed and discharge relationship, which accounts for the nonlinearity of the flood routing process, is implemented into the state-space model. The resulting model belongs to the family of multilinear models.

In several previous studies [15], [16] we have shown that the relationship between travel-time of flood peaks and the peak discharge can be used to parameterize a multilinear flood routing model based on the state space representation of the classical Kalinin - Miljukov cascade. In this study the time parameter of the state space model was allowed to vary with input discharge according to the travel-time peak-discharge relationship.

In this paper, which is methodologically extending the approach from Szolgay and Danáčová [17] an empirical model and piecewise linear model of that relation has been considered. The relationship between travel-time of flood peaks and peak discharge was studied on a reach on Torysa river (river reach between Prešov – Košické Olšany was selected). The empirical data between travel-time of flood peaks and peak discharge was also studied using a larger set of recent flood data. The shape and parameters of the piecewise relationship were fitted by optimisation of the multilinear routing model performance on a recent flood wave with the help of a genetic algorithm. To remain flexible in the determination of the shape of the travel-time discharge relationship, which is formed by a chain of consecutive linear segments (piecewise linear function), the number of these had to be selected a-priori. Four to ten segments were considered in this paper and the optimizations were performed using constraints in order to arrive at feasible solutions. To smooth the fluctuations of these relationships a 5th degree polynomial was fitted into these in the next step. All relationships were compared with the empirical data on travel-times and subsequently tested in the Kalinin and Miljukov model with several verification floods.

2 GENERAL METHODOLOGY AND DATA

The general applicability of using travel-time discharge relationships to model the time parameter in a multilinear routing model was further tested in this study. To account for the variability of travel-time with discharge including the effects occurring at the change from channel to flood plain flow, the time parameter of the discrete multilinear cascade reservoirs was allowed to vary as a function of the discharge in the river reach. It is expected, that this relationship is implicitly related to flow conditions in the channel by the fitted empirical relationships between the travel-time of flood peaks and inflow to the river reach.

2.1. Empirical travel-time and discharge relationships

Price [12] postulated a celerity (wave speed) and discharge relation, which can be interpreted as consisting of two power functions, one for the main channel and the other for over-bank flow, joined by an S-shaped transition curve (see Fig. 1, for the wave-speed and Fig. 2 for travel-time).

At first the flood wave-speed increases sharply with increasing discharge followed by a decrease. The maximum wave-speed usually occurs between one half and two thirds of bank-full discharge. The slight decrease in speed with the increasing discharge is probably due to increase of roughness caused by bank vegetation at full in-bank flow and the floodplains start to be filled by lateral outflow. The decrease in wave-speed occurs until the point when the floodplain is fully connected to the flow [12]. This type of relationship can be similarly interpreted as the travel-time and discharge relationship (see Fig 2).

To estimate the travel-time and discharge relationships from flood data the method used by Price [12] in the interpretation of Wong and Laurenson [19] was adopted here. Wave-speed was calculated as reach length divided by travel-time of the hydrograph peak. In contrary to these studies, the corresponding discharge was not taken as the average of the inflow and outflow peak discharges.

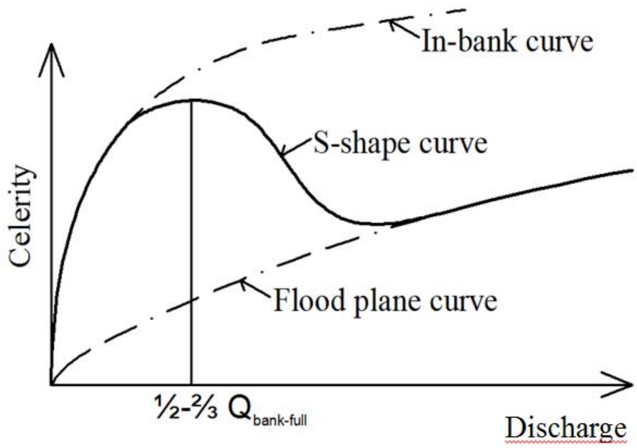

Fig. 1 The general shape of the wave-speed (celerity) and discharge relationship according to Price [12]

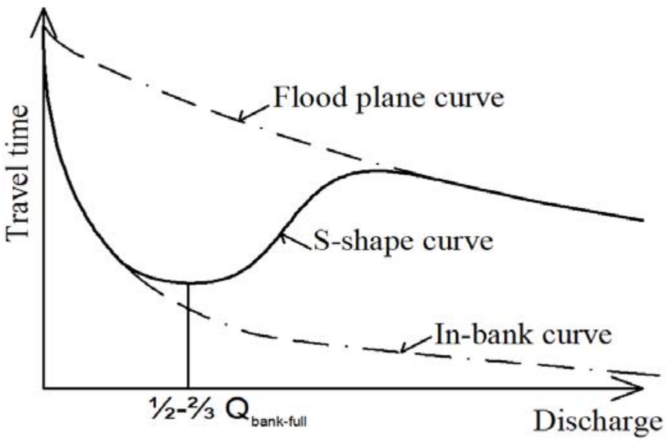

Fig. 2 The general shape of the travel-time and discharge relationship according to Price [12]

2.1. Data

Data pairs for the travel-time and discharge relationships were selected based on these criteria:

- Availability of hourly discharge data;
- Negligible lateral inflow or availability of measured lateral inflow data;

- Stable morphologic conditions and no anthropogenic changes of the river bed during the period of analysis;
- Uninfluenced hydrologic flow conditions (e.g. by reservoir operation, withdrawals etc.) during the period of analysis.

In this paper river reach between Košické – Olšany (45,3 km) was selected on the Torysa River (Fig. 3). River reach is located in its alluvial part (average slope 1,1‰).

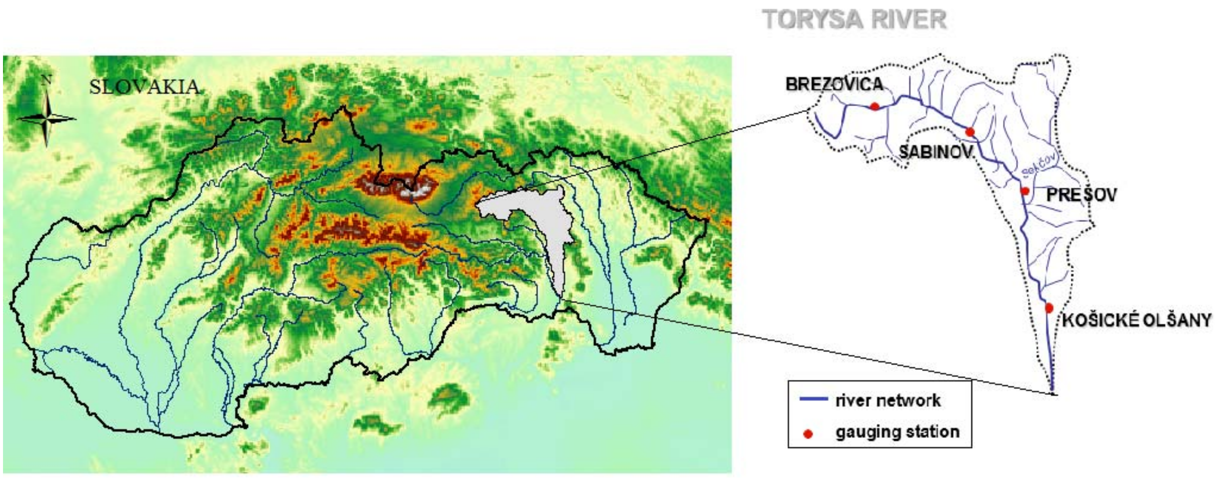

Fig. 3 The river network of the Torysa River with the river reach Prešov – Košické Olšany considered for analysis.

In our analysis of the flood wave travel-times, we used hourly discharge data from the years 1992-2003 (data were provided by the Slovak Hydrometeorological Institute). The resulting empirical data pairs of the relationship are shown in Figs. 4 and 5.

2.3 Multilinear flood routing

Development of conceptual non-linear reservoir type cascade models was one of the approaches how to incorporate non-linearity into hydrologic routing methods (see e.g. [10], [11], and [13]. These models use a non-linear storage-outflow relationship in conjunction with the lumped continuity equation.

As an alternative to the use of such a relationship, the process models can be assumed to respond linearly to the input at any point in time, but with the model parameters recalculated as a function of flow values. These techniques, commonly referred to as multilinear modelling, usually distinguish different components in the input hydrograph, each of them being subsequently routed through a linear sub-model. The overall output of the non-linear system consists of the outputs from the linear sub-models. The different inflow components can be obtained by dividing the input hydrograph into segments horizontally or vertically. The former method is called the amplitude distribution scheme; the latter is the time distribution scheme. Kundzewicz [6] gave an extensive description of the principles of these methods.

These concepts served as a basis for the development of the multiple linearization flow routing model [5] and the non-linear threshold model [1], [2]. The multilinear discrete cascade model for channel routing based on a discrete representation of the Nash cascade as derived by O'Connor [7] was used in Perumal [8], [9] and [3]. The discrete state space

version of the Kalinin-Miljukov model [4] was used to simulate flood routing in a reach of the Danube River with two linear sub-models [15].
The discrete state space version of the cascade of linear reservoirs (the classical Kalinin - Miljukov cascade [4] was used here as the routing model [18, [14]. The fact that the state vector in the state space model contains the complete past history of the modelled process at a given time is used here for building the multilinear discrete cascade model according to the time distribution scheme as described in Kundzewicz [6], and Becker and Kundzewicz [7]. It is based on the algorithm for dividing the input I into a series of consecutive non overlapping series I_1, I_2, … I_t and a set of distinct linear sub-models driven by the respective inputs I_i. The response Q of the (in general non-linear) multilinear model consists of the composition of responses Q_1, Q_2, … Q_t of these sub-models to the corresponding input signals I_1, I_2, … I_t.

3 PARAMETRIZATION OF THE MULTILINEAR DISCRETE CASCADE MODEL BY EMPIRICAL TRAVEL-TIME DISCHARGE RELATIONSHIPS

It is known that the product $n.k$, where n is the number of linear reservoirs in the series and k is the storage coefficient, can be regarded as the travel-time of the modelled reach in the Kalinin-Miljukov scheme [4]. In several previous studies it was shown that the relationship between travel-time of flood peaks and the peak discharge could be used to parameterize this model. The time parameter of the state space model was allowed to vary with input discharge according to the travel-time peak-discharge relationships (e.g. [16], [11]).

3.1. Estimation of the travel-time discharge relation from empirical data

The travel-time and discharge relation was fitted to the travel-time data by regression. Only for the tendencies suggested by the data was accounted for. No theoretical models of the travel-time and discharge relationships were considered. The performance of the respective multilinear models was verified on a set of verification floods. Model performance was evaluated with the Nash Sutcliffe criterion (see results in Tab. 1).

3.2. Estimation of the travel-time discharge relationship by optimizing the multilinear model's performance

To remain flexible in the determination of the shape of the travel-time discharge relationship, this is proposed as a chain of consecutive linear segments (piecewise linear function). The number of segments and their parameters were estimated by maximising the Nash-Sutcliffe criterion of the simulation runs of the multilinear routing model on one large flood wave with the help of a genetic algorithm. Large flood wave had to be selected in order to cover the entire interval of flows and their corresponding travel-times. Four to ten segments were considered when the constrained optimization was performed. To smooth out the fluctuations of these relationships (as it can be seen in Fig.5) a 5th degree polynomial was also fitted into these chains of segments (Fig. 5) as an alternative.
The piecewise linear relationships were estimated from running the multilinear model on the large flood from 14.03.1993 to 02.04.1993). All these relationships (including the averaging polynomial) fit the empirical data reasonably well (see Fig. 5).

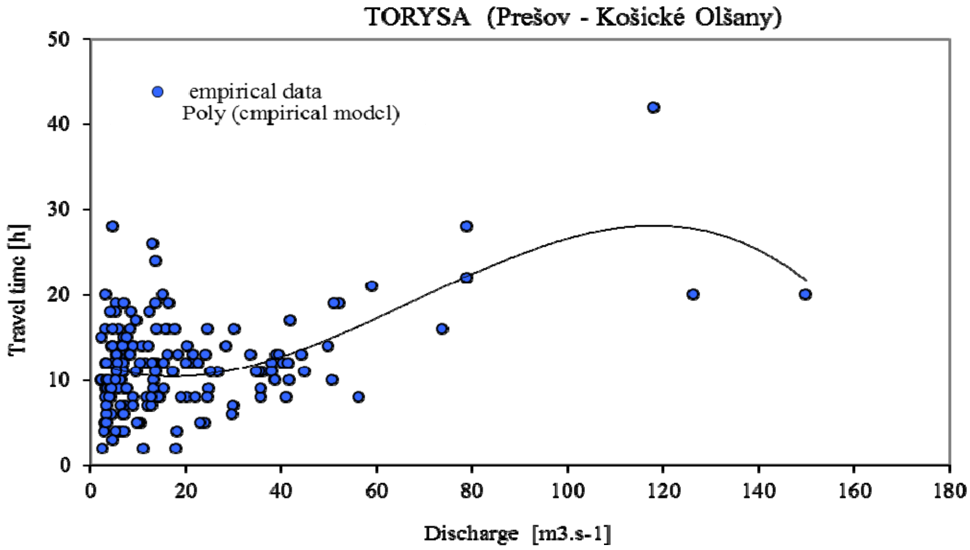

Fig. 4 Empirical travel-time and discharge relation estimated by regression (third order polynomial fitted to the data)

4 VERIFICATION OF THE EMPIRICAL TRAVEL-TIME DISCHARGE RELATIONSHIPS

Five floods were selected for model verification (see Table 1). The values of the Nash-Sutcliffe criterion were computed for these models: the best linear models optimized by genetic algorithm for each wave separately, the best piecewise linear model consisting of 8 segments, the averaging polynomial and the 3rd order polynomial regression model, respectively. In principle it can be seen that the multilinear models based on the variable travel-time discharge relationships performed slightly better than the optimal linear model and the 3rd order polynomial model. These results can be therefore considered as further step toward the proof that a time variable storage parameter can be used in the Kalinin-Miljukov model (but this possibility has to be verified empirically for each application [16]).

5 CONCLUSIONS

The discrete state space representation of the Kalinin-Miljukov model was used as a multilinear flood routing model. The time distribution scheme of model inputs was employed in the multilinear model and the travel-time parameter of the model was allowed to vary with discharge.

The relationship between travel-time of flood peaks and peak discharge was studied on a reach of the Torysa River. An empirical model and several piecewise linear model of that relation has been considered. The shape and parameters of the piecewise linear models were fitted by optimization of the multilinear model performance on one large flood wave with the help of a genetic algorithm.

The resulting relationships fitted well the empirical data on travel-times of flood peaks and were consistent with the findings in the literature regarding and empirical evidence. The fitted empirical piecewise linear model was used to model the variability of the time parameter in the discrete state space representation of the Kalinin and Miljukov model on five verification floods. The modelling results showed that the inclusion of empirical information on the variability of the travel-time with discharge from only one flood enabled satisfactory accuracy of the prediction of flood propagation. The method can thus be used for calibrating the conceptual multilinear flood routing model on a small set of flood waves.

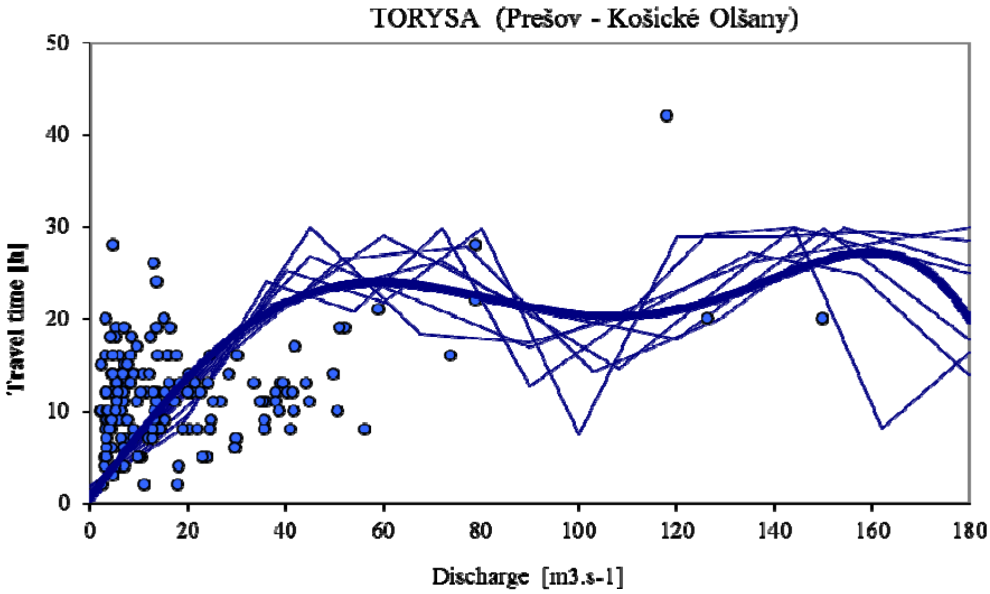

Fig. 5 Piecewise linear relationships fitted by optimizing the multilinear model performance and a 5th order smoothing polynomial fitted to the piecewise linear models (thick line)

Tab. 1 Nash Sutcliffe coefficients obtained for the verification runs of the multilinear model compared to the optimal linear model and the empirical regression model

No.	**Flood duration**	**Peak discharge** [$m^3.s^{-1}$]	**Model**			
			GA optimized			Empirical polynomial
			8 segments	Smoothing polynomial	Optimal linear	
1	01.11. – 25.12.1991	21	0.965	0.965	0.977	0.977
2	03.04. – 01.06.1995	45	0.956	0.956	0.945	0.945
3	10.04. - 10.06.2001	83	0.954	0.954	0.956	0.955
4	15.03 – 30.04.2000	129	0.922	0.922	0.924	0.925
5	10.07. – 03.08.2001	158	0.867	0.867	0.858	0.853
Average Nash-Sutcliffe coefficient			**0.933**	**0.933**	**0.932**	**0.931**

Acknowledgements

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REGIONALIZATION OF DEPRESSED AREA OF EAST SLOVAKIAN LOWLAND ACCORDING TO SOIL´S WATER CAPACITY

A. Tall[1] and M. Gomboš[2]

Abstract

Most of present research in soil physics is oriented to numerical methods used for obtaining hydrophysical characteristics of the soil from data, which can be easily measured (in most cases from the particle size distribution of soil). Outputs are called "pedotransfer functions", which are useful for calculation hydrophysical characteristics of soils. This paper deals with application of pedotransfer function for regionalization of clay-loam soils in East Slovakian Lowland according to soil´s water capacity.

Keywords

Clay soil, Pedotransfer function, Saturated water content, Soil´s texture

1 INTRODUCTION

East Slovakian Lowland (ESL) is characterized by high variability of soil types, when on relatively small area of land, there are soils with different textural composition. At ESL therefore occur soils from light soils (with high content of sand fraction), over loamy soils, to extremely heavy clayey soil with a dominant clay content. This heterogeneity is caused by difficult tectonic evolution of the area of ESL. The texture of soil is closely related to its water retention capacity. In general, with an increasing content of fine particles in the soil, also increases the ability to hold more water. Especially clay particles are characterized by their high ability to hold water in its structure and thereby highly increase retention capacity of heavy soils. Water capacity of the soil can be easily quantified using hydrolimit Θ_s (saturated water content). This hydrolimit determines the maximum amount of water that the soil can

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absorb. Θ_s is a maximum moisture from water retention curve (pF = 0). In general, saturated water content of soil, is almost equal to soil´s porosity. This assumption is not valid in heavy soils due to their volume changes and entrapped air in the pores. Saturated water content in heavy soils mainly depends on textural composition.
This paper presents the results of regionalization of depressed area of ESL according to saturated water content of soil.

2 MATERIAL AND METHODS

A typical area of ESL with various soil types was chosen for the regionalization according to the soil´s water capacity. The area is bounded on its west side by the river Laborec, south by the river Uh and east by water channel Revištia - Bežovce. Northern border is formed by south coast of Zemplínská šírava together with road no. E50. It is a lowland area with central depression. In the midst of this depression are located Senianske ponds, which are subsidized by the water from the Zemplínska šírava through the channel Čierna voda. Circuit of examined area is 72.3 km and its area is 278 km^2. The average altitude of the area is 103 m above sea level (ASL). Minimal altitudes (98 m ASL) are in the central part of the depression around Senianske ponds and maximal altitude (133 m ASL) is in the northern edge of the area (see Fig. 1).

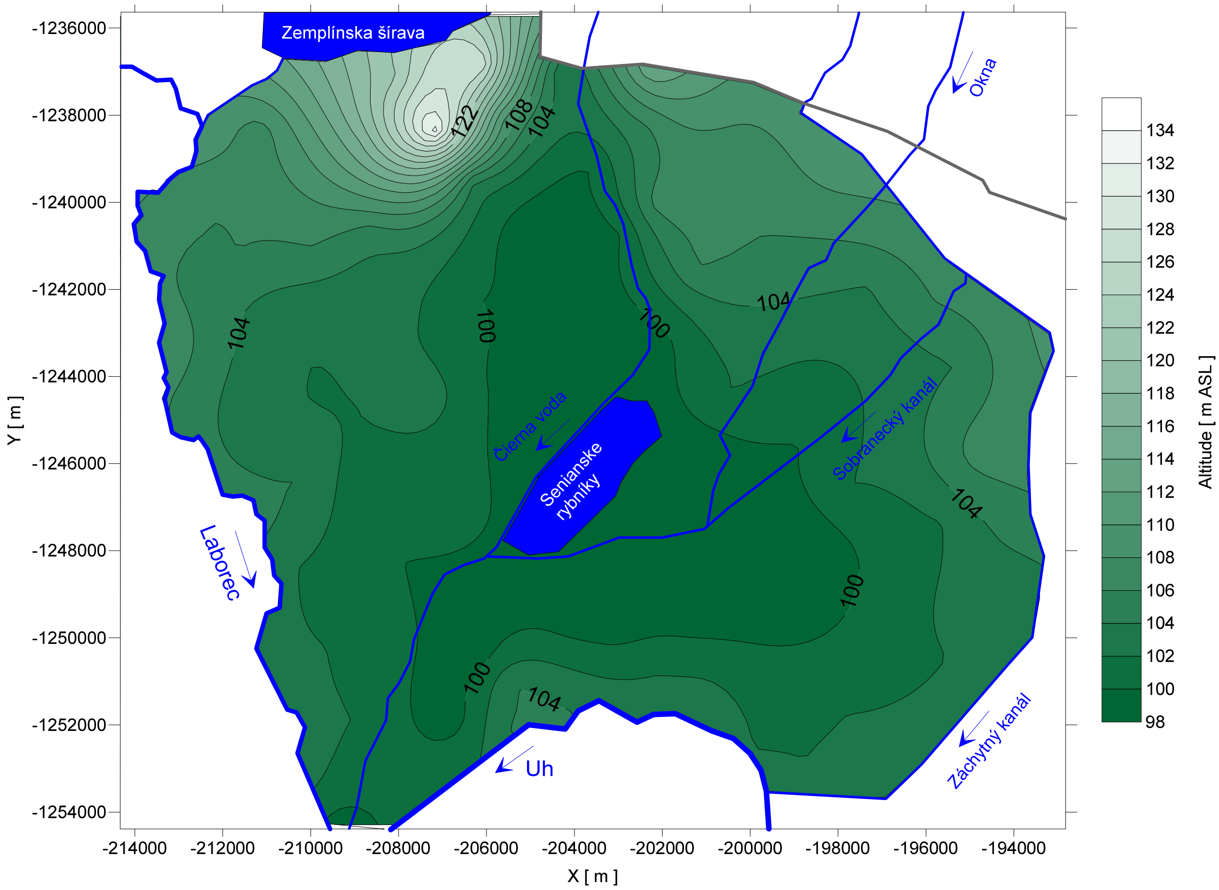

Fig. 1 Terrain map of observed area

In the studied area was performed a field survey and picked up were 106 disturbed soil samples from a depth of 0.5 m. Sampling density in the area was 1 sample to 2.6 km^2.

Coordinates of each sampling site were recorded using GPS. The situation of sampling sites is shown in Fig. 2.

Fig. 2 Map with sampling sites

From the samples was in laboratory performed particle size analysis by the method of Cassagrande and were classified soil types. Results from particle size analysis are shown in Fig. 3 and Fig. 4. According to these figures we see that the textural composition of investigated soils ranging from loam to clay with a dominant position of silty - clay loams. The next work was focused on evaluating of retention capacity of soils. Used was hydrolimit Θ_s – saturated water content. Direct determination of Θ_s in the laboratory is time-consumpting process. Necessary is pick up undisturbed soil samples in the field, then saturate and dry of samples, and measure of weight and volume. In the laboratory of Department of Lowland in Michalovce were over the past years determined many values of Θ_s from ESL. Based on these determinations was created pedotransfer function, which makes it possible to indirectly determine the value of Θ_s using textural analysis of soil. In this work was for the calculation of Θ_s used next pedotransfer function [1], [2]:

$$\Theta_s \text{ [vol \%]} = -242{,}025 + 3{,}09953 \cdot (\text{\% I.fr.}) + 2{,}91079 \cdot (\text{\% II.fr.})$$

$$+ 2{,}62629*(\%\ \text{III.fr.}) + 2{,}85083*(\%\ \text{IV.fr.}) \qquad (1)$$

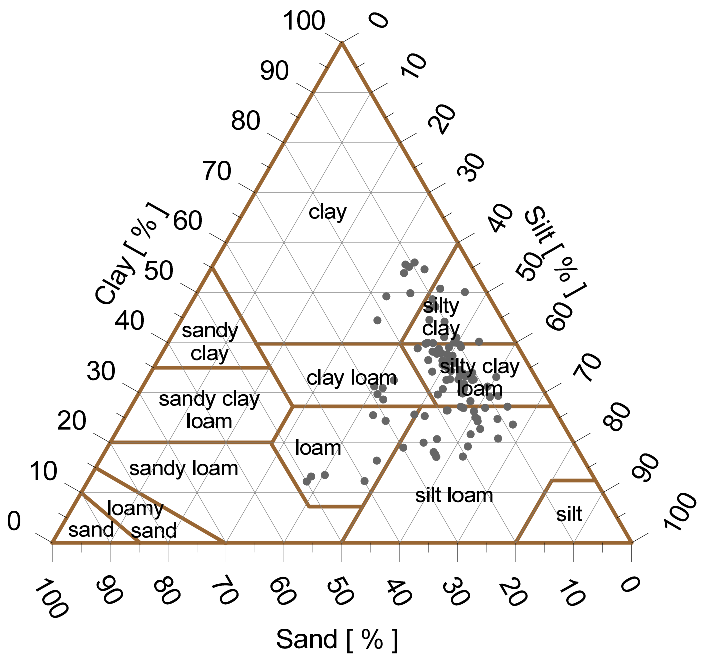

Fig. 3 Soil texture triangle according the USDA

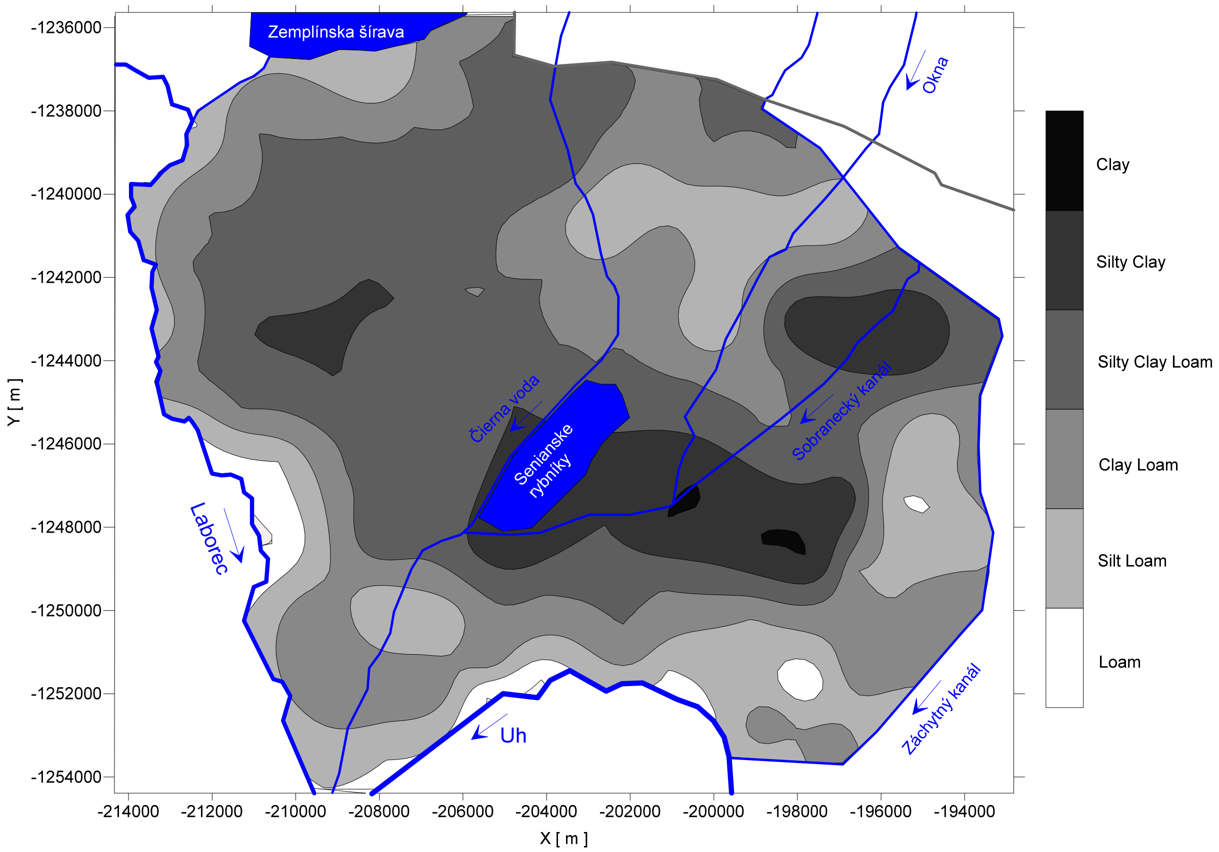

Fig. 4 Soil texture map according the USDA

To investigate the influence of textural composition on the value of Θ_s were used statistical methods. This indirect method of determining the Θ_s of the soil is preferred because it is quick and easy. Reliability of this method is proved by very high degree of correlation (r = 0.966) between the measurements and the equation (1).

3 RESULTS

Calculated results of regionalization of soils according the Θ_s are graphically shown in Fig. 5. Θ_s values are expressed in percentages and their values vary from 16.39% to 53.57%. The average value of Θ_s is 40.72%. For the illustration, there is in Fig. 6 shown spatial distribution of colloidal clay particles (particles < 0.001 mm – (I. fraction)). Particles < 0.001 mm (I. fraction) have the highest influence to the ability of soil to hold water. From the Fig. 6 it is evident the high areal variability of the clay particles and thus hydrophysical properties of soils.

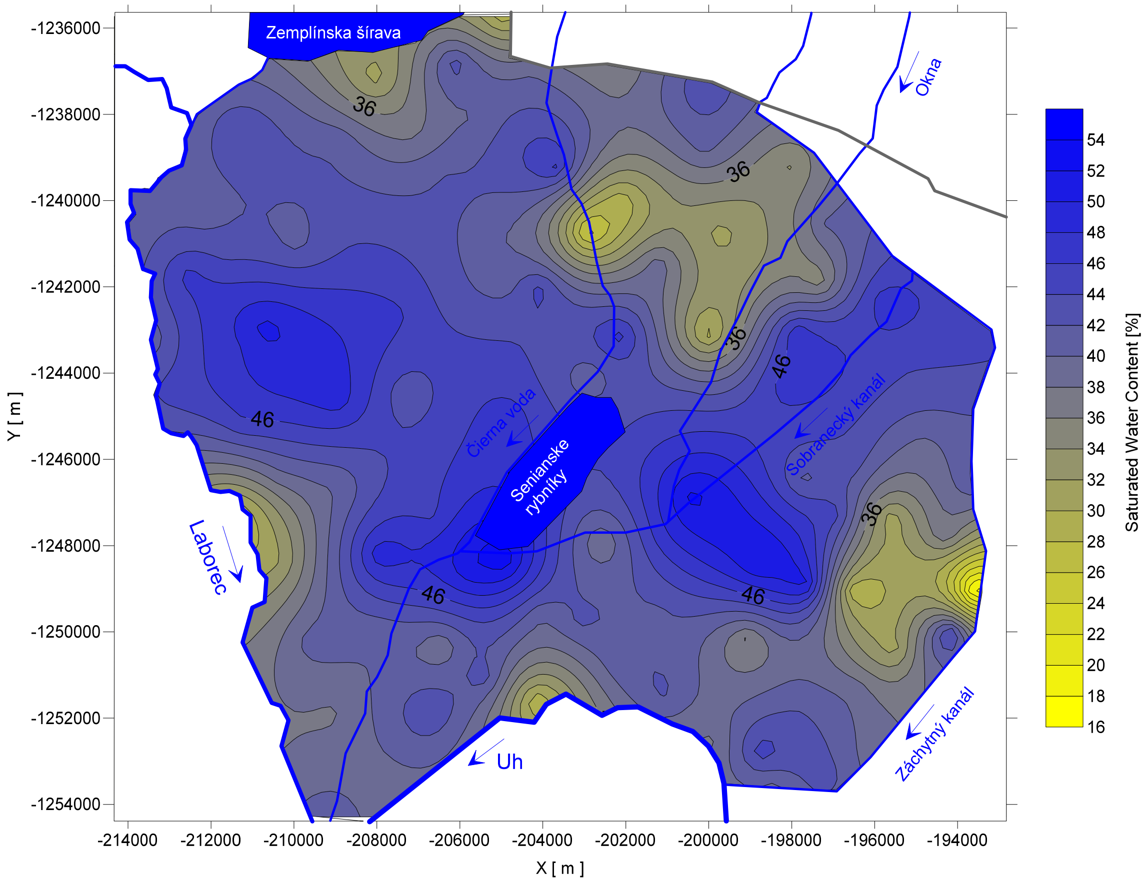

Fig. 5 Regionalization of depressed area of ESL according to soil´s water capacity

Areas with the highest values of Θ_s and hence the places with the highest occurrence of clays are found in the central part of the investigated area. This part is the depression of the investigation area with the lowest altitudes (see Fig. 1). This is given by the genesis of clays, which have their origin in the water meadows in depressed parts of the area. On the other hand, the places with the lowest values of Θ_s are located near rivers Laborec and Uh, and correlate to the content of sand fraction, which has its origin in the transport activities of these rivers. Map containing sand fraction is shown in Fig. 7.

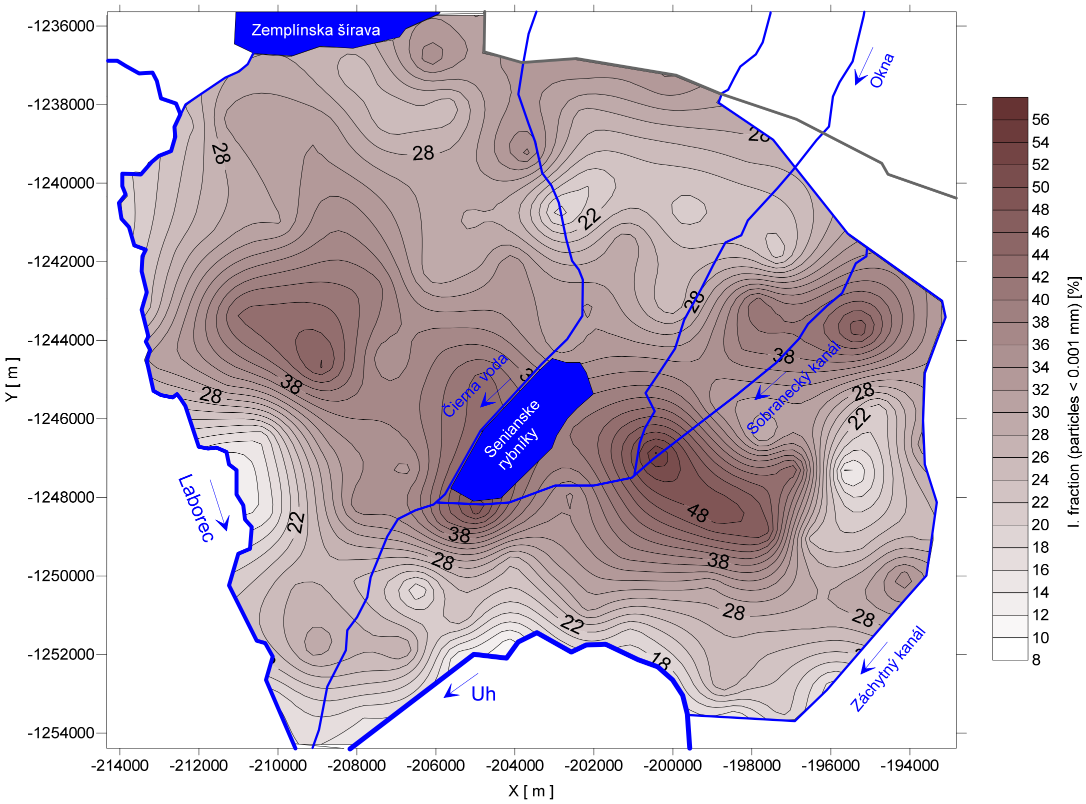

Fig. 6 Map of colloidal clay distribution (particles < 0,001 mm)

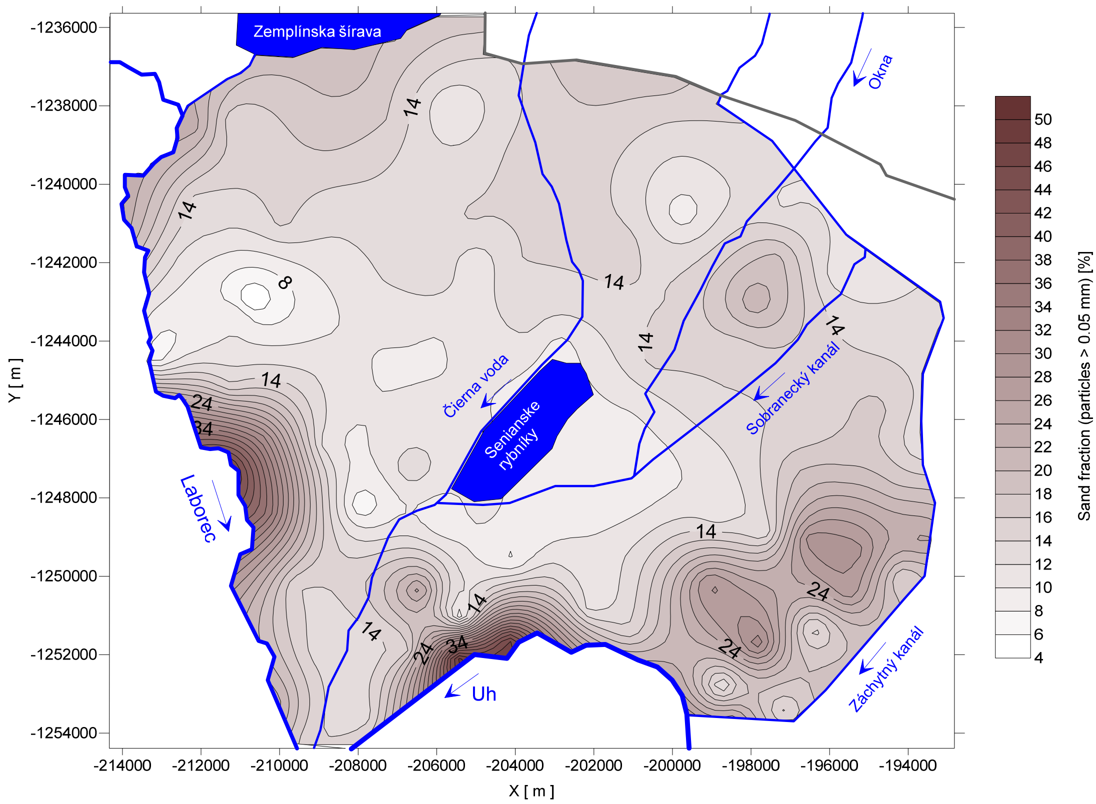

Fig. 7 Map of sand distribution (particles > 0,05 mm)

4 CONCLUSION

In recent years, a lot of research work is focused on the development of computational methods and procedures for effectively obtaining the necessary hydrophysical soil characteristics from the relatively easy obtained data, such as the data of soil texture. This paper gives an example of using pedotransfer function for regionalization of depressed area according to the value of saturated water content. For this research were picked up 106 disturbed soil samples from the predefined area of ESL. Based on the particle size analysis was used pedotransfer function for quantification of Θ_s. According the values of Θ_s was performed regionalization of investigated area of ESL. Areas with the highest values of Θ_s strongly correspond to a depression in the central part of the investigated area. The lowest values of Θ_s were identified in soils close to rivers, where dominates the sand fraction.

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CONSTRUCTED WETLANDS IN CROATIA – PROJECT OF STARO PETROVO SELO CONSTRUCTED WETLAND

D. Vouk [1], D. Malus [2], R. Van Deun[3], M. van Dyck[4]

Abstract

Constructed wetlands present alternative technological solution for wastewater treatment, applicable for smaller rural areas, as well as for industrial wastewater treatment. Low construction, operation and maintenance costs with simple operation and high treatment efficiency rank them among the more popular solutions of wastewater treatment worldwide. This paper in summary comprises a view on the application of constructed wetlands in Croatia. In more detail the constructed wetland Staro Petrovo Selo (800 population equivalents) will be analyzed with the description of the most relevant technical issues. The whole project of Staro Petrovo Selo constructed wetland is part of the international cooperation between Flemish government (Belgium) and Croatia (Project name: CEE/KRO/001/06 - "Sustainable extensive wastewater treatment in small communities in Croatia").

Keywords

constructed wetland, wastewater, treatment, Croatia, Staro Petrovo Selo

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1 INTRODUCTION

A constructed wetland (CW) is an artificially formed wetland used for treatment of wastewater running through it.
Through processes taking place in natural aquatic systems, constructed wetlands as complex integrated systems of interaction between water, substrate (porous filling), plants (wetland vegetation, etc.), animals, micro-organisms and environment (sun, soil, air) improve the water quality. By combination of physical, biological and chemical processes within the constructed wetland, waste substance is removed from wastewater.
Simple operation, high efficiency of treatment and comparatively low costs of construction, operation and maintenance (in relation to conventional treatment technologies) characterize CW as good and acceptable solutions of wastewater treatment. Their attractiveness is upgraded by aesthetic and ecological values (biological and landscape diversity of wetland habitats).

Fig. 1 Constructed wetland – aesthetically acceptable solution

Constructed wetlands are used primarily for treatment of household (sanitary) wastewater of minor communities distant from urban centres. They are also successfully applied in processing of industrial wastewater from farms, slaughterhouses, seepage water from plant nurseries and rainfall inflow from roads.
The advantages of constructed wetlands are receiving growing recognition in Croatia, as well as in other developing lands, resulting in more frequent application, not only for treatment of sanitary wastewater, but also for wastewater from industrial activities. So far, there are no exact guidelines for forming and sizing of constructed wetlands applicable at a wider level. Most countries have formed their own guidelines. Even in some countries different regions use different guidelines. A great number of different guidelines for forming and determining of the size of CW is summarized in numerous papers: [1], [2], [3], [4], [5], [6], [7], [8], [9], [10], etc.
This paper gives a review of application of constructed wetlands in Croatia, emphasizing the great potential of application in future. The paper will also present the design solution of the constructed wetland Staro Petrovo Selo, with the capacity of 800 PE, designed in the framework of international cooperation between the Flemish government (Belgium) and Croatia (Project: CEE/KRO/001/06 - "Sustainable treatment of wastewater in small communities in Croatia").

2 CONSTRUCTED WETLANDS IN CROATIA

In Croatia, application of constructed wetlands in wastewater treatment started about 10 years ago. So far, one CW has been built for treatment of sanitary wastewater of a constantly inhabited settlement (Žminj), two CW for treatment of wastewater of motoring camps (Bijar – island of Cres, Glavotok – island of Krk), and one CW for treatment of seepage water of the municipal waste disposal (Jakuševac, Zagreb). In comparison with other developing countries, and with developed countries, application of CW in Croatia so far can be assessed as negligible. In spite of the comparatively small number of existing CW in Croatia, it may be pointed out that a large number of CW is at present in the designing phase, plus some of them in the construction phase. The paper will give a summary description of basic properties of built and designed CW in Croatia.

2.1. Existing CW in Croatia

2.1.1 Motoring camp – island of Cres

CW for treatment of sanitary wastewater of the motoring camp Bijar (Fig. 1) has been built for the present capacity of the camp of 1,000 PE. In relation to the sensitivity of the recipient (sea), CW of the motoring camp Bijar is 2^{nd} degree of wastewater treatment. Treated wastewater is discharged into coastal sea in the immediate vicinity of the camp.
The results of analyses show high efficiency of wastewater treatment through reduction of the values of the following parameters: (Tab. 1): BOD5 99 [%], COD 97 [%], TSS 92 [%], anionic detergents 99 [%], total oils and fats [%] and bacteria of 99–100 [%]. According to Croatian legislation, effluent from CW of the motoring camp Bijar meets the limit values of wastewater emissions.

Fig. 1 Constructed wetland for treatment of wastewater of the motoring camp Bijar [11]

Tab. 1 Chemical and bacteriological parameters in wastewater at entrance, centre and outflow of the constructed wetland Bijar [11]

CW motoring camp BIJAR – Island of Cres		**MIN**			**MAX**			**AVERAGE**			**EFFICIENCY. (%)**
Date of sampling: from 29.06.to 31.08.2005.		INLET	AVER.	EFFL.	INFL.	AVER.	EFFL.	INFL.	AVER.	EFFL.	INFL. / EFFL.
CW capacity:	PE	272			782			498			
Air temperature:	°C	23,0			30,0			25,8			
Inflow:	m^3/d	13,5			73,5			54,4			
Water temperature:	°C	23,0	22,0	21,0	25,0	25,0	25,0	24,0	23,4	22,6	
pH	-	7,0	7,3	7,4	7,6	7,7	8,2	7,4	7,6	7,6	
NH_4:	mgN/l	71,06	16,70	6,15	109,14	69,22	51,69	91,07	48,10	21,76	77
NO_3:	mgN/l	0,95	1,99	5,32	2,74	18,67	33,52	1,41	9,73	22,35	
TKN:	mgN/l	78,79	29,12	7,67	123,48	72,69	52,98	101,32	51,80	24,41	76
TN:	mgN/l	80,00	41,93	15,15	124,94	79,70	65,87	102,74	62,52	47,04	55
TP:	mgP/l	9,22	2,95	2,68	14,59	13,07	10,58	12,20	9,14	7,19	42
Phosphates:	mgP/l	8,14	2,84	2,45	13,72	12,02	10,52	11,02	8,47	6,66	41
COD	mgO_2/l	305	15	5	996	58	27	579	32	17	97
BOD5	mgO_2/l	175	3	2	640	14	7	324	8	3	99
TSS:	mg/l	23,0	4,8	1,2	109,6	27,0	5,6	55,3	10,9	3,3	92
Anionic detergents:	mg/l	0,46	<0,10	<0,10	8,14	0,30	0,17	3,00	<0,10	<0,10	99
Total oil and fats:	mg/l	11,44	0,49	0,23	26,89	7,19	1,21	18,86	2,25	0,64	97
Total coliforms:	nc/100ml	460000	3500	4800	19300000	1640000	192000	7530000	264250	47920	99
Faecal coliforms:	nc/100ml	330000	1000	800	6300000	750000	78000	3604000	109360	14210	100
Faecal streptococci:	nc/100ml	90000	100	100	1100000	330000	10000	568000	37830	2568	100

2.1.2 Motoring camp Glavotok – island of Krk

CW for treatment of wastewater of the motoring camp Glavotok (Fig. 2) has been built only for the first phase and the capacity of 300 PE, while in the peak of the tourist season maximum load rises to about 750 PE.

Fig. 2 Constructed wetland of motoring camp Glavotok [11]

Therefore the results of chemical and bacteriological parameters in wastewater before and after treatment in CW indicate that the plant is overloaded (Tab. 2). Inflow of large quantities of wastewater, loaded with high concentrations of suspended solids, in particular towards the end of the tourist season, also indicate inadequate mechanical pre-treatment, which exerts additional load on the plant. Considerable increase of suspended solids in water at the entrance into the constructed wetland is noticed in samples at the end of August. Treated wastewater is not released into the sea, but is used as technological water for toilet flushing in the camp and irrigation of olive trees planted on the ground adjacent to the constructed wetland.

Tab. 2 Physical-chemical and bacteriological characteristics of wastewater at entrance, centre and outflow of the constructed wetland Glavotok [11]

CW motoring camp GLAVOTOK – Island of Krk		MIN			MAX			AVERAGE			EFFICIENCY. (%)
Date of sampling: from 29.06.to 31.08.2005.		INLET	AVER.	EFFL.	INFL.	INLET	AVER.	EFFL.	INFL.	INLET	AVER.
Capacity:	PE	300			751			534			
Air temperature:	°C	22,0			33,0			25,7			
Inflow:	m^3/d	14,0			29,9			26,2			
Water temperature:	°C	23,0	22,0	21,0	27,0	26,0	25,0	24,6	24,0	22,3	
pH	-	7,4	7,5	7,5	7,9	7,9	7,7	7,7	7,7	7,7	
NH_4:	mgN/l	119,22	145,77	23,24	221,48	204,62	179,76	183,34	175,83	119,03	37
NO_3:	mgN/l	1,41	0,81	0,17	4,10	2,95	27,74	2,37	1,35	3,83	
TKN:	mgN/l	131,94	155,51	27,83	297,98	223,16	181,94	216,16	189,75	124,37	43
TN:	mgN/l	134,66	156,33	30,30	302,09	224,11	182,15	218,54	191,13	128,53	42
TP:	mgP/l	8,40	11,42	2,75	30,68	17,67	16,25	18,76	14,88	11,96	35
Phosphates:	mgP/l	7,57	11,20	2,35	16,34	15,27	14,50	13,68	13,39	10,33	27
COD	mgO_2/l	538	264	76	2598	473	193	1238	390	116	88
BOD5	mgO_2/l	300	110	8	1136	290	60	597	196	25	94
TSS:	mg/l	58,0	28,7	2,3	1832,6	354,0	51,2	633,8	114,9	23,7	90
Anionic detergents:	mg/l	0,66	0,10	<0,1	7,88	2,51	0,75	2,59	0,94	0,26	92
Total oil and fats:	mg/l	13,42	3,51	0,85	139,31	12,03	3,09	53,86	7,00	1,65	94
Total coliforms:	nc100ml	200000	2400000	51000	21000000	12000000	3000000	10314000	6100000	991600	80
Faecal coliforms:	nc/100ml	180000	150000	18000	7200000	3000000	530000	3656000	1706250	212900	85
Faecal streptococci:	nc/100ml	91000	26000	2900	3500000	1300000	300000	985100	389500	55690	89

2.1.3 Constructed wetland Žminj – Istria

CW Žminj in Istria is designed for 750 PE, and consists of mechanical pre-treatment in the septic tank and downstream three pools of the constructed wetland with subsurface flow, in the first one with horizontal flow, the second with vertical flow, and the third with horizontal flow. The last pool in the line is used as a reservoir for treated water which is used for irrigation of the football ground. The total area of all three pools of CW Žminj is 1,700 sq. m. CW Žminj was built and put into operation in May 2003. The pools of CW Žminj are planted with *Phragmites australis and Juncus effusus*.

Fig. 3 Constructed wetland in community Žminj [12]

A part of the pools of CW Žminj is at present clogged, with surface flow occurring in some places, which results in reduced efficiency of the plant. The reason of clogging in inadequate mechanical pre-treatment, caused by its too small size, inadequate shaping and inadequate size of the distribution system at water inflow into CW pools.

2.1.4 Constructed wetland Vinogradci-Belišće

Constructed wetland Vinogradci treats sanitary wastewater of the community Vinogradci. The plant has been in operation since 2007. The total plant capacity is 300 PE. It consists of the mechanical pre-treatment in the septic tank of 22 cu. m. Wastewater from the septic tank flows into three interconnected pools of the constructed wetland with horizontal subsurface flow, of total area of 900 sq. m. (Fig. 4). At the outflow from the constructed wetland, effluent quality meets the legal standards.

Fig. 4 Constructed wetland Vinogradci

2.1.5 Constructed wetland Jakuševac

CW Jakuševac treats a part of seepage water at the municipal waste dump of the city of Zagreb. It comprises the area of 100 sq. m. (Fig. 5). It consists of the inflow basin with mechanical pre-treatment and three pools with horizontal subsurface flow serving for water treatment. Treated wastewater is released into the storage basin which has been turned into a lagoon by planting of vegetation. The lagoon has already become the habitat of several bird species.

Fig. 5 Constructed wetland Jakuševac (URL_2)

2.2 Designed constructed wetlands for wastewater treatment

In addition to operational CW mentioned above, other constructed wetlands currently in the designing or construction phase are listed below:

- Conceptual design of constructed wetland for treatment of sanitary wastewater Božava – Dugi otok, 1.000 [ES].
- Conceptual design of constructed wetland for sanitary wastewater treatment Gvozd, 1.700 [ES].
- Conceptual design of constructed wetland Osor and A/C Preko mosta.
- Conceptual design of constructed wetland for sanitary wastewater treatment Podgorač-Našice, 1.300 [ES].
- Conceptual design of constructed wetland for treatment of seepage water from municipal waste dump Jelenčići.
- Detailed design of constructed wetland of Teachers Training High School Petrinja.
- Conceptual design of constructed wetland for treatment of sanitary wastewater Pagubice-Istria.
- Construction design of constructed wetland for treatment of sanitary wastewater Jasenovac, 1.300 [ES] – plant in construction.
- Detailed design for treatment of sanitary wastewater KP Valtura, 200 [ES].
- Conceptual design of constructed wetland for treatment of sanitary wastewater Vižinada-Istria, 350 [ES].
- Detailed design of constructed wetland for treatment of sanitary wastewater Staro Petrovo Selo, 800 [ES].
- Detailed design of constructed wetland for treatment of sanitary wastewater Sv. Ilija, 800-1000 [ES] – plant in construction.

3 CONSTRUCTED WETLAND STARO PETROVO SELO

3.1. General

The paper discusses the constructed wetland as optimum technological solution for treatment of wastewater (WWTP) for a part of the community Staro Petrovo Selo, situated in the western part of the County Brodsko-Posavska, east from the town Nova Gradiška, along the highway A3 (Zagreb-Lipovac). The constructed wetland has been designed in the framework of the program of international cooperation between the Flemish government (Belgium) and Croatia (Project: CEE/KRO/001/06 - "Sustainable efficient treatment of wastewater in small communities in Croatia").

In the present situation, there is no public sewerage system in the community Staro Petrovo Selo, and wastewater is managed by means of collecting or septic tanks. Based on preliminary analyses, the optimum concept of wastewater disposal in the municipality of Staro Petrovo Selo was defined, characterized by decentralized sewerage system with a number of minor autonomous systems with corresponding WWTP. The estimated capacity of WWTP analyzed in this paper is 800 PE. With regard to the large number of influential factors (WWTP capacity, field conditions, availability of free space, costs and simplicity of construction, operation and maintenance, etc.), the constructed wetland with horizontal subsurface flow (CWHSF) was selected as the optimum technological solution. The basic technical elements of the designed solution of the constructed wetland Staro Petrovo Selo will be described hereafter.

3.2 Technical solution of the constructed wetland

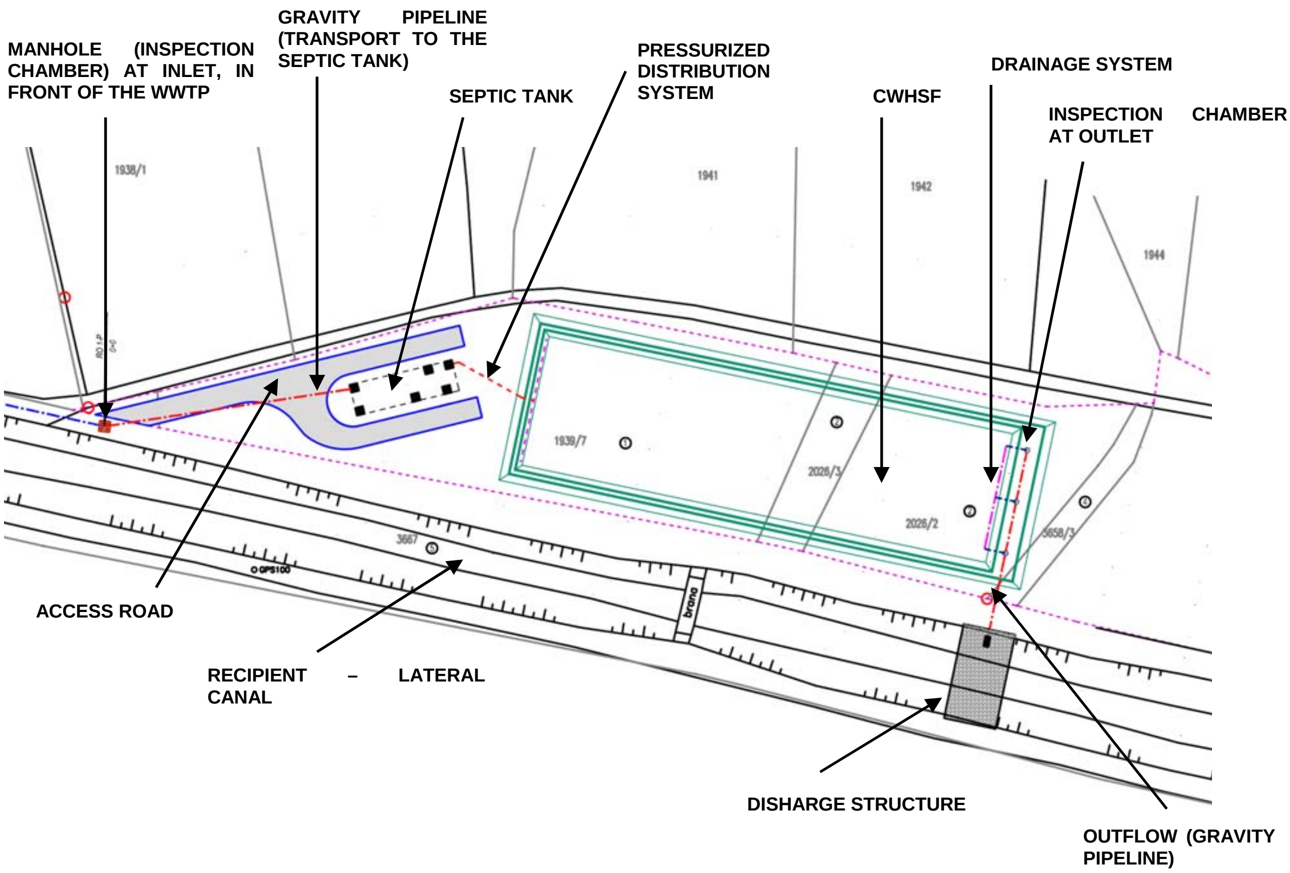

Fig. 5 Situation of the constructed wetland Staro Petrovo Selo

3.2.1 Pre-treatment

3.2.1.1 Inlet pipeline

Wastewater is brought to the septic tank by the gravity pipeline from the last (downstream) manhole planned on the public sewerage system. The profile of the pipeline is DN 400, and the total length is 48 m.

3.2.1.2. Septic tank

Outer dimensions of the septic tank are 20.2x6.3x4.5 m, and the total tank volume is approx. 380 cu. m. while the planned operating volume is approx. 290 cu. m. The operating volume of the septic tank is divided into three chambers, the first with the operating volume of approx. 178 cu. m. and the other two with approx. 56 cu. m. each. The flow between chambers is planned through openings in partition walls of 0.24 sq. m. (0.8x0.3 m), situated 1.6 m above the tank bottom. The retention time of wastewater in the septic tank is 2 days.
The septic tank will be constructed completely as a reinforced concrete structure with four chambers. The first three chambers will act as pre-treatment, and the fourth represents the pumping chamber.
The septic tank will be equipped with 5 (revision) manholes for efficient maintenance and emptying of the contents (separated sludge and floating solids). Openings of manholes are 1.0x1.0 m.

3.2.1.3. Pumping station

With regard to field conditions (altitude of the septic tank in relation to CW), it is necessary to interpolate a pumping station after the septic tank to pump preliminarily cleared wastewater to CW. Therefore, construction of the pumping chamber is planned as the fourth chamber within the septic tank. The bottom of the pumping chamber will be at the same level as the bottom of the septic tank. The dimensions of the pumping chamber are 1,0x5.5x3.8 m and the total volume is 16.5 cu. m. The operating volume is 2.2 cu. m. which means that operating height between pump switching on and off is 40 cm. On top of the pumping chamber, there is a manhole with the opening of 1.0x1.0 m. The septic tank is connected with the pumping chamber by a preshaped T-element of DN 300 profile. There will be two submerged pumps in the chamber (operating + spare) of equal capacities of 5.0 l/s. The pump power is 1.4 kW.

3.2.2. Constructed wetland

3.2.2.1 Water distribution system

The selected concept solution of the plant includes inflow of preliminarily cleared wastewater under pressure to CW, which ensures efficient distribution of wastewater inflow over the entire width of CW. The pumping station in the septic tank will be connected to the constructed wetland by the pressure pipeline DN 90, 17 m long. The pipeline is brought through the side wall of the inflow part of CW and connected to the distribution pipeline. The distribution pipeline is laid over the entire width of CW, perpendicular to the direction of water flow through CW. The pressure inflow pipeline and the distribution pipeline are connected in the middle of the distribution pipeline, which has two horizontal “arms”, each 12 m long. The distribution pipeline will be made completely of PEHD, profile DN 90. On the bottom of the distribution pipeline 13 holes will be drilled on site, of 15 mm diameter which will let pre-treated wastewater into CW. Also, 4 manholes will be installed on the distribution pipeline, of the same material as the pipeline itself, and on the surface side the manholes will be closed by specially shaped plastic lids. The distribution pipeline is laid in the surface part of the inflow part of the plant.

3.2.2.2 Constructed wetland body

CWSPS is shaped as a shallow pool with horizontal flow of wastewater through porous substrate of corresponding particle size. The entire pool of CW will be filled with porous substrate.

The constructed wetland, as designed, will be 90 m long, 25 m wide, with depth ranging from 0.8 m (inflow part) to 1.25 m (outflow part). The slope of side walls of the constructed wetland is 1:1, and the gradient of the pool bottom is 0.5 percent.

The entire pool of the constructed wetland, including the sides is lined with PEHD watertight geomembrane 1.5 mm thick. Additionally, to protect the geomembrane it will be lined on both sides with geotextile, thickness 200 g/sq. m. Substrate is placed on top of the geomebrane and geotextile.

The body of CWSPS consists of three characteristic parts – inlet zone, central filtering zone, and the outlet zone.

The inlet zone, 0.8 m deep, is filled by coarser substrate – rock (32/64), and the distribution pipe is placed in the surface part, additionally covered by the protective substrate layer of 30 cm, of the same particle size. The inlet zone of CW is 5.0 m long.

The central zone of CWSPS is characterized by river gravel of particle size (4/8). The total length of the central zonet is 82 m. As regards wastewater treatment, the central filter zone is the most active part of the plant.

The outlet zone of CW, 3.0 m long, the entire depth (1.25 m) will be filled by coarser substrate (32/64)

The entire area of the CW pool will be planted by reed seedlings (Phragmites australis). With regard to the total area of the plant (2250 sq. m.) the planned number of reed seedlings is 13,500 – 6 seedlings per sq. m. In the region, reed is an autochthonous species, and problems regarding its growth and development are not expected.

3.2.2.3 Drainage and outflow pipeline

In the outlet zone of the constructed wetland, in the drainage layer of river rock of particle size 32/64, the drainage and outflow pipe will be installed. The drainage pipe is designed to be made of PVC pipes of the profile DN 160. The drainage pipe is laid at the bottom of the outlet zone along the entire width of the wetland, in the total length of 24 m.

Three aeration pipes, profile DN 160, will be connected to the drainage pipe. All three aeration pipes will be laid along the sides of the plant body and closed on the upper side by perforated covers. At three points, the drainage pipe will be connected to outflow pipes (PVC DN 160) which transport treated water out of the plant body. Each of the three outflow pipes is connected to the level control chamber, by installing the bend on the outflow pipes at 90°. A flexible plastic pipe will be installed on the bend which will make it possible to regulate the water level in the plant without personnel having to enter the chamber.

The level control chamber will be interconnected by outlet pipeline PEHD DN 300, which will release treated water into the recipient.

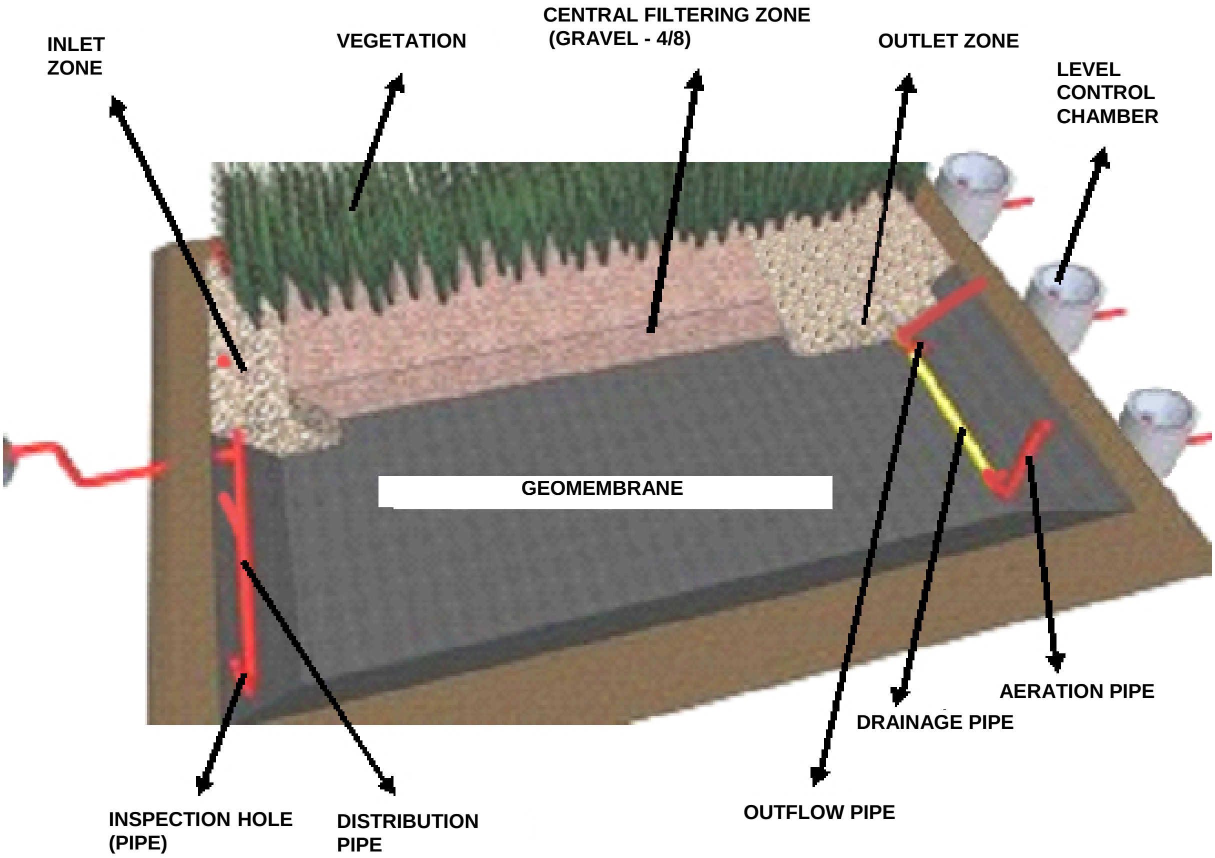

Fig. 5 Diagram – cross section of the body of constructed wetland Staro Petrovo Selo

4 CONCLUSION

From the viewpoint of power consumption, technology and economy, constructed wetlands are efficient solutions for treatment of sanitary wastewater for minor communities with sufficient available space in zones with moderate climate. With this technology it is very easy to achieve the effects of the second degree of treatment.

A particularly favourable circumstance is that the operation and maintenance costs are considerably lower than with conventional treatment methods, which is exceptionally important for rural local communities with low economic potential.

With respect to all advantages and disadvantages of constructed wetlands, and to the experience of many countries worldwide using CW for many years, it is possible to point out the great potential of their application in Croatia.

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FLASH FLOOD AND EROSION PREVENTION, PROTECTION AND MITIGATION MEASURES IN SENSITIVE AND PROTECTED AREAS

Nevena Dragičević[1], Barbara Karleuša[2], Nevenka Ožanić[3]

Abstract

This paper gives an overview of the research related to flash flood and erosion prevention and mitigation measures conducted on the Dubračina River catchment area in Vinodol Valley (Croatia), with special emphasis on its sub-catchment Salt Creek. The purpose of this paper is to show the possibilities of applying different measures focused on prevention and mitigation of flash floods, with emphasis on those with long-term effect on erosion and landslide affected areas that can be applied on entire catchment area and/or riverbed itself. The combination of extreme rainfall event can potentially form the flash flood, followed by erosion sediment transportation and activation of landslides in the area of Dubračina River, including torrential tributaries. Such event could present a serious threat to downstream areas, especially to the center of the town Crikvenica.

Keywords

torrent, erosion, landslides, protection and mitigation measures, flash floods

1 INTRODUCTION

The torrents are permanent or occasional streams whose characteristics are: highly variable discharges, high slope gradients of the bottom, high scouring activity, transport and deposition of sediment and frequent changes of channel dimensions. They are often followed by erosion processes and as its result, downstream erosion sediment transport. All of these can potentially trigger landslides in the area. The main characteristics of torrents are their

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tendency to reach the maximum discharge extremely rapidly as well to, subsequently, decrease it again with equal speed [1, 2]. So, in the aspect of flood management, flash flood comes really high on the list of priorities for prevention and mitigation development strategy. Flash floods are short-term events that occur within 2 to 6 hours after the beginning of intensive precipitation and are characterized by a sudden increase of water level, flow and velocity that often end with significant material and environmental damage [3, 4]. In the last twenty years, progressive urbanization often accompanied by more intensive land use, especially in areas prone to flood, erosion and landslides, caused an increasing need for effective hazard protection [5].

All over the world, extreme rainfall have become more frequent, thus causing the increased number of flood events with catastrophic consequences, human, material and environmental. According to the World Meteorological Organization (WMO), the most lethal form of natural hazards are flash floods, which tend to occur frequently but at relatively small scale [6, 7]. To reduce such consequences, flood management, as well as erosion management, needs to be based on structural and non-structural flood protection and mitigation measures. Structural measures are considered traditional engineering measures used in prevention against flash flood and erosion. Traditional approaches most commonly use civil engineering solutions such as barriers and dykes to protect the local population from flood and its consequences (economic as well as social). Sustainable drainage systems, used for storm water retaining and reducing the water runoff, especially in urban areas, are becoming more and more favourable as flood protection measures. Unfortunately, the main disadvantage of structural measures is their significant effect on the environment. Besides that, their durability, substantial construction and maintenance expenses, are also considered to be a disadvantage. They also provide false sense of security from flood and erosion to local population, making them less cautious in time of imminent danger [2, 8]. Dams that can be formed by landslide debris can partially or even completely block water flow in river channels. Such dams, can last from several minutes during the rain event up to thousands of years (Fig. 1).

Fig. 1: Example of “temporary” dams caused by combination of natural (sediment transportation) and anthropogenic (waste disposal) at Duboki river sub–catchment in Dubračina river catchment area

Ones that are formed and destroyed during one rain event can potential cause or/and intensify flash flood and aggravate flooding in the catchment area. Because of their unpredictability and short term duration it is very difficult to plan mitigation measures against them [9].

In this paper the overview of the research related to flash flood and erosion prevention and mitigation measures conducted on the Dubračina River catchment area in Vinodol Valley (Croatia) will be given. The area of Vinodol Valley is an extremely valuable natural and cultivated landscape, with its biodiversity, cultural and historical heritage, which provided to the Valley the status of a protected area with great importance for Primorsko – Goranska County so the selection of flash flood and erosion prevention and mitigation measures is even more complex but also very limited. Dubračina consists of many tributaries with torrential characteristics (Duboki, Bronac, Cigančica, Leskovnik, Sušik, Ričina Tribaljska, Pećica, Slani Potok, Mala Dubračina, Kućina, Malenica). The area is also known for land instabilities, such as landslides and erosion affected areas, whose processes are intensified due to the combination of high annual precipitation, its significant seasonal rainfall oscillations, flash floods, anthropogenic influences and geological characteristics. The most endangered area by erosion and landslide processes is the catchment area of the tributary Salt Creek where some parts of surrounding villages are already endangered.

The combination of extreme rainfall event at the area of Dubračina River can form flash flood, followed by erosion sediment transportation and even activation of landslides. Such events would present a serious threat to downstream areas, especially to the center of the town Crikvenica.

2 DEVELOPMENT OF NEW APPROACHES TO FLASH FLOOD AND EROSION PREVENTION AND MITIGATION

Today, increasingly gaining on its importance in flood management are nonstructural measures (early warning system, appropriate land use promotion, education, flood and erosion insurance, land use spatial planning, encouraging public participation ..) that are applied for flood mitigation purposes and often don`t have exclusively the prevention function. They have proven to be very effective for both short and long term flood risk and flood damage reduction. In comparison to structural measures, non-structural measures are sustainable, require much less capital investments, and their impact on the environment is negligible or minor. The biggest deficiency of these measures is their dependence on the cooperation of the population with organized network of state institutions in order to achieve their efficiency [2, 4, 10].

Rapid occurrence and spatial dispersion of areas in threat of flash flood are considered to be closely related to the impossibility of precise flash flood forecast and the inability to issue well timed flood warning to local inhabitants [11]. Preparedness on flood related hazards is essential in order to reduce its impact on the society. Today, the development and implementation of early warning systems, which is one of nonstructural measures, are encouraged. They are based on numerical weather predictions and their purpose is to detect flood events with sufficient advance in time in order to issue the flood warning. They are used to detect flash floods, debris flows, mud flows, rainfall-induced landslides, river floods and coastal floods [12]. The definition of early warning is “the provision of timely and effective information, through identified institutions, that allows individuals exposed to hazards to take action to avoid or reduce their risk and prepare for effective response” [13].

An example of an early warning system (EWS) can be found on the web platform Meteoalarm (14) where issued meteorological warnings of (Fig. 2) 33 countries through

Europe are collected. Although these warnings are general for entire country and its counties and do not provide detailed information. Among the first EWS were the Extreme Rainfall Alert of the British Flood Forecasting Centre (FFC) and Swiss warning system for point precipitation that are designed to give early predictions of upcoming rainfall events that can potentially cause flooding [13].

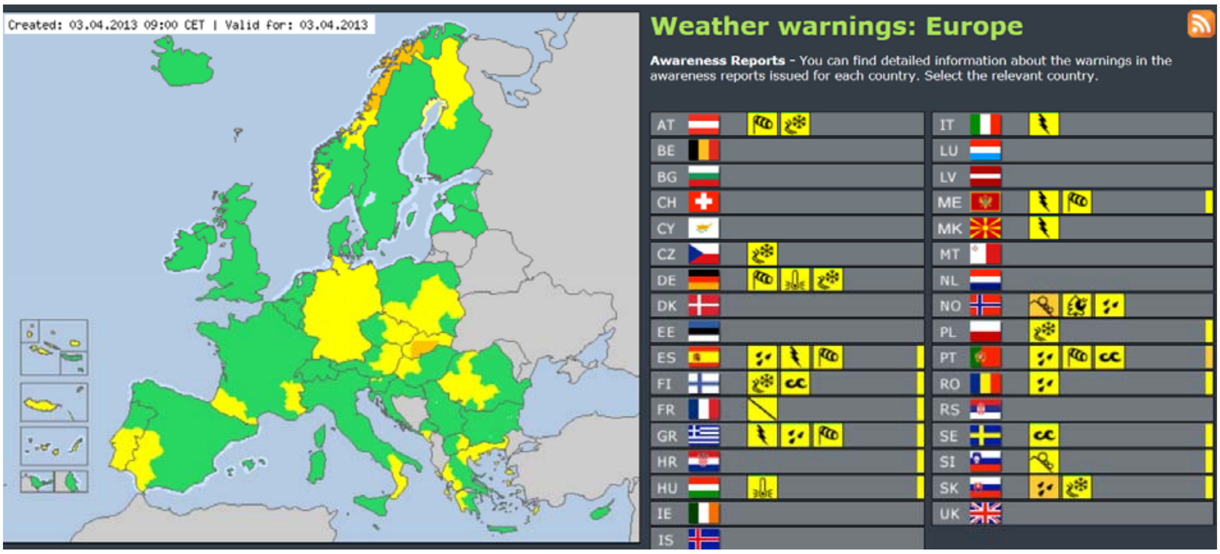

Fig. 2: Joint Weather Information System for weather warnings issues [14]

Activation of landslides at flash flood prone areas, intensive water erosion processes, combined with the devastating flash floods present a problem mostly caused by anthropogenic influences. Protection measures focused on flash floods, erosion and land stability from the aspects of water management are selected based on several criteria, of which are particularly significant: the degree of vulnerability and sensitivity of analyzed area, the importance and endangerment of water structures and water management systems as well as the estimated cost of the investment. Development of forecast methodology is not enough to reduce consequences of flash flood which are considered to be complex hydrometeorological and sociological phenomena [6, 15]. All of this is even harder in the area characterized by karst land where hazards that include landslides, debris flow, flash floods mixed with erosion have a higher meaning. Underground aquifers of unknown dimensions and hierarchical network of groundwater flow paths characterize karst. Areas with agriculture and land use changes tend to degrade the landscape and its vegetation. So, such combination, high quantities of groundwater flow, rapid surface flow, degraded vegetation and erosion affected areas, increase the possibility of recurrence of floods and its morphological consequences [16]. There are several different elements that make the flood risk managements complete (Fig. 3): prevention of damage caused by flood, protection against flood using structural and nonstructural measures, long-term actions toward flood preparedness by information exchange within local government and local population, development of emergency response plans for each catchment area endangered by flood and recovery actions focused on returning to normal life in the shortest period of time and applying learned lessons from the past flood disasters on future mitigation and prevention strategy against such events [13]. Based on that, successful flood management depends on a proper selection and combination of appropriate structural and non-structural measures, based on the characteristics of the research area, its physical and morphological characteristics, economic, social, political and environmental conditions [6, 15].

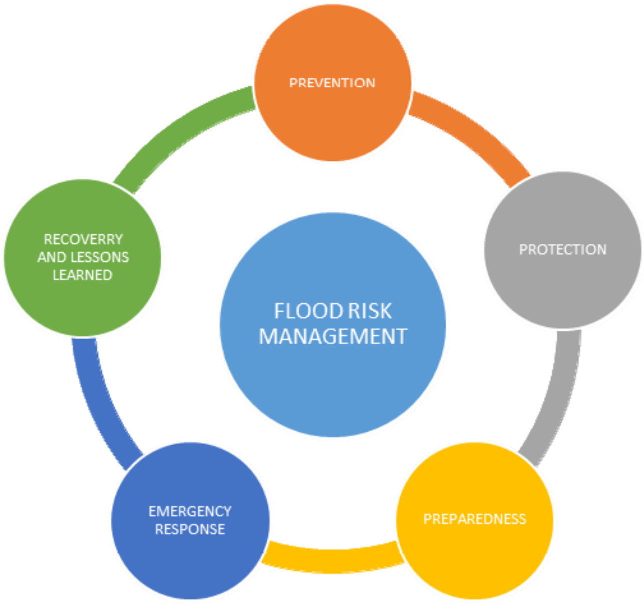

Fig. 3: Flood risk management elements

2.1 Hazard prevention and mitigation management in Croatia

Croatian laws and regulations as a prevention and mitigation measures for flash flood control specify actions that fall into the category of structural measures with characteristics of erosion protection and river bed stabilization. Such works are protection barriers, river regulation construction and maintenance of structure with water protection purpose, reforestation of catchment areas, cultivation and maintenance of protective vegetation as well as clearing of vegetation on required areas, removal of sediment from waterbed, construction and maintenance of structures for prevention and mitigation of erosion and flash flood, prohibition and limitation for excavation of sand, gravel and stone, etc. [1, 5, 17]. According to Croatian Water Act [1] anti-erosion measures include various legislation measures, education of population regarding problems of erosion and flash floods, systematic monitoring of erosion processes, formation of databases about erosion affected land and applied anti-erosion measures, integrating erosion protection measures in spatial planning, and so on.
In Croatia a project “Risk Identification and Land-Use Planning for Disaster Mitigation of Landslides and Floods in Croatia”, is conducted. Its aim is to combine landslides, erosion and flash flood hazards. Within the project there are many activities that include flood, flash flood, mud flow, debris flow, erosion and landslide research and analysis. Their purpose is the establishment of monitoring system that makes a foundation for development of the EWS for such hazards. Such system needs to encompass the specific hydrological and geological conditions of research areas where their implementation is planned, define hazards zones, hazard sensitivity assessment, hazard scenario development, hazard risk identification and many more activities in order to provide these protection and mitigation long-term measures its sustainability and success [4, 10, 18, 19, 20]. One of the project final objectives is to make protocols and guidelines for successful management of torrential catchment areas prone to flash flood, erosion and landslides. To make such versatile project successful, experts and

scientists from different fields are collaborating within four different working groups. The role of national government, regional government and especially local government and local inhabitants in decision making processes that leads to a final selection of the most acceptable prevention, mitigation and protection hazard measures and their future implementation, mustn't be diminish. Thus, one of the most important aspects that needs to be included in a project of this size is collaboration of scientists and experts with those institutions and especially local population (Fig. 4). For that, through the duration of the project local population risk awareness, hazard perception and their knowledge about measures against hazards are surveyed and analyzed [4, 10]. This is essential in order to determine vulnerability of a population to hazards as well as to enhance decision-making processes and eventually decrease population vulnerability to hazards by increasing effectiveness of implemented measures [4, 21].

Figure 4: Information about public presentation and photos from public presentation of Project aims and objectives to local population at Dubračina River catchment area – sub-catchment Slani Potok

Within this project at the Faculty of Civil Engineering University of Rijeka three research areas are analyzed:

- Catchment area of Dubračina River with emphasis on sub–catchment Slani Potok
- Catchment area of Rječina River with Grohovo landslide
- Catchment area of Mošćenička Draga

All three areas are specific and in some way different in its hydrological and geological characteristics, but one characteristic that is common to all of them is the karstic terrain, torrential attribute of rivers and their tributaries, and most importantly the presence of risk from flash flood, erosion and land instabilities. In this paper problems of flash flood, erosion and landslide risk at the research area of Dubračina River, which is also a sensitive and protected area, will be further detailed and explained.

3 FLASH FLOOD AND EROSION PREVENTION, PROTECTION AND MITIGATION APROACH IN DUBRAČINA RIVER CATCHMENT AREA – SALT CREEK

Vinodol Valley is a separated geographical entity in the eastern Kvarner area of Croatia. It is located between the Bakar Bay in the northwest and southeast of the town Novi Vinodolski, just in the hinterland of the town Crikvenica (Fig. 5) [19, 22]. This area is extremely valuable natural and cultivated landscape, with its biodiversity, cultural and historical heritage, which provided to the Valley the status of a protected area with great importance for Primorsko – Goranska County.

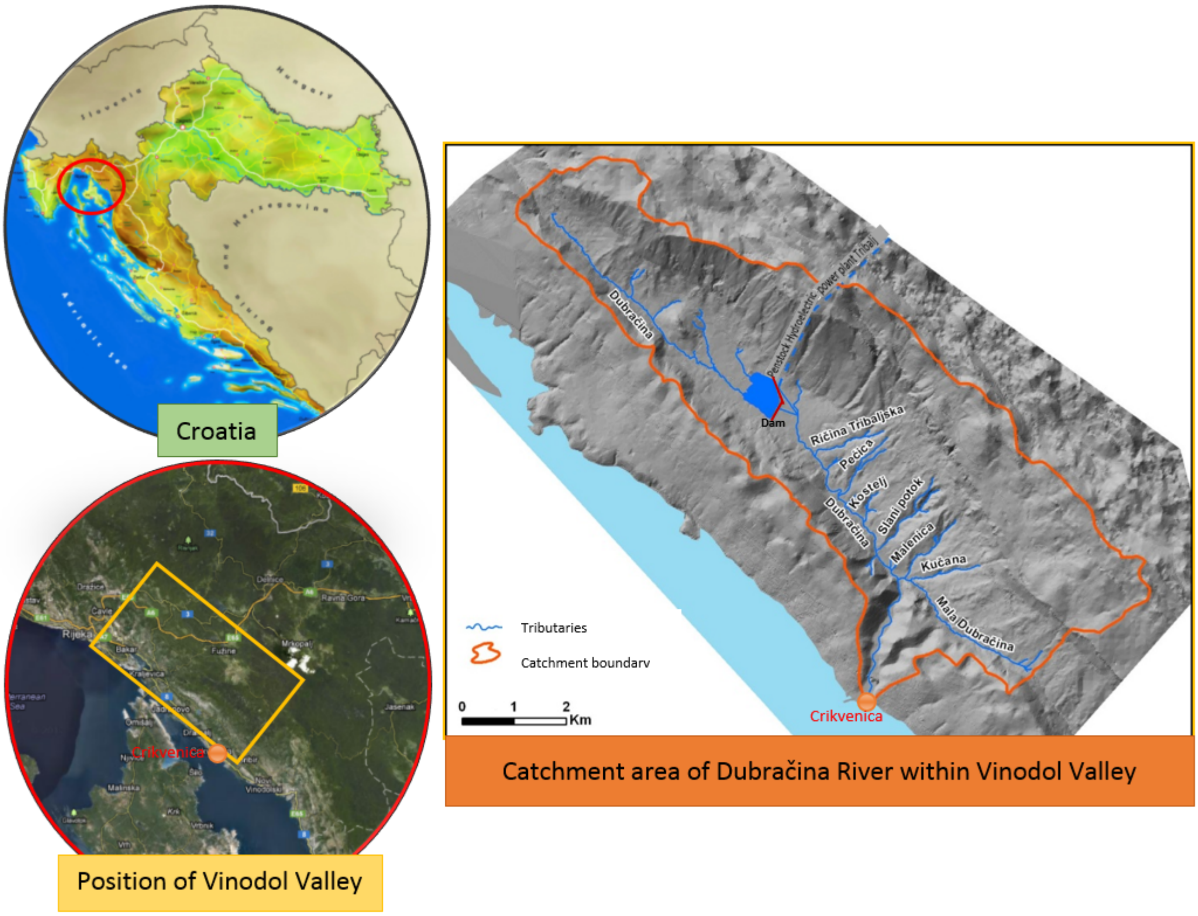

Fig. 5: Position of Vinodol Valley [23]

In the area, there have been recorded dozens of rare endangered and sensitive species of Croatian flora, as well as a large number of protected and endangered animal species. This area is known for significant changes in its vegetation cover, caused by the action of intensive anthropogenic influences over the centuries, which led to the negative effects on communities of natural forest in Vinodol Valley. Before the human impact, Vinodol Valley was covered by various types of dense forest. Today, it consists of a various types of grasslands, rocky pastures with partial denuded and erosion desolated rocky slopes. In recent decades, the neglect of agricultural land, has led to gradual return of vegetation to its original state as such allowing better protection of soil form erosion.

Within the Vinodol Valley Dubračina River catchment area is situated (Fig. 4). This catchment area consists of many tributaries with torrential characteristics. Most of the torrential tributaries of Dubračina river catchment area tend to dry out during the summer period, but in the rainy season have substantial oscillation in flow (Fig. 6). All its tributaries, along with Dubračina River itself in the combination with erosion processes can contribute to considerable flash flood events with devastating consequences, especially for lower parts of catchment area. During such event, the most endangered part would be the centre of the town Crikvenica that has the most high population density. The area of Dubračina River is mostly karstic terrain where infiltration of precipitation through the network of underground cracks is common. Unfortunately this leads to gradual degradation of terrain and its decomposition making already affected areas with erosion and landslides even more unstable.

One of the tributaries of Dubračina River catchment area with specific catchment characteristics is Slani Potok stream (Fig. 4) with the area of approximately 2 km^2. The lower part of Slani Potok catchment area is covered with flysh sediments (mud stone) while the upper part of it is mostly karstic plateau. This area is known for erosion processes and on it is the biggest surface affected by excessive erosion, with dimensions of 600 m in length and 250 m in width. The size of affected area is approximately 3 km^2, which is more than the catchment itself (Fig. 7) [19, 22]. Data about erosion threatened areas around Salt Creek stream catchment area date form the late 19-th century, with records of combined biological and technical interventions of forestation around torrent and landslides in the area of Drvenik, Grižane and Bribir. Restoration of erosion affected areas was conducted on several occasions, and it included maintenance of structures on streams, whose purpose was to reduce the speed of water flow. These measures, however, did not prevent further increase of erosion surfaces (Fig. 8). The danger on the catchment of Slani Potok presents the possibility of instability approaching villages in northern areas and as thus presenting the threat to downstream areas. Some parts of villages are already threatened by erosion, particularly those in the Slani Potok catchment. Proposed measures, such as supporting and improving restoration measures for erosion bases, ensuring the maintenance and improvement of existing anti-erosion systems, prohibition of new content due to geological sensitivity of area, monitoring and research of erosion processes, protection of cultural and historical valuable structures from erosion, torrents and floods, were proposed in the Spatial Plan of Vinodol Valley, but without any further elaboration [24, 25, 26].

To achieve success in flood, landslide and erosion management, basically in any natural hazard management, measures should be planned for long-term period of time. These measures should include both structural and non-structural measures. Unfortunately, in sensitive areas and in areas of significant importance it is not possible to base the protection exclusively on structural measures, because of their permanent impact on nature. In such cases and on such areas, hazard protection needs to be based on prevention of hazards and then on its mitigation. Such approach must encompass primarily non-structural measures whose impact on nature is almost insignificant, and only than those structural measures that are the most necessary. Final objective of such approach is to reduce the impact on nature in sensitive areas to minimum.

Fig. 6: Torrential tributary Slani Potok – seasonal fluctuations in water flow

For the successful flood and erosion management on Dubračina River catchment area, which falls within sensitive areas and areas of special significance, it is essential and recommended that measures proposed by Spatial Plan, are also complemented by measures such as public involvement; implementation of flood and erosion risk prevention and mitigation actions before, during and after hazard within educational institutions; establishment of continuously and long term monitoring of erosion processes and flow velocity with development of early warning system in the future.

Fig. 7: Erosion base in the upper part of catchment area of Slani Potok

Fig. 8: Result of erosion processes and soil instability on the road between sub-catchments Slani Potok and Mala Dubračina

Some of these measures, mostly at Slani Potok, are planned and partially implemented as a result of activities conducted within the project "Risk Identification and Land-Use Planning

for Disaster Mitigation of Landslides and Flood in Croatia". Flow velocity, water level, water temperature, air temperature, precipitation, insolation, and many other parameters are being measured at the area of Slani Potok for over a year and a half. Based on the information's obtained at Slani Potok and their analyses, hydrological and erosion modelling will be developed for the catchment area on Slani Potok. All the tributaries of Dubračina catchment area have many similarities, such as torrential characteristics, water flow seasonal oscillation, geomorphological characteristic, with the exception of erosion affected areas that is mainly present on lower part of Dubračina River catchment area, from the dam downstream (Fig. 5). Based on their similarities, models obtained at Slani Potok will be applied at all the other tributaries in order to define their influence in flood occurrence and flood and erosion hazard occurrence, overall and individually. All this information's are relevant in order to achieve the final objective – establishment of early warning system for flash flood and erosion for the area of town Crikvenica, as well as all the endangered villages at Dubračina catchment area.
The research around the world has shown that public participation and risk awareness is essential in overall struggle against hazards. So, for the purpose of better decision making in the process of mitigation and prevention measures selection, as well as thorough historical, present and future comprehension of processes at researched area, that can contribute to hazard development, local population and government needs to be involved. For this reasons, local population at the area of Slani Potok has been surveyed regarding the problems of erosion and flash flood, risk awareness, knowledge of protection and mitigation measures as well as their interest in the subject. For the entire duration of the project public participation is encouraged by various public presentations at local communities, local and national television reports, local and regional newspaper reports as well as direct communication with local residents. All of these is important and is contributing to a better information exchange between local government, scientists, professional experts and local population. In order to achieve better results in the time of potential hazard threat, it is necessary to expand those measures on the whole Dubračina River catchment area [4, 10, 18, 19].

4 CONCLUSION

There are many factors that can contribute to a potential risk from flash flood and erosion hazard. Maybe the most significant are anthropogenic influences on nature. Since urban development can hardly be detained, the influence of a human society can be minimized by public participation in decision making processes regarding the problem in their area, through regular and well-timed information exchange between local government and local inhabitants. In order for any hazard management to be effective, collaboration between government, scientists, professional experts and local population is necessary.
It is especially important in sensitive areas and areas of significant importance to base protection not only on structural measures, but mainly on non-structural. In such way, the structural measures should be reduced to minimum, but not necessarily excluded, in order to minimize the impact on nature.
Today, more and more emphasis has been given on the establishment of an early warning system as a mitigation measure for flash flood and erosion hazard. Dubračina river catchment area is a perfect example of an area of significant importance both from the aspects of biodiversity, cultural and historical heritage, valuable natural and cultivated landscape, erosion affected areas and torrential characteristics of its streams. Because of all that, establishment of an early warning system in this area would quite contribute to mitigation of consequences from flash flood and erosion, both for nature and for society.

Acknowledgements
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THE ROLE OF DISTRIBUTED MODELLING IN THE ASSESSMENT OF THE IMPACT OF CHANGES IN LAND USE ON RUNOFF

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Abstract
Changes in runoff generation due to changes in land use, particularly those connected to agricultural and forest management, have often been documented in the literature. Distributed rainfall-runoff model simulations are often used to evaluate the impact of such changes on runoff generation. These models have the advantage of reflecting the effects of land use in spatially distributed model parameters. In this paper, simulated changes in the runoff regime in the Laborec River basin due to land use changes were estimated using the FRIER model. The parameters of the model were estimated using climate data from 1991-2000 and from three digital map layers: a land-use map, soil map and digital elevation model. Several scenarios of changes in land use were prepared, and the runoff under the new land use conditions was simulated. Long-term mean annual runoff and its components under the previous and changed land uses were estimated and compared. Limitations of the use of distributed models for estimating land use changes were discussed.

Keywords
Scenarios of changes in land use, distributed rainfall-runoff modelling, the Laborec River basin.

1 INTRODUCTION

Both the landscapes and river systems have undergone major changes in many regions in the world in the past. The pressure from the rapidly increasing population in many rural areas has often caused changes in terms of deforestation, reclamation of wetlands, etc., with the aim of

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increasing agricultural production. The effects of land use changes on the hydrological responses of catchments, particularly those connected to forest management, have been documented for smaller watersheds. The removal of forest cover is known to increase stream flow as a result of reduced evapotranspiration and increase peak flows as a result of changes in interception, infiltration, surface roughness and higher water tables.

The effect of land use on runoff generation is very complicated; as land use and soil cover have an effect on interception, surface retention, evapotranspiration, and resistance to overland flow. Due to the complexity of the processes involved, the magnitude of their impact on runoff generation and subsequent flood discharges into a river system is still highly uncertain [1]. In discussions among scientists and water resources managers about the possible impacts on water resources from previous and ongoing land use changes, it has become increasingly clear that there is a need for improved knowledge and quantitative documentation of the impact of changes in land use and management practice on land and water resources.

Therefore, one of the recent aims in hydrological modelling is the assessment of the effects of land use and land cover changes on water resources and their influence on storm runoff generation [2]. Quite a number of rainfall-runoff models have been developed; typical of these are lumped parameter models and distributed parameter models. Physically-based rainfall-runoff models have the advantage of reflecting the effects of spatially distributed model parameters such as land use on stream flows. Moreover, the present day availability of spatially distributed data such as digital elevation models, land use, and soil information makes the use of distributed models much easier.

In Slovakia, several physically-based hydrological models with distributed parameters have recently been used for assessing the impact of land use or climate change on runoff and snow melting processes and for simulating sediment transport, e.g., Wasim, Topmodel and UEB-EHZ [3], [4]; WetSpa [5], [6], [7]; and AGNPS ([8], [9], [10], [11]. Holko and Kostka [12] compared the impact of land use on runoff in mountain catchments with different scales.

In this study the impact of land use on runoff generation was illustrated for the Laborec River basin. The physically-based rainfall-runoff model FRIER with distributed parameters was used for simulating the runoff. Several land use scenarios were created, and changes in the simulated runoff and its components were compared with the reference stage.

2 METHODS

2.1. Model description

The FRIER model is based on the principles of the WetSpa model. Several of its components were modified in order to make it more appropriate for modelling runoff from rainfall and snowmelt in Slovakia ([7], [13]). The model is executed as an ArcView GIS extension, and the whole preparation of the spatially distributed data is linked to the GIS interface. Three spatial map layers of the catchmentś physiographic characteristics are needed: a digital elevation model, a map of the land use types and a map of the soil types. From these maps other physiographical characteristics are derived in a digital form as maps: a flow accumulation map, a flow direction map, a the stream network map, a slope map, a hydraulic radius map and a map of the sub-watersheds.

The routing parameters of the FREIR model are generated in a newly-developed extension of the ESRI ArcView GIS program in a GIS interface. Other benefits of the model are the parameterization of the land use types and hydrophysical soil parameters. The values of the parameters, depending on the land use types in the model, are listed in Tab. 1 [13].

Data from the input time series can be spatially distributed by the methods of the arithmetic mean of the closest stations, nearest neighbours (Thiessen polygons), lapse rate, or kriging. Potential global radiation can be computed with or without the slope orientation of each cell and the shading of its neighboring cells. The difference between short-wave and long-wave solar radiation is expressed by the net radiation balance. Together with the surface energy balance, net radiation balance is required for the determination of the potential evapotranspiration. It is possible to choose among many methods, which were selected on the basis of a detailed study [13], to determine potential evapotranspiration (Blaney-Criddle, Budyko, Hamon, Jensen-Haise, Mintz-Walker, Penman, Penman-Monteith, Priestley-Taylor, Schendel, Smith-Stopp, Tomlain, Turc and Turc-Wendling; for details see[13]).

Besides the large number of physically-based parameters derived from the physiographic properties of the catchment, the model requires 10 calibrated global basin parameters, which are not spatially distributed and which are kept constant for all the grid cells of the basin [14], [13]. Several methods for the calibration of the global parameters and also several objective functions for assessing the model's efficiency (e.g., BIAS, Nash-Sutcliffe, logNash-Sutcliffe) are incorporated in the model.

The outputs from the FRIER model are time series of the elements of the water balance or spatial maps. The time series output contains a simulated discharge and its 3 components (overland flow, interflow and base flow) for a daily or hourly time step. The individual mean quantities for the whole basin are calculated for each time step: air temperature, potential and actual evapotranspiration, rainfall, snow melt, interception, evaporation from stored interceptions and depressions, infiltration, depression storage, evapotranspiration from the root zone, percolation, soil moisture, transpiration from groundwater storage and snow water equivalent. The output maps can be layers of the total overland flow, interflow, base flow, actual and potential evapotranspiration, soil moisture and snow cover for each time step [14], [13].

Tab. 1 Parameters of the FRIER model depending on land use types.

Land use types / Parameters	Root depth	Interception	Manning's coefficient	PET coefficient	Emissivity	Albedo	Leaf area index
Units	mm	mm	$m^{-1/3}.s^{-1}$	-	-	-	-
Water areas	500	0.00 – 0.00	0.04	1.20	0.950	0.07 – 0.11	0.0 – 0.0
Wetlands	1000	0.00 – 0.50	0.20	1.20	0.910	0.07 – 0.14	0.5 – 2.0
Artificial areas	0	0.00 – 0.20	0.10	0.90	0.890	0.08 – 0.30	0.0 – 0.0
Bare soils	500	0.00 – 0.50	0.02	0.85	0.910	0.11 – 0.30	0.5 – 2.0
Agricultural areas	1000	0.00 – 1.00	0.15	1.10	0.890	0.11 – 0.25	0.5 – 6.0
Meadows (short grass)	1000	0.00 – 1.00	0.20	1.00	0.980	0.16 – 0.27	0.5 – 2.0
Natural meadows (tall grass)	1000	0.15 – 1.50	0.40	1.05	0.970	0.22 – 0.31	1.0 – 6.0
Shrubs	1000	0.15 – 1.50	0.40	1.15	0.980	0.07 – 0.19	5.0 – 6.0
Transitional woodland shrubs	1000	0.05 – 1.50	0.40	1.15	0.975	0.16 – 0.27	1.0 – 6.0
Evergreen broad-leaved forests	1500	0.20 – 2.00	0.60	1.20	0.970	0.22 – 0.27	5.0 – 6.0

Deciduous broad-leaved forests	2000	0.05 – 2.00	0.80	1.20	0.970	0.16 – 0.27	1.0 – 6.0
Mixed forests	2000	0.15 – 3.00	0.60	1.20	0.975	0.11 – 0.22	3.0 – 6.0
Evergreen coniferous forests	1500	0.30 – 4.00	0.40	1.15	0.980	0.07 – 0.19	5.0 – 6.0
Deciduous coniferous forests	1500	0.15 – 4.00	0.40	1.15	0.975	0.11 – 0.19	1.0 – 6.0

2.2. The Laborec case study

The Laborec River belongs to the river system of the Tisza (the Danube River system). It drains the most eastern part of Slovakia, the drainage area being 4,522.7 sq. km. The river rises in the mountain range of Nízke Beskydy at an elevation of 730 m a.s.l. The watercourse is 135 km long. In the spring area of the Laborec it flows through a flysh zone, the gradient averaging 7 meters per thousand. In the town of Humenné, the stream leaves the highlands to flow through the East Slovakian plain, the gradient being 0.7 meters per thousand, to meet the Latorica River at an elevation of 94 m a.s.l. The most important tributaries of the Laborec River are the Udava, Cirocha and Uh streams.

A part of the Laborec River basin with an outlet at the Humenne gauge station and an area of 1272 km^2 was selected as the pilot basin for the study. The FRIER distributed model was calibrated and validated in a daily time step on the climatological and hydrological data from the climate and hydrological stations for the period 1991-2000. The results of the model´s calibration are illustrated by comparing the simulated and measured mean daily discharges in the basin outlet (Fig. 1). The spatial distribution of the simulated long-term mean total runoff for the period 1991-2000 is shown in Fig. 2.

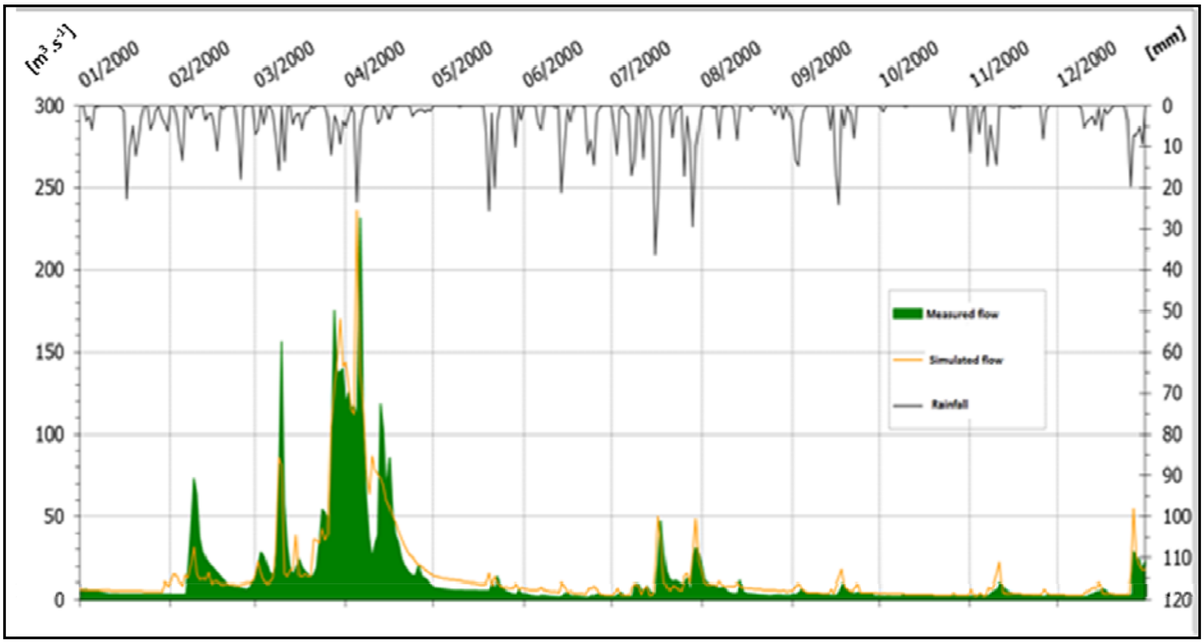


Fig. 1: Calibration of the Laborec – Humenné River basin.
Comparison of the measured and simulated flows in 2000.
Nash Sutcliffe = 0.781

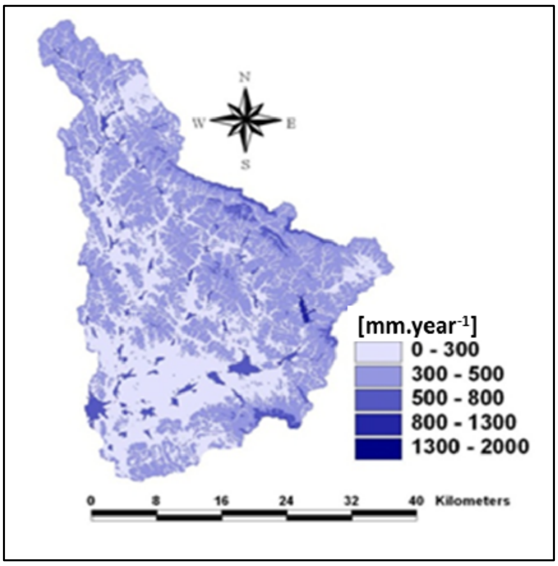
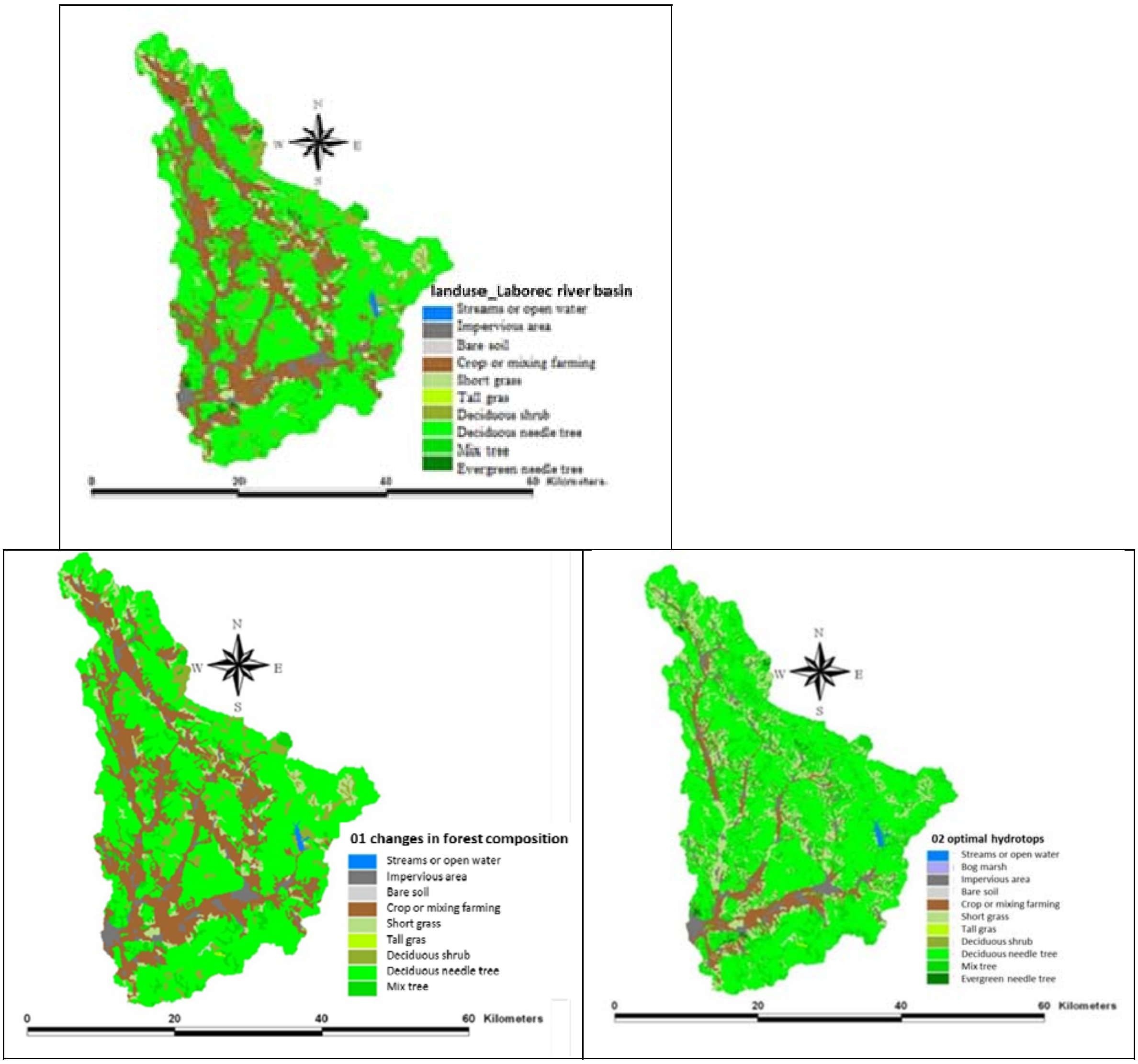


Fig. 2: Long-term average total runoff in the Laborec – Humenné River basin (1991 - 2000)



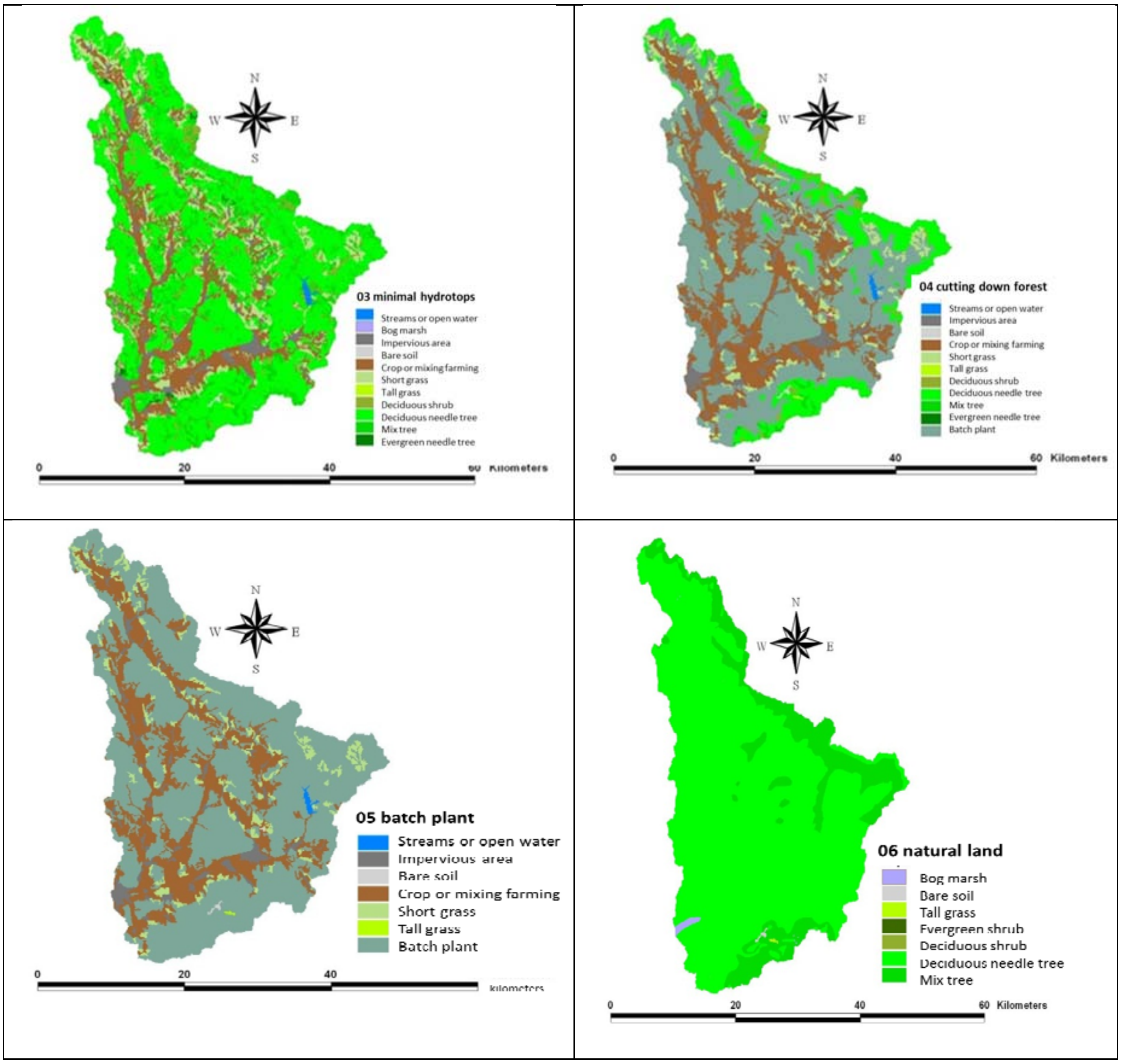


Fig. 3: Scenarios of land use changes for the Laborec – Humenné River basin: a) actual state; b) changes in forest composition; c) optimal hydrotops; d) minimal hydrotops; e) cutting down the forest; f) batch plant; g) natural land

Several scenarios of the land use changes for the Laborec River basin were created with an emphasis on changes in forests, grasslands and arable land. The simulated long-term men annual total runoff, its components and evapotranspiration (potential and actual) under these scenarios are listed in Table 2. The differences in the simulated mean annual total runoff and its components in comparison with the actual stages are listed in Table 3.

Tab. 2 Simulated long-term mean annual total runoff and its components in mm.year^{-1} for the actual status and several scenarios in the Laborec – Humenné River basin.

Scenarios	Surface runoff	Interflow	Groundwater flow	Total runoff	Potential evapotranspiration	Actual evapotranspiration
00 actual state	28	99	208	335	799	666
01 changes in forest composition	28	98	207	334	798	665
02 optimal hydrotops	27	94	204	326	804	673
03 minimal hydrotops	28	96	206	330	802	670
04 cutting down a forest	32	130	237	399	706	597
05 batch plant	35	142	245	422	674	573
06 natural land	12	89	196	297	833	705

Scenario – actual state	Total runoff	Actual evapotranspiration
00 actual state	335	666
01 changes in forest composition	-1	-1
02 optimal hydrotops	-9	8
03 minimal hydrotops	-5	4
04 cutting down a forest	64	-69
05 batch plant	87	-93
06 natural land	-38	39

Tab. 3 Differences in long-term mean annual total runoff and its components in comparison with the actual state in the Laborec – Humenné River basin.

Scenarios	Surface runoff	Interflow	Groundwater flow	Total runoff	Potential evapotranspiration	Actual evapotranspiration
01 changes in forest composition	0	-1	0	-1	-1	-1
02 optimal hydrotops	-1	-4	-4	-9	5	8
03 minimal hydrotops	0	-3	-2	-5	3	4
04 cutting down the forest	4	31	30	65	-93	-69
05 batch plant	7	43	37	87	-125	-93
06 natural land	-16	-10	-12	-38	34	39

In scenario 01, “Changes in forest composition”, almost no change in the runoff in comparison with the actual state was simulated. Scenario 02, "Optimal hydrotops", showed a decrease in total runoff by -9 mm.year^{-1}, which is (-2.6%) in comparison with the reference stage. For the scenario 03, "Minimum hydrotops", the total runoff decreased by -5 mm.year $^{-1}$ (-1.5%), which is also almost no change from the actual state. The surface, interflow and groundwater discharges in these scenarios remain unchanged. The actual and potential evapotranspiration in both scenarios increased only slightly. In scenario 04, "Cutting down the forest", it predicts cutting down the forest to an altitude of 574.2 m a.s.l.. The total runoff is increased by +65 mm.year^{-1}, an increase of (+19%). The surface runoff is increased by +4 mm.rok^{-1} (+14%), the interflow by +31 mm.year^{-1} (+31%), and the groundwater increased by +30 mm.year^{-1} (14%). The actual evapotranspiration was reduced by -93 mm.year^{-1} and the potential evapotranspiration was reduced by -69 mm.year^{-1}. In scenario 05, "batch plant", the total runoff increased significantly, by about +87 mm.year^{-1} (+26%), and also the surface runoff by +7 mm.year^{-1} (+25%), the interflow by +43 mm.year^{-1} (+43%) and the baseflow increased by +37 mm.year^{-1} (+18%) compared with actual state. The accentuated decline in this scenario occurred in the potential evapotranspiration, which decreased by -125 mm.year^{-1}

(-16%) and in the actual evapotranspiration, which was lower by -99 $mm.year^{-1}$, i.e., about (-14%) compared with the actual state. The last scenario 06, "Natural lands", in this basin showed a reduction of the total runoff by -38 $mm.year^{-1}$ (-11%) compared with the current state. The surface runoff was lowered by -16 $mm.year^{-1}$ (-57%), the interflow by -10 $mm.year^{-1}$ (-10%) and groundwater discharge about $mm.year^{-1}$-12 (-6%). The actual and potential evapotranspiration was reduced only slightly, by - 5%.

3. Conclusions

Some authors believe that truly physically-based models with distributed parameters require no calibration and take their reliability from physically-based equations [15]. However, our ability to assess or forecast the progress of hydrologic processes in a basin is considerably limited in spite of many models, huge databases of climatic, geological, hydrological and other data, and advanced technology for obtaining and processing them. Distributed parameter models are indeed suited to predict the hydrological effect of land use changes when their parameters have a physical interpretation, and the structure allows for an improved representation of the spatial variability. However, the current generation of distributed models can often be considered as semi-distributed conceptual models, because they use equations based on small scale physics and apply them on a grid scale. In ideal cases (intensive data collection), model parameters are measured or estimated from catchment characteristics. More commonly, however, distributed models have their parameters determined from calibration, because of the unknown spatial heterogeneity of the parameter values and the cost involved in their measurement. Nevertheless, physically-based rainfall-runoff models are used for a wide range of applications, such as the extension of stream flow records, the estimation of flows for ungauged basins, the prediction of the effects of land use change, and examinations of the effects of climate change.

Acknowledgements

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EXTREME RUNOFF SCENARIOS IN THE UPPER HRON RIVER BASIN

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Abstract

Hydrological modelling plays an important role in modern hydrology. It is also used in the water management, for example, to simulate extreme hydrological events, impacts of floods on society, etc. The study deals with the analysis and simulation of extreme runoff events in the Upper Hron River basin (in the Banská Bystrica profile) and also takes a more specific look at a small catchment of the Čierny Hron River basin located in the Upper Hron River basin. The analysis was carried out with the use of the HRON conceptual semi-distributed rainfall-runoff model. Two extreme flood events were chosen in the catchments, one in the summer and one in the winter period, to create scenarios of extreme runoff events in a daily-time step. In almost all the cases the cause of the high peak discharge was a cyclonic precipitation event lasting up to 10 days. We compared the significance of the floods in both basins and the simulated summer and winter floods with the higher degree of significance. The results of the analysis indicate that summer floods are characterised more by a flood peak while winter floods are more dangerous because of the high increase in the volume of the flood wave caused by snow melt. The results obtained can be applied to proposals for flood protection measures in both basins.

Keywords

Calibration, HRON model, scenarios of extreme floods, rainfall-runoff modelling.

1 INTRODUCTION

The aim of this paper is to consider the upper Hron River basin in terms of threats from extreme runoff events in the winter and summer periods and to take a more specific look at a small sub-catchment of the Čierny Hron River basin.

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A need to create scenarios of extreme flood situations is arising from the importance of designing effective flood defences. Therefore it is important to evaluate the possibilities for extreme flood events caused by combinations of extreme meteorological events and previous catchment conditions.
Mathematical modelling of rainfall-runoff processes is increasingly important in engineering hydrology and is also applied by forecasting and design purposes [1]. Worldwide, there is a wide range of conceptual models (e.g. HBV [2]; WatBal [3] and TOPMODEL [4]) used to simulate hydrological processes. In this study the HBV - type model HRON [5], which was developed at the Department of Land and Water Resources Management of the Slovak University of Technology in Bratislava (DLWRM), was used in the modelling of extreme events. The original HBV model [2] is a conceptual semi-distributed model, which simulates river discharges by the use of precipitation, temperature and evapotranspiration as inputs. The HBV model and its modifications are widely used for modelling in European catchments and worldwide, e.g., modelling the impacts of climate change on a Norwegian high-head hydropower system [6], or investigating uncertainties in the modelling of the hydrological impacts of climate change on projected flood frequencies in South China [7]. The HRON model was used for modelling in the works of, e.g., Hazlinger and Zvolenský [8], in the usage of nowcasting in forecasting flash floods and in numerous works at the DLWRM.

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2 METHODS

2.1. The HRON model

The HRON model is a conceptual semi-distributed model that can be used in a daily or hourly time step for rainfall-runoff modelling; it consists of three submodels with 15 parameters (DLWRM; [5]) (Figure 1):

1. Accumulation and snow melt submodel (degree-day method, 6 parameters)
2. Soil moisture accounting submodel (calculation of actual evapotranspiration and actual soil moisture, 3 parameters)
3. Routing and response function submodel (one nonlinear and one linear reservoir to simulate surface, subsurface runoff and base flows. Basin runoff is calculated by Muskingum or a triangular weighting function, 6 parameters)

The model requires the catchment size, the average precipitation, the air temperature and the potential evapotranspiration as inputs, and the observed discharge data are needed for model calibration and validation.

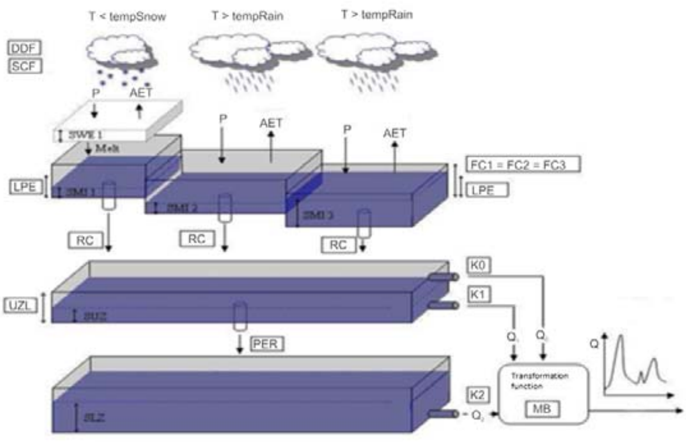

Fig.1 Scheme of the HRON model [5]

2.2. Estimation of design values

For estimation of design values in this study, the DVWK/101 methodology [9] was applied. The plotting positions of the analysed data are calculated according to Cunnane, [9] as:

$$P = \frac{m - 0.4}{n + 0.2} \tag{1}$$

where:

n - the sample size,
m - the rank of the observations in a descending order.

Three methods were used for estimating parameters of the theoretical distribution functions – the method of moments (MOM), the maximum likelihood method (ML), and the method of probability weighted moments (PWM). The methodology uses the following theoretical distribution functions:

- E1 – Gumbel (MOM, ML, PWM),
- GEV – generalised extreme value (MOM, ML, PWM),
- ME – Rossi (ML),
- LN3 – 3-parameter lognormal (MOM, ML, PWM),
- P3 – Pearson III (MOM, ML, PWM),
- LP3 – logPearson III (MOM, ML, PWM),
- WB3 – 3-parameter Weibull (MOM, ML, PWM).

To select the most appropriate all of the fitted distributions, a statistical test is recommended. The testing criterion is computed from the relationship:

$$D + n\varpi^{2} + \left(1 - r_{p}\right) \qquad (2)$$

where:
D - the value of the Kolmogorov test,
ϖ^2 - the value of the omega squared test,
r_p - is the correlation coefficient between values of descending sorted discharges and their distribution quantiles.

The best fit gives the lowest values of D and ϖ^2, and the highest values of r_p; by minimizing the value of the equation (2).

2.3. Input data

The Hron River basin is located in central Slovakia. The study was focused on the Upper Hron River basin (Figure 2) in the Banská Bystrica profile with a catchment area of 1765.96 km^2 and also takes a more detailed look at a small sub-catchment of the Čierny Hron bordered by the Čierny Balog profile (64.61 km^2).

Fig. 2 The location of the Čierny Hron sub-catchment in the Upper Hron River basin in Slovakia with the Banská Bystrica and Čierny Balog gauging stations

The input data in a daily time step (precipitation, air temperature and average daily discharge data) were obtained from the Slovak Hydrometeorological Institute in Bratislava (SHMI) from the time period of 1981 to 2000.

The average daily precipitation totals for the whole studied catchment area were obtained from 21 rain gauge stations (Figure 3). The inverse distance weighting method was used to convert the station's data to the basin's average daily precipitation data for use in the HRON model. For the sub-catchment calculations only data from 3 gauging stations (Čierny Hron, Lom nad Rimavicou and Brezno) were used for the Thiessen polygons method.

Fig. 3 The location of the rain gauge stations in the Upper Hron River basin

The average daily air temperature data in the Upper Hron River catchment from the climate stations located in the catchment were calculated by linear regression between the air temperature and the altitude of the Banská Bystrica –Zelená, Brezno, Chopok, Lom nad Rimavicou and Telgárt stations. Data from the Čierny Balog, Brezno and Lom nad Rimavicou stations were used for the Čierny Hron sub-catchment

The duration of the sunshine in the whole Upper Hron River basin was calculated by a SOLEI 32 model [10]. The sunshine index was calculated as a ratio of the duration of the daily sunshine and the duration of the total annual sunshine for the individual grids of a digital elevation model (DEM). The average sunshine index was calculated from the catchment area in GIS IDRISI from raster maps with cell size of 100 x 100 m. The daily potential evapotranspiration for the Upper Hron River catchment and the Čierny Hron River basin was calculated from the average sunshine index value by the Blaney-Criddle [11] formulae (3):

$$PET = -1.55+0.96*(8.128+0.457.T)*I_O \quad \text{(mm/day)} \qquad (3)$$

where:

I_O – sunshine index (-),
T – daily average air temperature (°C).

3 ANALYSIS AND SIMULATION OF EXTREME RUNOFF EVENTS

The 15 parameters of the HRON rainfall-runoff model were calibrated by a cross-calibration method, where the data from the years 1981 to 2000 were divided into two time periods. The successful calibration of the whole Upper Hron River basin was performed with a Nash-Sutcliffe (NS) coefficient of 0.82. Then the extreme flood waves selected were calibrated individually. The initial input values for this calibration were taken from the output data (modelled stages - soil moisture, snow water equivalent (SWE), upper groundwater box and lower groundwater box) from the whole calibration of 1981-2000 from one day before the start of the calibrated wave in the both Upper Hron and Čierny Hron basins.

2.1. Summer floods - Calibration of a extreme summer flood wave

The highest flood peak for a summer wave appeared on 24.5.1984 in both basins. In the Upper Hron River basin it achieved a value of 219.195 $m^3.s^{-1}$, and in the Čierny Hron River basin it was 12 $m^3.s^{-1}$. This extreme runoff situation was caused by a 5-day precipitation event in the Upper Hron River basin and a 10-day precipitation event in the Čierny Hron River basin. The results of the calibrations by harmony and genetic search algorithms were slightly better for the smaller catchment: 0.86 for the Upper Hron River basin (Figure 4) and 0.91 for the Čierny Hron catchment (Figure 5).

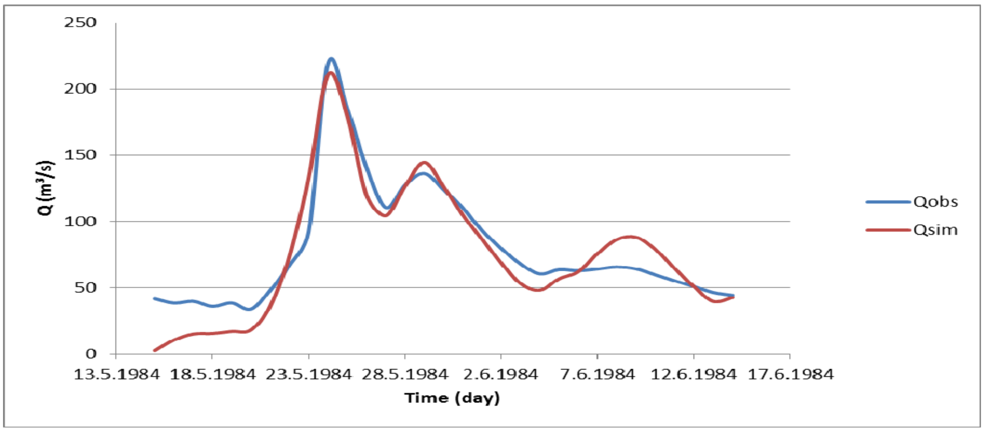

Fig. 4 Summer flood wave in the Upper Hron River basin (NS coefficient of 0.86)

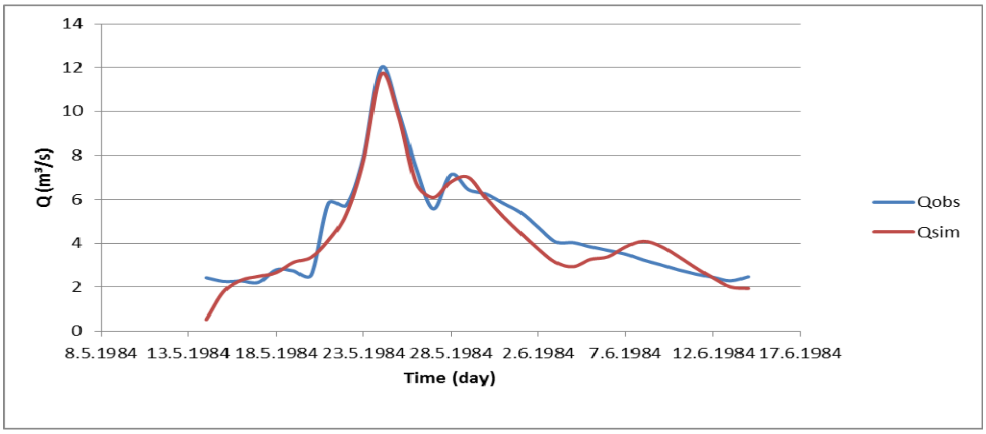

Fig. 5 Summer flood wave in the Čierny Hron River basin (NS coefficient of 0.91)

2.2. Winter floods - Calibration of an extreme winter flood wave

An extreme runoff situation in the Upper Hron River basin in the winter season was caused by a combination of a 5-day precipitation total which fell on the catchment with a high accumulation of snow and a subsequent rise in the temperature. The measured daily discharge was 236.7m^3.s^{-1} on 12.3.1981. The result of the calibration with the NS coefficient of 0.90 is shown in Figure 6.

For a comparison, a flood wave in the same time period was simulated in the Čierny Hron sub-catchment. The highest observed peak discharge (3.8 m^3.s^{-1}) in this time period appeared on 17.3.1981 and was preceded by a 10-day precipitation event. The results of the calibration are displayed in Figure 7 with a NS coefficient of 0.96.

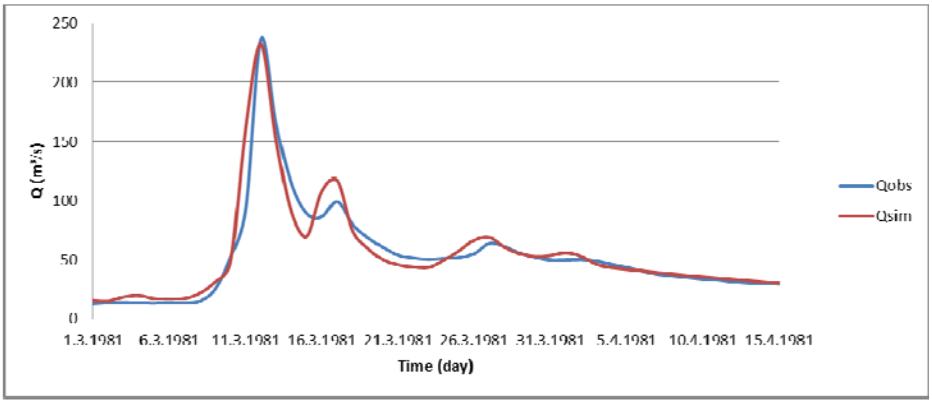

Fig. 6 Winter flood wave in the Upper Hron River basin (NS coefficient of 0.90)

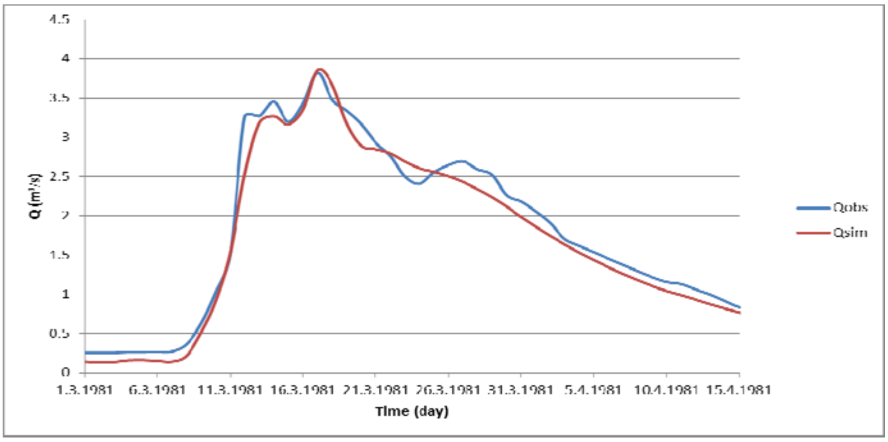

Fig. 7 Winter flood wave in the Čierny Hron River basin (NS coefficient of 0.96)

2.3. Scenarios of extreme flood events

The scenarios of extreme flood events were created by exchanging the values of the actual measured causal precipitation of selected waves with the design values. A design rainfall value with a return period 100-years was chosen from the precipitation scenarios for the summer season estimated by Kohnova et al. [12] For the winter season, precipitation scenarios with a 100-year return period were also estimated [12]. A SWE value (290.2 mm) for the winter season scenarios with a return period of 100 years was calculated (SHMI) and used as an input in the combined SWE-precipitation scenario (Tables 1 and 2).

Tab. 1 Values of design 10-day precipitation totals calculated for Čierny Balog – Krám station [12]

	DVWK methodology	100 -year precipitation total (P100) [mm]
Winter period	WB3/PWM	177
Summer period	GLO/LM	158.8

Tab. 2 Values of design 5-day precipitation totals for the Upper Hron River basin – calculated from the average precipitation totals for the Upper Hron River basin [12]

	DVWK methodology	100 -year precipitation total (P100) [mm]
Winter period	AE/PWM	189.7
Summer period	EI/PWM	197.5

2.3.1 Summer season

In the next step scenarios with increased precipitation values with a 100-year return period were modelled. Table 3 shows the results for the modelling of the summer season scenarios in both basins. In both cases the peak discharge and volume of the flood wave increased significantly. From Table 3 we can observe that the highest percentage increase (223%) of the flood peak was in the Upper Hron River basin. The volume of the flood wave also increased more significantly for the Upper Hron River basin flood wave (91%) in comparison with the volume of the observed flood wave.

Tab. 3 Scenarios of extreme flood events with 100 precipitation return periods for the summer season (Qmax – peak discharge, P100 – precipitation return period 100-years)

	Qmax (m^3/s)			Volume (m^3)		
	Observed	P100	Increase in (%)	Observed	P100	Increase in (%)
Upper Hron	219.2	708.2	223	$2.1.10^8$	$4.1.10^8$	91
Čierny Balog	12	17.4	45	$1.3.10^7$	$1.5.10^7$	26

2.3.2 Winter season

Table 4 describes the results of the scenarios modelling in the winter season, where scenarios with a rainfall return period of 100 years, and a SWE with a return period of 100 years as well as their combinations were modelled. Table 4 is divided into two parts, where in the first half the modelling results for the peak discharges are shown. The peak discharges increased significantly in both basins. The second half of the table shows the increase in volume in all the modelled cases. In the first part of Table 4 we can see that the highest peak simulated in the Upper Hron River basin increased the observed discharge by 385%. The increase in volume was more apparent in the Čierny Hron River basin, where the highest increase in volume of 210% was obtained in the scenario with a combination of a 100 year precipitation totals return period and a 100 year SWE values.

Tab.4 Scenarios of extreme flood events with a 100 year precipitation and a100 year SWE return periods and their combination for the winter season (SWE100 – a 100 year return period of SWE)

	Qmax (m^3/s)							Volume - increase in (%)		
	Observed	P100	Increase in (%)	SWE100	Increase in (%)	P100+SWE 100	Increase in (%)	P100	SWE100	P100+SWE 100
Upper Hron	236.7	1148.4	385	232.4	-1.8	1148.3	385	65	65	118
Čierny Hron	3.8	10.8	184	9.2	142	10.8	184	94	139	210

Comparison of changes in design values

In the last step a suitable statistical distribution for estimating the design return periods of the simulated maximum daily discharges and corresponding volumes were tested using the DVWK (1989) methodology, and the return periods were estimated [9]. For estimating the design volumes in the summer and winter seasons in the Hron River basin, the WB3/PWM distribution was the most suitable, and the GEV/ML distribution was best for discharges in the winter season. The LP3/ML distribution was selected to estimate the return period of the discharges in the summer season.

In Table 5 a comparison of the return periods of the observed and simulated mean daily discharges in both basins in the summer period is presented. According to the results the simulated flood is more significant in the Upper Hron River basin.

Tab. 5 A comparison of the return periods of the observed and simulated discharges and flood volumes for the scenarios with 100-year precipitation totals in the summer season for both basins

	Return period in years	
	Observed	P100
Upper Hron	60	more than 500 years
Čierny Hron	25	50

In Table 6 a comparison of the return periods of the observed and simulated discharges and flood volumes in the winter season in both basins is presented. The return period of the peak discharges increased more significantly in the Upper Hron River basin, but the return period of the flood wave volume rises in both cases to more than 1000 years in all the scenarios.

Tab. 6 A comparison of the return periods of the observed and simulated discharges and flood volumes for the scenarios with different precipitation and SWE return periods and their combination in the winter season

	Return period in years				Volume - increase in (%)	
	Observed	P100	SWE100	Combination	SWE100	Combination
Upper Hron	20	more than 1000	20	more than 1000	more than 1000	more than 1000
Čierny Hron	2	10	10	10	more than 1000	more than 1000

4 CONCLUSION

In this study two extreme flood events were chosen in the Upper Hron River basin and the Čierny Hron sub-basin, one in the summer and one in the winter period, to create scenarios of extreme runoff events in a daily time step. The HRON rainfall-runoff model was used to obtain the results. In the first step the Upper Hron river basin was calibrated; the results of the calibration were later used as the inputs for the calibration and creation of the extreme runoff scenarios for the selected historical flood waves. In almost all the cases the cause of the peak discharge was a five-day-long cyclonic precipitation event; in the case of the winter flood in the Čierny Hron sub-basin, it was a 10-day-long precipitation event.

The results of the analysis show that the summer floods are characterised by a high flood peak, and the scenarios suggest that they are more significant in the Upper Hron River basin, while winter floods are dominated by a high increase in the volume of the flood wave caused by snowmelt. The scenarios for the 100 year return periods of the precipitation and snow water equivalent show an increase in the volumes for more than 1000 years in all the cases. We can conclude that by modelling the changes in floods with a higher SWE, the volume of the flood wave is generally more significant than the peak discharge.

The results obtained can be applied for proposed flood protection measures in both basins, and as a tool for evaluations of flood protection designs and flood impacts.

Acknowledgements
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SIMULATION OF HYDROLOGICAL RESPONSE TO THE FUTURE CLIMATE IN SLOVAKIA

Zuzana Macurová[1], Kamila Hlavčová[1], Ján Szolgay[1]

Abstract

The impacts of climate change on hydrological processes are often estimated by defining the scenarios of changes in climatic inputs in a hydrological model from the output of global (GCM) or regional circulation models (RCM). Global Circulation Models can reproduce climate features on a large scale reasonably well, but their accuracy decreases when proceeding from continental to regional and local scales because of the lack of resolution. In the region of Central and Eastern Europe the need for high resolution studies is particularly important. In the paper, the potential impact of climate change on the mean monthly runoff in selected basins in Slovakia, which were chosen as representative of mountainous regions; were evaluated. The Hron (Slovakia) conceptual rainfall-runoff model which was calibrated and validated with data from the period 1981-2010, was used for modelling changes in runoff with daily time steps. Changes in climate variables in the future were expressed by three regional climate change models developed at the Department of Astronomy, Earth Physics and Meteorology of Comenius University in Bratislava and within the framework of the 6 FP CECILIA (Central and Eastern Europe Climate Change Impact and Vulnerability Assessment) project. Climate outputs from the scenarios were represented as daily precipitation totals and the air temperatures for the future time horizons of 2021-50 and 2071-2010 in gridded 10 x 10 and 25 x 25 km forms, respectively. These gridded climate outputs were spatially averaged over the selected basins. The runoff change scenarios for the selected basins in the future time horizons show changes in the runoff distribution within a year. It could generally be concluded for both of the time horizons investigated that during the winter and early spring periods, an increase in the long-term mean monthly runoff could be assumed. On the other hand, a decrease in the mean monthly runoff can be expected during the summer season.

Keywords

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ALADIN-Climate, HBV model, KNMI, Laborec River basin, MPI.

1 INTRODUCTION

The impact of climate change on hydrological processes is usually estimated by using the climatic inputs of global circulation models (GCMs) in a hydrological model. This statement was also reported in the IPCC Fourth Assessment Report [1]. Global Circulation Models can reproduce climate features on a large scale reasonably well, but their accuracy decreases when proceeding from continental to regional and local scales because of the lack of resolution. The study presented is therefore aimed at the impact of climate change on hydrological processes in future periods using three different regional climate models – ALADIN-Climate, KNMI and MPI.

In this study the potential impact of climate change on runoff in the Laborec River basin was evaluated using the Hron conceptual rainfall-runoff model in a daily time step. Calibration of the model parameters was performed for the reference period of the years 1981-1995. Simulations of changes in runoff were realized for the future time horizons of 2021-2050 and 2071-2100. The results of the simulations were compared with the reference period and expressed as changes in the long-term mean monthly runoff at the basin outlet in Humenné.

2 METHODOLOGY

The methodology of this study consists of three steps:

1. Calibration of the conceptual rainfall-runoff hydrological model in a daily time step for the Laborec River basin. For the calibration we used the mean daily basin data of rainfall and air temperatures; and the mean daily discharges in the basin outlet.
2. Simulation of the mean daily runoff series for the reference period of 1961-1990 and future time horizons of 2021-2050 and 2071-2100 using the climate data input from three different regional climate scenarios.
3. Comparison of the differences between the seasonal runoff distribution in the reference period and future time horizons.

2.1 Area studied

Laborec is a Slovak river, which is 129 km long and rises in the Nízke Bezkydy at an altitude of 682 m above sea level. For this study the sub-basin of the Laborec river until the Humenné gauge station (Fig. 1) with a area of 1281.29 km^2 (28.3% of the entire Laborec River basin) was selected.

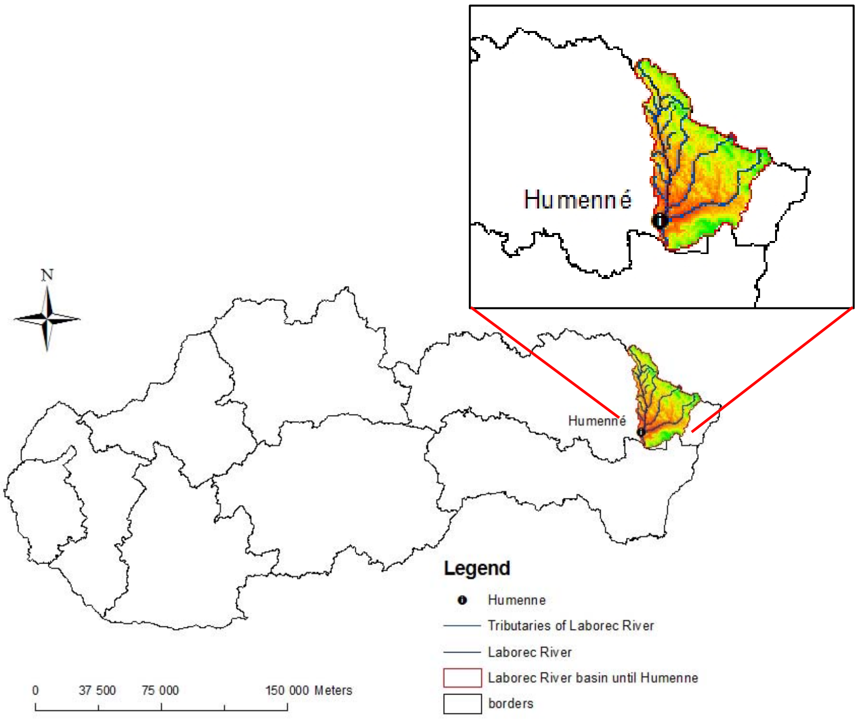


Fig.1 Laborec River basin until the Humenné gauge station

The long-term mean annual discharge of the Laborec river in Humenné is 54.5 m^3/s; it should be noted that there are high fluctuations in the discharges. The minimum discharge measured by the year 2010 was 0.49 m^3/s and the maximum was 457 m^3/s. The Laborec River has also been well known for severe floods in the past that were followed by significant property damage. These floods also occur in the present. It is therefore understandable that the first hydrological studies in the former Czechoslovakia were focused on flooded areas. Among the first of such comprehensive studies belongs a study prepared in 1935 by the T.G.Masaryk State Institute of Hydrology in Prague - Podbaba called “Hydrological studies on the treatment of the Bodrog large river basin and its tributaries”.

The predominant soil on the territory of the Laborec River basin until Humenné is a clay loam, which is located on more than 85% of the territory. Other soils are mainly represented by loam and sandy loam.

The dominant land use of the area studied is various kinds of forest, mostly deciduous broadleaf forest (54%) and deciduous needle leaf forest (10%). In the mountainous areas there is mostly deciduous broadleaf forest, and in the lowlands there are croplands (25% of the total area) and urban areas (4%).

2.2 The Hron model

The HBV model is a conceptual hydrological model, the development of which started back in the 1970s at the Swedish Hydro-meteorological Institute [2]; it has undergone tremendous developments. Several versions of this model have been created (e.g., HBV - light).

The version used in this work is called Hron model and is reprogrammed in the Matlab environment as a function package and stand-alone application. This version of the model makes it easy to interface the model with other existing tools. The Hron model has already been successfully tested in several river basins and is still active due to continuous improvements and the addition of various new components [3].

The Hron model divides a basin into subcatchments, which are considered as primary hydrological units; within these units altitude zones are intended, and land use is classified. Subcatchments are selected to form a homogeneous geographical or climatological unit. The model works with a daily time step, but can also be used for shorter steps.

The model contains three main sub-models: a snow sub-model, a soil sub-model, and a runoff sub-model.

1. The snow sub-model is based on a simple "degree-day factor" method, i.e., the method takes into account the air temperature factor in a given day. This sub-model simulates the accumulation and melting of snow in a basin.
2. The soil sub-model represents the part of the hydrological cycle which is happening under the surface. It includes processes such as the infiltration of precipitation and melted snow, the distribution and storage of water in the soil layers, the evapotranspiration and generation of the surface, and the subsurface and groundwater runoff. The soil layer is represented by two tanks: a top one (the surface and subsurface runoff) and a lower one (underground drainage). The calibration is controlled by eight parameters.
3. The runoff generation sub-model is used to transform the total runoff from the catchment. It is possible to use three different transformation functions: two triangular transformation functions or one function of the linear reservoir cascades.

2.3 Climate change scenarios

The most recent climate change scenarios for Slovakia have been scenarios based on the outputs of the Canadian CGCM3.1 global circulation model, the German ECHAM5 global circulation model, the Dutch KNMI regional model, the German MPI regional model [4] and the ALADIN-Climate regional model [5]. The characteristics of the CGCM3.1, ECHAM5, MPI and KNMI circulation models belong to the group of so-called coupled atmosphere-ocean models with more than ten atmospheric layers and 20 oceanic depths of various model equations.

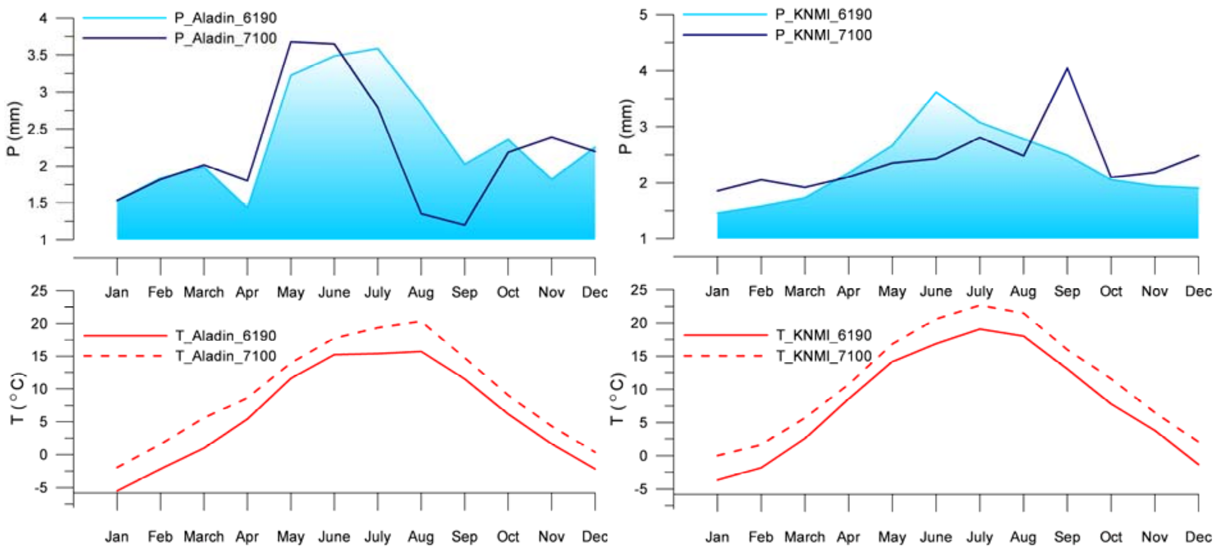
The KNMI and MPI regional models which were used in this work represent a more detailed integration of atmospheric and oceanic equations with a grid resolution of 25 x 25 km. The boundary conditions for both models were taken from the GCM ECHAM5. The models on the territory of Slovakia have 190 grid points. According to the methodology developed at the Department of Astronomy, Earth Physics and Meteorology, MFFI University in Bratislava and published in Lapin et al. [6], [7], the processing of outcomes as a time series can be divided into several steps. The outputs of the model have almost the same statistical characteristics in the reference period of 1961-1990 as the measured values. However, it is obvious that the values of the measured and modeled data differ across days and months. There are several reasons for these differences. The first one is that the climate models do not predict the weather; instead, the predict the climate as several possible alternatives. Other reasons included the different altitudes of the observation stations and grid points, differences in the modeling and the actual topography, and interpretations of oceans or some other physical processes in the model. The downscaling methods are described in Lapin et al. [7].

The outputs of the KNMI and MPI climate scenarios are processed from 1950 to 2100. Three simulated periods were selected for possible comparisons with another RCM (ALADIN-Climate) used: 1961-1990 (the reference period), 2021-2050 and 2071-2100.

The ALADIN-Climate model was developed by an international consortium of several European and North African countries, led by France. Its history dates back to the early 1990s, when it was developed and used only for the downscaling of short-term weather forecasts (72 hours) generated by GCM APREGE at Météo-France. The first study, which was based on research on longer-term predictions, was made in 1995 [8]. The results showed that the model is suitable for providing stable and sufficiently actual results of the presentation of a climate; no accumulation of destructive biases has been observed. Later, after 2000, this version of the model was tested independently by CNRM/Météo-France and the Czech Hydro-meteorological Institute.

The results confirmed the stability of the model, which has proven to be useful and functions well enough to obtain reliable information about the climate in Central Europe for the purpose of experiments relating to the current and past climates. Recently, the model was also used for climate change research purposes in several national and international projects such as the EC FP6 ENSEMBLES [9] or EC FP6 CECILIA (www.cecilia-eu.org). As a part of the CECILIA (EU FP6 project CECILIA) sixth framework program of the European Union, the ALADIN-Climate/CZ regional climate model was created to provide information about the future climate in high definition on the territory of Central Europe. The ALADIN-Climate/CZ climate scenario was created based on the global APREGE GCM model and the A1B emission scenario according to the IPCC [5]. The outputs of the climate scenarios are in a daily step. The reference period for the model is the same period as for the MPI and KNMI models (1961-1990). The outputs from the ALADIN-Climate model are for the reference period and the two future periods of 2021-2050 and 2071-2100.

The outputs of the air temperature and precipitation totals from the ALADIN-Climate, KNMI and MPI climate scenarios can be seen in Figure 2.



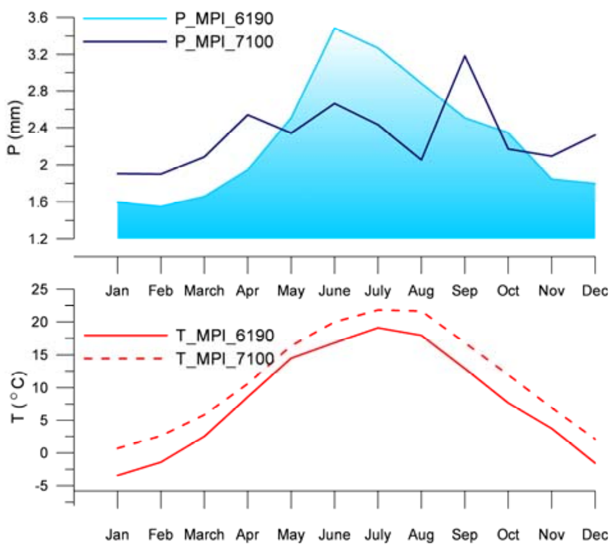


Fig.2 The outputs of the air temperature and precipitation totals from the ALADIN-Climate, KNMI and MPI climate scenarios

3 CALIBRATION OF THE HRON MODEL

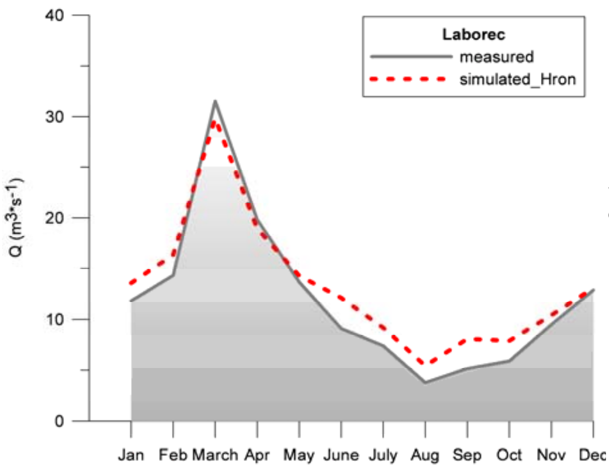
The model was calibrated for a period of 15 years from 1.1.1981 to 31.12.1995. The validation period was from 1.1.1996 to 31.12.2010. Manual as well as automatic calibration was used to calibrate the model parameters.

With the manual calibration the optimized parameters of the model are adapted by entering their values, based on the user's experience. The combination of parameters is then evaluated based on the optimization criterion chosen.

The automatic calibration is based on the use of optimization methods and the introduction of optimization algorithms into the model. The target is to investigate many combinations of the parameters until the model meets the best value of the optimization criterion selected. The advantage of the automatic calibration compared to the manual is that it is less time-consuming. More combinations of the parameters can be investigated at the same time, and the user does not have to be so professionally educated.

Genetic algorithms and harmony search were tested for the automatic calibration. Genetic algorithms are more time consuming compared with harmonic search, but it was confirmed that in the case of using the Hron model, they provide better results, i.e., they reach a higher value of the Nash – Sutcliffe coefficient, which was used as the optimization criterion in this work.

The results of the calibration are shown in Fig. 3, where the simulated mean daily and monthly discharges are compared with the measured values. The results are shown as mean monthly discharges at the outlet of the river basin, which were calibrated for the entire calibration period, and the mean daily discharge at the basin outlet is shown for the selected year 1995.



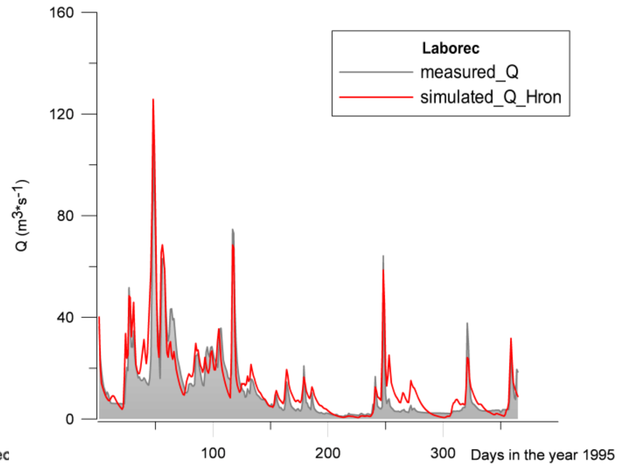


Fig. 3 Mean monthly discharges at the basin outlet calibrated for the entire calibration period and the mean daily discharges at the basin outlet for the selected year 1995.

4 RESULTS

The changes in the discharges were evaluated on the basis of the expected changes in the precipitation and air temperature under the different climate scenarios - ALADIN-Climate, KNMI and MPI - for the future periods of 2021-2050, 2071-2100 and the reference period of 1961-1990.

According to the Hron model simulation, the mean monthly discharges at the basin outlet were evaluated. The results were expressed as changes in the discharges in the future periods of 2021-2050 and 2071-2100 compared to the reference period of 1961-1990. On the basis of the comparison the changes for the individual climate scenarios were evaluated. The ALADIN-Climate scenario is referred to as Aladin to simplify the graph.

The results of the changes in the mean monthly discharges for the periods of 2021-2050 and 2071-2100 compared to the reference period (1961-1990) are illustrated in Table 1. Only the period of 2071-2100, which is more extreme, has been processed graphically as a percentage change in the mean monthly discharges for the different scenarios compared to the reference period (Fig. 4).

According to all the climate scenarios, the expected changes in the mean monthly discharges are almost the same. This means an increase in the mean monthly discharges in the winter and a decrease in the spring and summer periods (Fig. 4). Such results were also documented in the study by Middelkoop, et al. [10]. Autumn is the only season that varies, depending on the use of the climate scenario. The ALADIN-Climate and MPI scenarios assume a decrease in the autumn discharges (October); the KNMI scenario assumes an increase in the discharges since September.

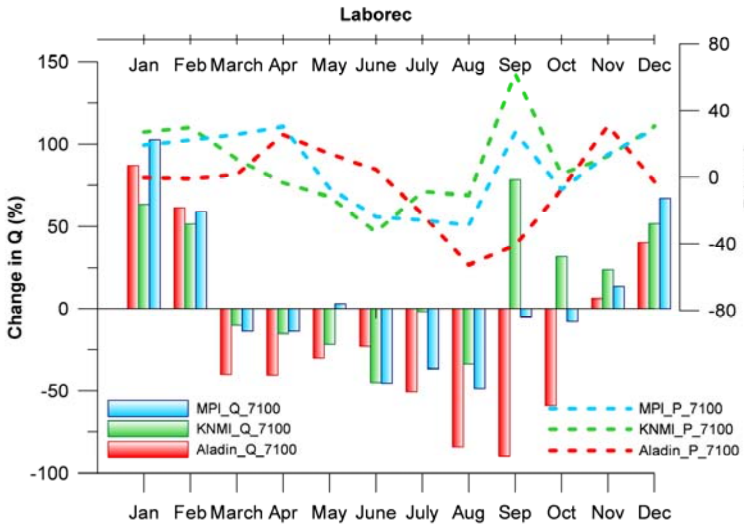


Fig.4 Changes in the mean monthly discharges for the period 2071-2100 compared to the reference period (1961-1990)

Tab. 1 Changes in the mean monthly discharges for the periods 2021-2050 and 2071-2100 compared to the reference period (1961-1990)

Change in Q compared the reference period(m3/s)	Jan	Feb	March	Apr	May	June	July	Aug	Sep	Oct	Nov	Dec
Hron												
Aladin_2150	1.8	6.6	-3.7	-5.7	1.0	1.9	-0.9	-2.1	-1.6	-1.9	1.6	5.2
Aladin_7100	6.1	7.9	-14.0	-8.4	-3.9	-1.7	-3.2	-3.4	-2.3	-2.8	0.4	4.1
KNMI_2150	0.7	4.0	-4.8	-7.0	-2.7	-2.1	3.7	-1.8	5.3	4.0	-1.1	2.2
KNMI_7100	8.4	8.9	-2.5	-3.4	-3.2	-6.8	-0.2	-3.4	6.6	3.2	2.7	7.9
MPI_2150	0.8	6.9	-5.0	-6.7	0.0	-3.7	-0.3	-1.5	4.6	0.6	-2.6	3.8
MPI_7100	10.4	8.7	-3.9	-3.6	0.4	-6.7	-3.8	-4.4	-0.5	-1.0	1.5	7.2

The most significant decrease in the mean monthly discharges is expected in August and September according to the ALADIN-Climate scenario. The decrease could be almost 90%, which represents a decrease of 3.2 to 3.4 m^3/s.

The most significant increase in discharges will be for all the scenarios in the month of January. It will be caused by the changing of snowfalls into rain due to increasing air temperatures. With the increasing air temperatures intensive snow melting could also occur. The most significant increase in January is expected according to the MPI scenario (more than 100%, i.e., 10.4 m^3/s).

The results of the changes in discharges were also compared with the results indicated by the European Environment Agency (EEA). In 2012, the EEA processed simulations for the all of Europe on the basis of the 11 RCM and SRES A1B emission scenarios. The results also show an increase in discharges in the winter period and a decrease in the spring and summer periods. These results are documented in the figure, which indicates the percentage comparison of changes in discharges for the period 2071-2100 compared to the reference period 1961-1990 (Fig. 5).

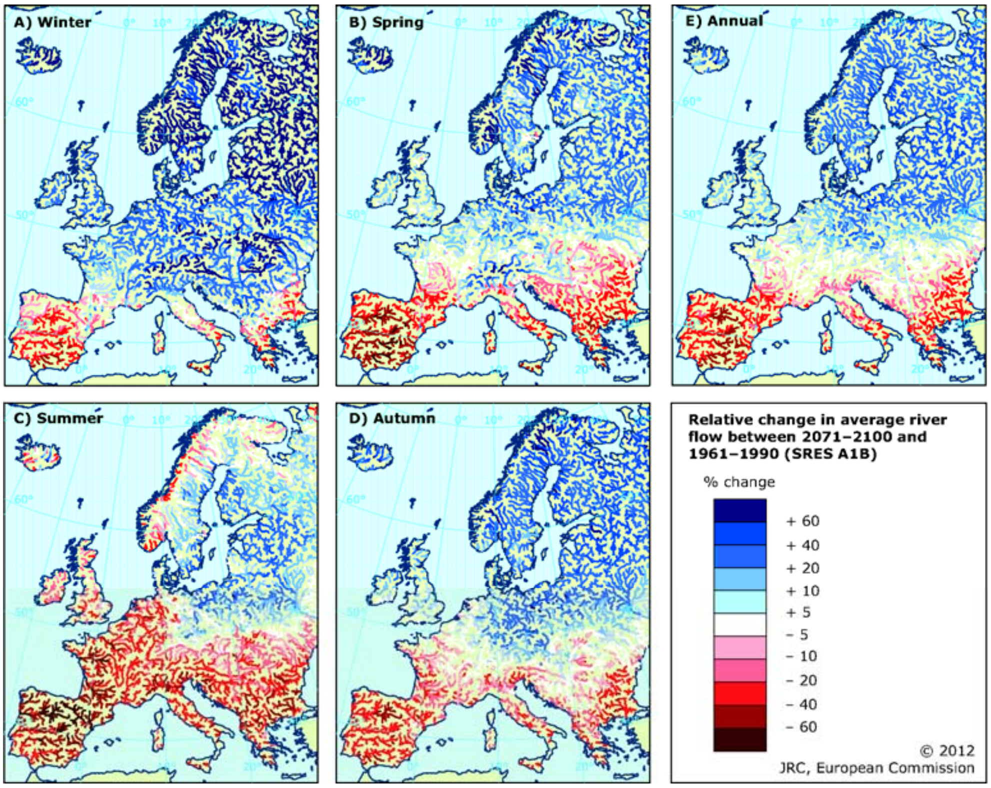


Fig. 5 Percentage comparison of changes in discharges for the period 2071-2100 compared to the reference period 1961-1990 according to the EEA.

5 CONCLUSION

While the overall development of discharges within the same year is compliant, i.e., an increase in the winter period and a decrease in the summer and spring periods, the percentage changes are often different. It may be noted that when dealing with the problem of the impact of climate change on runoff processes, it is advisable to use several climate scenarios and choose the most convenient once according to the focus issues.

On the basis of these results and new knowledge it is clear that climate change could have a significant impact on the runoff processes of Slovak rivers. Probably the most significant changes and the most commonly problematic nowadays are flood situations, which are currently very challenging from the perspectives of water managers, foresters and land managers. The results show an overall increase of up to 100% in the mean monthly discharges for the spring period at the catchment of the Laborec River until the year 2100. Based on the above results, which have been predicted by using more climate scenarios, it can be stated that the issue of flood protection, which is already a very hot topic today, will also require much attention in the future.

Acknowledgement

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CLIMATE CHANGE AND FLOOD PROTECTION PLANNING

Zuzana Macurová[1], Jana Kráľová[1] and Viliam Macura[1]

Abstract

Possible changes in drainage conditions due to climate change are currently one of the main sources of uncertainty in water resources management and flood protection planning.
In this study the impact of climate change was evaluated on the Váh River basin until Liptovsky Mikuláš. The study was aimed at evaluating the impact of climate change on runoff processes using three regional climate models - ALADIN-Climate, KNMI and MPI. Long-term change in average monthly flows and their allocation within the year, change in the design maximum flow rates and average daily minimum flows in summer were assessed.
The results for the period of years 2071-2100 show an overall increase in long–term average monthly discharges for the spring and winter and decrease in summer, when we can also expect an increase in the maximum design flow according to the climate change scenarios. From these findings it can be stated that the issue of flood protection, which is already a very hot topic today, will require much attention in the future.
According to the scenarios of climate change it is also expected that in the summer period the minimum average daily flow will significantly decrease. This change itself will have a significant impact on the quality of aquatic habitat.
Based on the results it can be concluded that in terms of flood protection not only the change in maximum flow rates or changes in flow redistribution within the year is important. When planning the flood protection or water management plans it is important to take into account the areas that could be threatened by such interventions, especially when we expected the deterioration in their quality even at the current state. Individual measures should therefore take into account the ecological view on the issue.

Keywords

Hron model, RHABSIM, climate change, climate scenarios

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1 INTRODUCTION

At the present, the issue of climate change is a very frequent term especially in the relationship with global warming. Possible changes in drainage conditions, due to climate change, are currently one of the main sources of uncertainty in water resources management and flood protection.
Floods in Central Europe have caused deaths and widespread property damage across parts of the Czech Republic, Germany and Austria. Such events are likely to increase in Europe for several reasons including climate change, according to recent assessments from the European Environment Agency (EEA).

Floods, storms and other hydro-meteorological events account for around two thirds of the damage costs of natural disasters and these costs have increased since 1980, according to a recent EEA assessment of climate change impacts in Europe.

Increased flooding is likely to be one of the most serious effects of climate change in Europe over coming decades. Some of the conditions which may contribute to flooding in the area of Slovak republic are highlighted in an Eye on Earth map from the European Environment Agency (EEA) (Fig. 1).

Fig. 1 Change in annual mean number of days with extreme precipitation (> 200 mm /day) for 2071-2100 compared to the reference period.

In presented study the impact of climate change was evaluated on the Váh River basin until Liptovsky Mikuláš. The study was aimed at evaluating the impact of climate change on runoff processes using three regional climate models - ALADIN-Climate, KNMI and MPI. Long-term change in average monthly flows and their allocation within the year, change in the design maximum flow rates and average daily minimum flows in summer were assessed.

2 METHODOLOGY

Methodology of the study is divided into a four larger units, which are: the calibration and validation of the Hron model [1] for the Váh and Hybica River basins, processing the outputs from climate scenarios, evaluation of the simulated results of average, maximum and minimum flows and the assessment of the impact of changed flows on the biota of the river.

3 CALIBRATION AND SIMULATION FOR THE VÁH RIVER BASIN

First, the calibration of the Hron model for the period of years 1981-1995 was done. According to the calibrated parameters and three climate scenarios - ALADIN-Climate [2], KNMI and MPI [3] the simulation for the Váh River basin, for the reference period 1961-1990 and two future time horizons (2021-2050 and 2071-2100), was carried out. All three climate scenarios are based on the emission scenario SRES A1B (IPCC, 2007). The simulation of mean monthly runoff for the reference period (1961-1990) and the future time horizon (2071-2100) show that there will be a decrease in runoff during the summer period. An increase can occur during the autumn, winter and early spring period due to changed precipitation and air temperature, where we can consequently expect an increase of flood events (Fig.2).

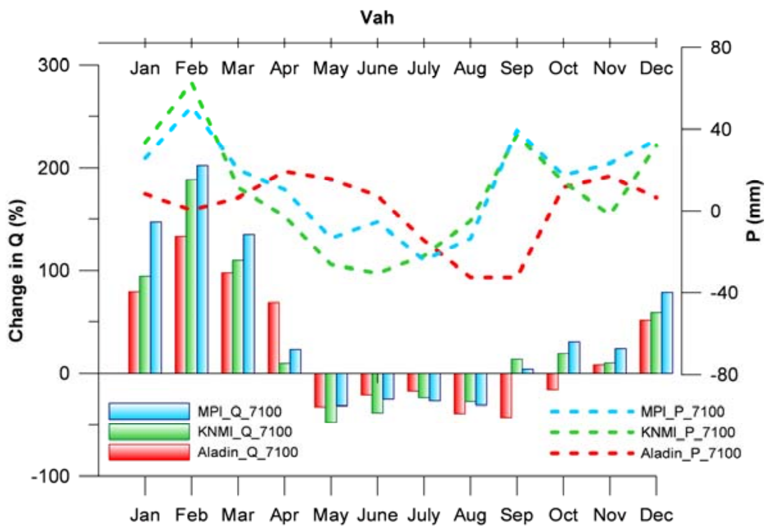

Fig. 2 Comparison of the flow reta in the reference period (1961-1990) and the future time horizon (2071-2100) for the Váh River basin.

For the period, where we assume the decrease in long-term mean monthly runoff we have done the study of changes in maximum flow. This study was based on the frequency analysis and climate scenario MPI that was the only one, which simulates the occurrence of long-term yearly maximum flow (Fig. 3).

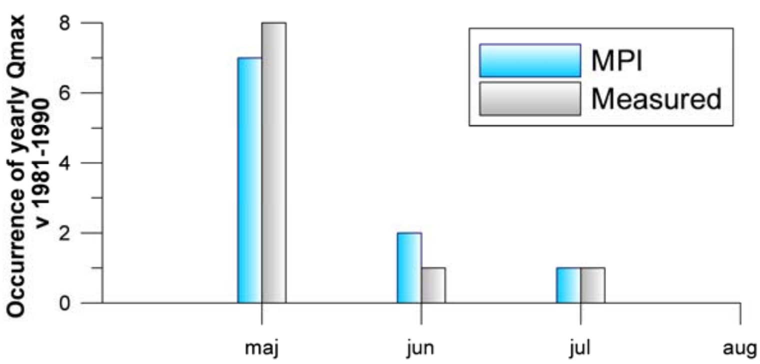

Fig. 3 Analysis of the reliability of the different climate scenarios to simulate the occurrence of long-term yearly maximum.

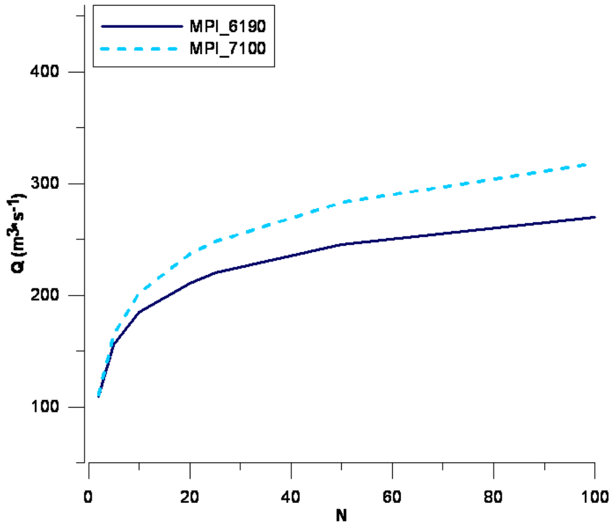

Fig. 4 Simulation of the change in maximum flow during the summer period.

According to the results, the maximum runoff for the Váh River basin will increase (Fig.4). Therefore it is expected that there has to be a significant change (decrease) in minimum flows during this period. It is evident that for the summer period the change in runoff will be the most significant according to the ALADIN-Climate model compared to the KNMI and MPI models (Fig.2). Therefore, the change in minimum flows was evaluated according to the outputs of ALADIN-Climate scenario.

Changes in the rate of flow under the influence of climate change have been analyzed in terms of the quality of aquatic habitat for the Hybica River basin, which represents a mountain stream ecosystem.

Focusing on smaller mountain streams has the following reasons:

- Mountain streams are the most vulnerable ecosystems in terms of temperature and flow changes. Minimum flows in summer are crucial to the overall state of the river biota. At low flow rates, the water quickly overheats, which has a significant effect on the oxygen regime, which is sensitive to the biota.

4 CALIBRATION AND SIMULATION FOR THE HYBICA RIVER BASIN

From the measured data, it was determined that the minimum flow rates and simultaneously the most demanding conditions for biota of the river occur in August. Calibration of the model was performed based on the measured data of air temperature and precipitation in each station from 01.11.1994 to 31.10.2002. Evaluation was based on the Nash-Sutcliffe coefficient [4] and especially on graphical results. This graphical evaluation was mainly focused on the minimum flows in August where we were able to achieve almost perfect correspondence between the calibrated and measured data (Fig. 4a, b). The match was for the long-term monthly average values as well as for a particular value of the minimum daily flow that occurred for measured and for the calibrated period at the same day, i.e., 27.8.1995; the measured flow rate at that date had a value of 0.156 m^3/s and the calibrated 0.151 m^3/s.

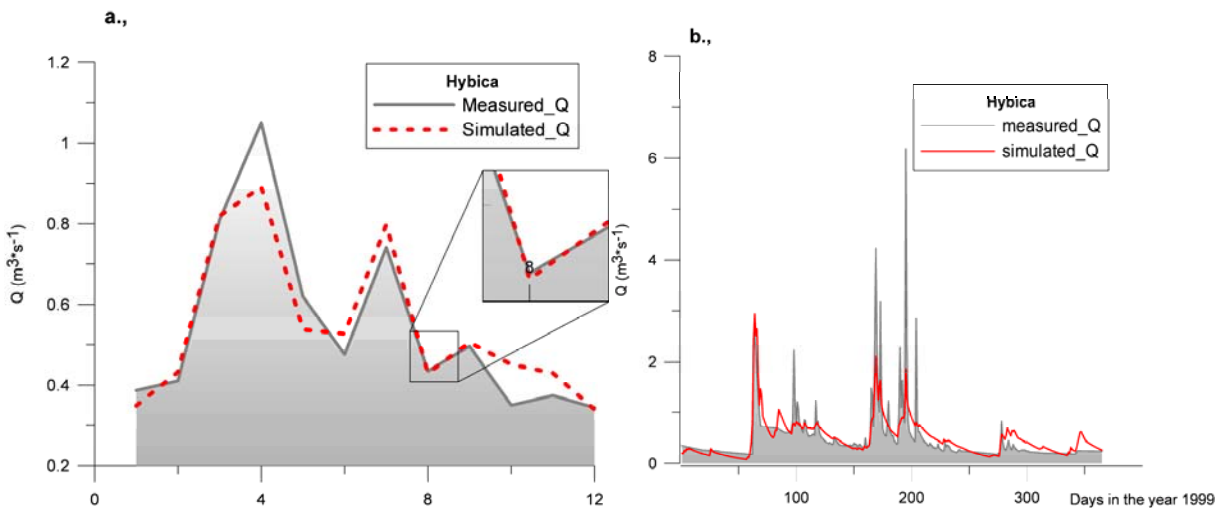

Fig. 4 Comparison of measured and calibrated flow rates as long-term average values (a) and as daily values of flow rates (b) in the final profile of Hybica River basin.

Based on the calibrated parameters and the outputs from the ALADIN-Climate scenario the simulation for the reference period 1961-1990 and two future horizons 2021-2050 and 2071-2100 was carried out.

The minimum daily flow rate for each 30-year period was evaluated. For the period 2021-2050, the minimum flow rate was 0.17 m^3/s, which occurred on the 29th of August 2025 and for the period 2071-2100 the minimal flow was 0.06 m^3/s and can be occurred according to the simulation on 22nd of August 2079.

5 IMPACT ASSESSMENT OF THE CHANGED FLOW UNDER THE CHANGE OF CLIMATE CONDITIONS ON AQUATIC BIOTA OF FLOWS

To assess the impact of climate change on aquatic biota of river the IFIM methodology and model RHABSIM were used. Using this model, the condition of the river and its suitability for the biota under the changed minimum flows (changed depth and velocity of the flow) were simulated.

5.1 IFIM methodology and model RHABSIM

For modeling the flow impact on the quality of aquatic habitat the data from the Department of Land and Water Resources Management were used. There were evaluated 43 rivers and 52 representative sections of Mountain Rivers in Slovakia. Measurements are adapted to the requirements of modeling the aquatic habitat quality by the methodology IFIM in the program RHABSIM (detailed topographic, hydraulic and ichthyologic measurements on selected reference sections of rivers), so the forecast of impacts of climate change on the quality of aquatic habitat will use the above methodology and model.

The IFIM methodology divides the basic habitat parameters of the river into abiotic and biotic parameters. Abiotic parameters resulting from the data of hydraulic model, and specifically in the field of impact of climate change it is also regional scenarios of climate change in the short (daily) time step, which allow to review possible changes in extreme phases of the runoff processes in the dry season. Biotic parameters of river habitat are represented by the fishes as bio-indicators. The relationship of abiotic and biotic characteristics is represented by the suitability curves of individual fish species. One of the most discerning areas of aquatic habitat modeling is the determination of suitability curves [5].

From existing research of the Department of land and water resources management implies the hypothesis that if there is a geometric and dynamic similarity of rivers, there should also be biotic similarity. Correlation of abiotic and biotic characteristics is the first step to confirm the hypothesis, which is the basis for a generalization of biotic characteristics of the rivers. So far, according to the evaluation of the reference sections of streams in Slovakia this correlation was confirmed.

Based on the above results it was possible to deduce the biotic characteristics of the flow represented by ichthyofauna in conditions affected by climate change. Therefore, it was possible to evaluate the change in the quality of aquatic habitat in terms of the impact of climate change on stream flow regime. In Fig. 5, 6 and 7 a gradually change in the quality of aquatic habitat with changes in flow resulting from climate change scenario are shown. Habitat quality is assessed by individual cells. The range of suitability level is shown on a color scale from 0 to 1. The highest level of suitability has a value of 1 and the lowest level of suitability is indicated by dark black color and has a value of 0. Figures 4 to 6 show that the quality of habitat for two modeled future periods logically decreases with a decrease in flow.

6 RESULTS

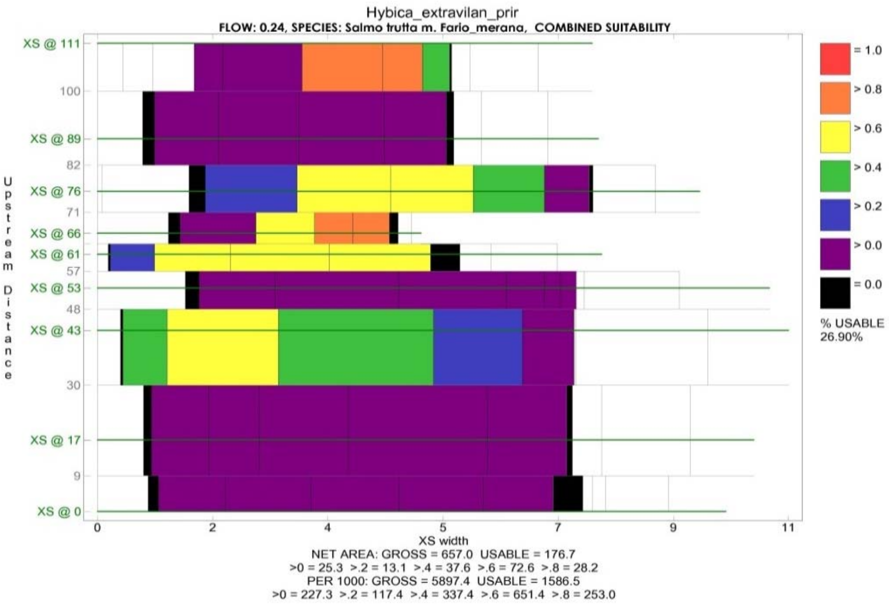

Fig. 5 Evaluation of the quality of aquatic habitat by model RHABSIM for the Hybica River at a flow rate Q = 0.42 m^3/s.

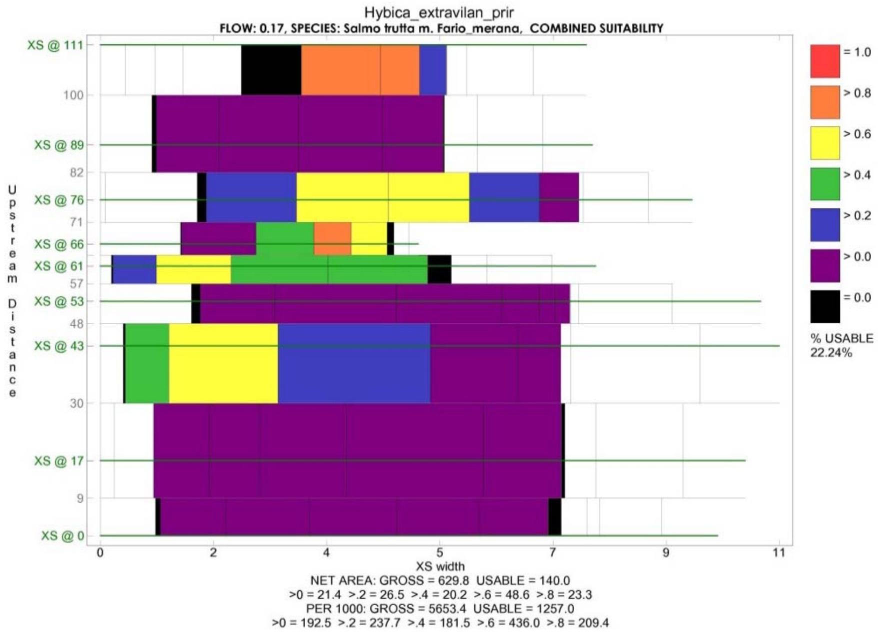

Fig. 6 Evaluation of the quality of aquatic habitat by model RHABSIM for the Hybica River at a flow rate Q = 0.17 m^3/s.

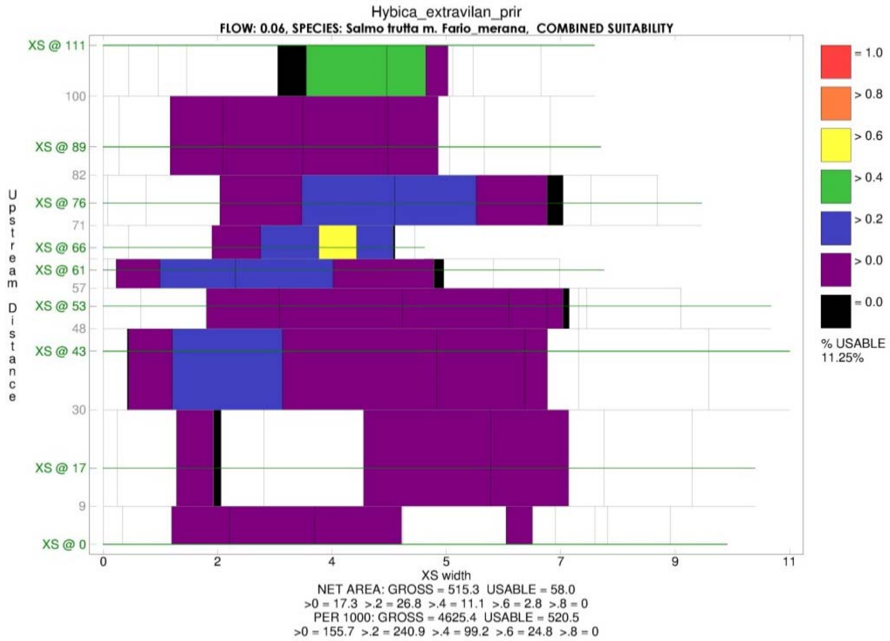

Fig. 7 Evaluation of the quality of aquatic habitat by model RHABSIM for the Hybica River at a flow rate Q = 0.06 m^3/s.

At minimum flow the quality of micro-habitat, where the biota of the river is focused, is the most important. The rest of the river does not have a significant impact on the maintenance of the river biota. Otherwise interpreted, it is important that the flow has a satisfactory habitat (suitability rate of 0.6). The figures 6 and 7 show that the quality of habitat for the modeled future decreases with a decrease in flow compared to the reference period (flow rate of 0.42 m^3/s - the flow rate at the time of the ichthyologic measurement ($Q_{365} = 0.037$)). At a flow rate 0,06 m^3/s (Q_{355}) is a significant change. Suitable habitat is reduced only on a single cell. It means, that the flow rate of 0,06 m^3/s would have a serious impact on the reduction, as well as the generic change, of the aquatic habitat of the river. Based on the results it can be concluded that according to the scenario ALADIN-Climate there is a significant change in the quality of aquatic habitat.

7 CONCLUSION

From these findings it can be clearly stated that the issue of flood protection, which is already a very hot topic today, will require a close attention in the future. In recent years there have been a large number of conferences, where participants managed to bring together the different views and opinions on this issue. Numbers of solutions have been proposed. Based on the results above it can be noted that in relationship with the problematic of flood protection we should not have in mind only the change in maximum flow rates or changes in flow redistribution within the year. It is important when design the flood protection to keep in mind the possible threat of natural biota of flows.

Acknowledgement
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IMPACT OF ANTHROPOGENIC INTERFERENCES ON ECOSYSTEM OF ZELIENKA WETLAND

M. Pásztorová[1], J. Skalová[2] and M. Jarabicová[3]

Abstract

Wetlands are areas that are in the world, but also in Slovakia, considered as very rare and relict communities. But nevertheless, they are among the most endangered ecosystems because they have been in past and also nowadays exposed to negative anthropogenic impacts. These impacts have a consequence of changes in the water regime of wetlands and their environmental conditions. Therefore, it is necessary to deal within the protection of the natural environment with the solution of quantification of changes in the water regime of wetlands to protect these unique natural ecosystems. Analysis of anthropogenic interferences impact on wetland ecosystem has been applied to Zelienka wetland, which was in 1980 declared as a national nature reserve.

Keywords

National Nature Reserve, wetland, nature protection, anthropogenic interferences, restoration measures, soil water regime

1 INTRODUCTION

Civilization trends of 20th century in the world, but particularly in Europe, have led to an unfavourable conversion and often to the liquidation of existing wetlands. This was especially due to efforts to transform wetlands into production, or otherwise "wisely" utilized land. The situation became critical, what results in the need for access to international cooperation for the conservation and wisely use of wetlands. Protection and wisely use of wetlands has become part of the government policy of many member countries of Ramsar Convention at all

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levels, including Slovakia. In recent years the question of protection of wetlands and also the restoration of damaged and destroyed wetlands get to the fore of interest.
As the most endangered wetland ecosystems in Slovakia, and also in the world, are classified peatlands, from which remnants remain only at present. In the context of increasing area acreage of agricultural land in the previous period in Slovakia large-scale drainage of waterlogged areas was implemented. The result was the rapid disappearance of these extremely rich and productive ecosystems. To such relict communities can be also include peatland in Zelienka National Nature Reserve (NNR).
The negative impacts of anthropogenic changes on wetlands have consequences of the need to deal with the protection of the natural environment and the solution of quantification of changes in the water regime of wetlands to protect these unique natural ecosystems. It demonstrated the need of the design and implementation of restoration measures to restore the wetland water regime and its ecological functions to ensure the protection of the environment.

2 MATERIAL AND METHODS

Zelienka NNR represents a peat bog wetland community of relic origin with an open water surface. Occupies an area of 82.52 ha, whereas the area of peat bog itself is 60 ha. It is located in the Protected Landscape Area (PLA) Záhorie and on the basis of territorial and administrative division it is a part of Trnava region, Senica district and specifically lies in the cadastre of Lakšárska Nová Ves (Fig. 1). It is situated under the Kobyliarka in Záhorská basin, on the eastern edge of the Šaštínsky forest (200-210 m above the sea level) and it is a springfens depression with originally predominant peat vegetation, due to drainage largely destroyed, now with preserved rare phytocoenosis of birch - oak - pine forests [1].

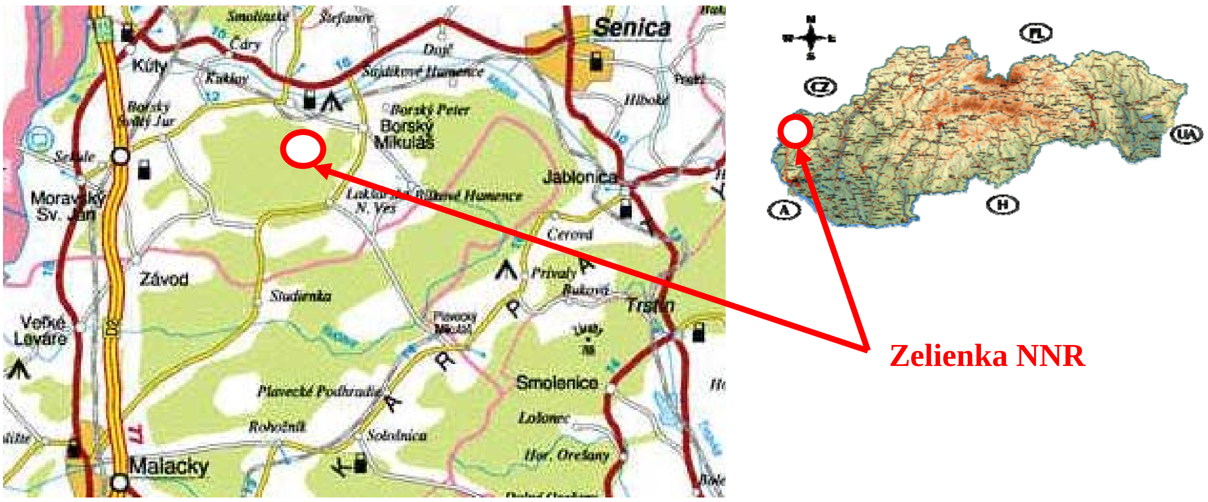

Fig. 1 Localization of Zelienka NNR

In 1980 was Zelienka NNR declared as one of the last preserved peat bog wetland community on area of Slovakia. It is very special type of peat bog, because it was not created by the presence of rainwater, but by the presence of surface and groundwater acting on the surface.
Zelienka NNR belongs to the river basin of Morava River, which left-side tributaries on area of Slovakia are spring in the north foothills of the White Carpathians and the Small Carpathians. Catchment of the studied wetland is shown in Fig. 2. Original river network in

the area of Záhorie PLA was in the past significantly altered by human intervention (the movement and flow control, establishment of irrigation and drainage channels). Intensive development of melioration adjustments in the Záhorie area was mainly during the years 1960 - 1964. The main focus of adjustments was consisted of soil drainage, what expresses the area of drained land - 7 039 ha. Water flows, including the drainage channels, were adjusted in length of 413 km [2]. Also during the years 1979 - 1984 was due to forestation of adjacent area to the site built up a network of drainage channels. Originally from this wetland flowed out Petrmez creek, but parallel to it was built drainage channel, what lead to the gradual drying out. At present, any natural water flow do not leak in this way, but Šaštinský stream is in the vicinity (about 600 m east), that during the period with the higher flows subsidizing the wetland with water [3].

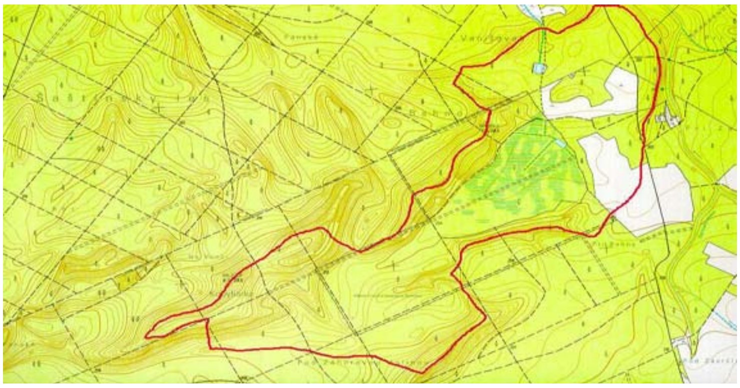

Fig. 2 Catchment of Zelienka NNR

3 RESULTS

In the last century there has been an increase in area acreage of agricultural land to ensure food self-sufficiency. Therefore, large-scale drainage of waterlogged areas in Slovakia was implemented. In connection with the contemplated Zelienka locality, its drying by drainage channels wad occurred. At present, when the surrounding land is not interesting to agricultural use, the question of restoration measures gets to the fore of interest.

In the 80s main drainage channel in the area between the edge of the bog and forest road was partially buried with the soil by workers of State Nature Protection, which was positively reflected some improvement of the water regime in the future. More precise measurements of groundwater level regime were not carried out during this period. But it is obvious that the negative consequences of drainage on peat bog communities would become even more pronounced without this intervention.

3.1. The solution elimination of the negative consequences of human activity on water regime of wetland

Decrease of groundwater levels and the rapid drainage of rainwater had a negative impact on the total hydrological regime, namely on the reducing of evaporation from the area, thus also on the total humidity of climate, on the change of runoff conditions, what leading to a violation for a thousands of years constituting balance of hydrological water balance of territory.

The distinct decrease of groundwater levels caused by undue interferences in the water regime resulted in notable retreat of the most important species of phytodiversity of the site,

especially several ecologically easily vulnerable hygrophyte and hydrophyte elements. In the site have persisted only a few fragmentary developed wetland biotopes. Compared to the optimum condition of the water regime (70s of the 20th century) flora habitat suffered the most. It was a habitat of transition peat and quaking bogs (Natura 2000, the closer characteristics is given by Viceníková and Polák, 2003 [4]). This European important habitat on the Zelienka already do not exist, the problems of restitution (rediscovery) are very difficult to remove. But it should be noted that after 1971 there has also been some change in climate conditions in the wider area, such as reducing of the annual and seasonal rainfall, decreasing of relative humidity and increasing of potential evapotranspiration. This led ultimately to the reduction of soil moisture and a decrease in groundwater levels. Negative state peaked during the hot summer 1992 and extremely dry year 1993. Moreover, since 1990 there were some very warm years and vegetation period and until 2009 there was no one even colder summer than the long-term average.
To eliminate the negative anthropogenic impacts conducted until 2000, which led to a gradual reduction of wetland and its ecological functions, have been designed and implemented restoration measures. In order to monitor their effects on the dynamics of the water level in the area of interest water gauge was in the drainage channel on the northeast edge of wetland equipped in March 2000.
The first restoration measure which was aimed at the restoration of the water regime in wetland was implemented in 2000 through the change of function of drainage channels to irrigation. Realized technical measure consisted of building dams in the main drainage channel to achieve an increase in water level and reduce the hydraulic gradient of water level (Fig. 3). For the creation of dams clay-sandy material from the original excavation was used, which appeared to be sufficient. If necessary, the dams could be sealed, e.g. by geotextile. During the years 2000 - 2001 were built 12 dams. The volume of used clay-sandy material in the dams is about 25 m^3.

Fig. 3 An example of built dam using the surrounding natural material

The creation of such dams has a positive effect on the water regime of peatland in a relatively short time. It has a consequence of increasing water level by 56 cm during the period September 2000 - July 2001 (Fig. 4), what made a complete compensation of water level decrease due to the construction of drainage channels in 80s, when the groundwater level decrease more than 50 cm [5]. Also in 2002 a positive trend (increase) of water levels of

wetland due to functional dams of drainage channels continued, what caused water retention in wetland. The positive trend of restoration measures implemented on area of Zelienka wetland is also documented by Fig. 5.
The total increase in the retention volume of peatland (the amount of water accumulated on surface of peatland) in the dry period in 2001 is representing by volume of 100 000 m^3. This amount of water can participate in small water hydrological cycle, in balancing of temperature differences and humidification of climate, especially in periods of deficient rainfall.

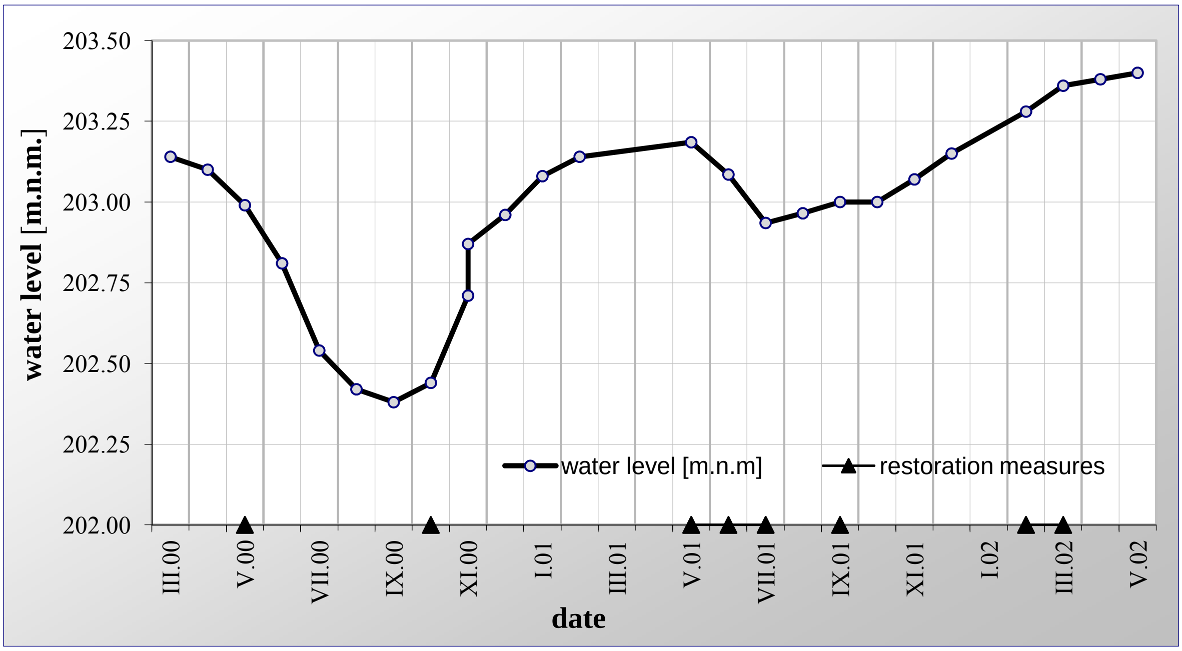

Fig. 4 The impact of restoration measures to change of water level in Zelienka

3.2. The impact of anthropogenic interventions on the ecosystem of Zelienka wetland and impact of restoration measures on ecological condition of wetland

In the state before the drainage of Zelienka wetland in the second half of the 70s, phytocoenosis of transitional bogs dominated at site, especially Peucedano Communities - Cariteum lasiocarpae and Caricetum elatae. There were dominated numerous high sedge, subalpine sedge (Carex lasiocarpae), diverging sedge (Carex appropinquata), panicle sedge (Carex paniculata, etc.). Davall sedge (Carex davalliana) was physiognomically significantly applied in this area, with extensive coverage of sphagnum and mosses. During this period there was no problem to found in this site relict, in origin mountainous, an extremely important species of flora of Záhorie - Rhynchospora alba. On peat bog carpet was bound Drosera rotundifolia, Viola palustris, Comarum palustre, Pedicularis palustris, Menyathes trifoliata and many other species. At the edges of water areas of peat bog widely grown protectionistic and scientifically most prominent plant of site - relict Rhynchospora alba [6].
Longer term terrestrial ecophase that in the Zelienka site occurred after the construction of the drainage channels heavily disturbed wetland phytocoenosis of this site. Dominant high sedge (Carex elata) was replaced by degrading and more xerophilous community of Molinio - Arrhenatheretea. Available meadow areas began to increasingly settle by Calamagrostietum canecens, which was for the original habitat of the transitional peat bog disposal.

May 2002 May 2002

April 2005 October 2006

May 2009 May 2009

Fig. 5 The positive trend of restoration measures on water regime of Zelienka wetland

Currently, as a result of many years of adverse water regime, originally dominant specific wetland flora whose presence on Zelienka site was the main motive to declare the wetland as protected area is more or less destroyed (Fig. 6). In places where historically dominated the mentioned wetland plant species, there was a significant movement to more xerophilous plant communities dominated by air raids of pines, birches, partially alders. It should be noted that in 1977 the current dry habitat were permanently flooded [5].

Obr. 6 Degradation of the site due to occurrence of xerophilous plant communities

From the protectionist-important species remained on this site Hydrocotyle vulgaris. Further, it was observed a small meadow fragment threatened by gradual overgrowing by seeding trees. Although the water regime was not optimal, still are here preserved fragments of Caricetum elatae with Cirsium palustris and Carex elata. Crops of Caricetum elatae bound to long-term flooding are also developed only in fragments [6].
From the conservation view are in the area of interest optimally developed fen alders (Alnetea glutinosae). Disposal of drainage channels benefits to their current positive development. Alder are optimal flooded and classified to the most representative species within Záhorská lowlands. From the tree species significantly dominant Alnus glitinosa, this is in the front of the site added by birch. Batula pubescens is also present, in some areas mixed with oak, furthermore there are more species of willow (Salix). From the shrubs can be mentioned Frangula alnus, Sorbus aucuparia and Viburnum opulus. In the herb floor is significantly applied yellow iris (Iris pseudacorus), in some areas is present proprietary Hottonia palustris, Utricularia australis, Cardamine amara, Scirpus sylvaticus, Caltha palustris, in roots of alders applies distinguished Dryopteris cristata, Thelypteris thelypteriodes and frequent is also Thysselinum palustre. Based on the views of the listed habitat in the last years recurrence of Menyanthes trifoliata is also probable.
The positive impact of restoration measures was reflected by redevelopment of community of sphagnum, which have been also documented in wetland (Fig. 7).
It can be stated that up to now carried out restoration measures aimed at improving the state of hydro-ecological elements of the site significantly contributed to the restoration of the original vegetation, but this process is time-consuming. It would be appropriate to continue in these restoration interventions that should be enhanced by the removal of seeding trees from the site to ensure reliable development of phytodiversity. After active conservation interventions it can be expected at the edges of the drier pine forest the occurrence of protected Blechnum spicant. However, all restoration measures need to be modified in the light of ongoing climate change, which in this area also leads to a gradual increase of average air temperature, change of rainfall regime, melting of snow cover and ultimately a change in evapotranspiration and water balance.

Obr. 7 Sphagnum carpet

4 CONCLUSION

The negative impact of anthropogenic activities on water regime of Zelienka NNR in the past managed to eliminate by implementation of simple restoration measures implemented since 2000, which consisted of changing the function of drainage channels to irrigation, building dams in the main drainage channel. Up to now carried out restoration measures aimed at improving the state of hydro-ecological elements of the site, significantly contributed to the restoration of the original vegetation. But it is a time-consuming process and results could not be fully reflected in such a short time after the restoration interventions. Therefore it would be appropriate to continue in these revitalization measures, i.e. maintain functionality of built dams that cause water retention in wetland. It would be also appropriate to revitalize the creek springing in wetland ecosystem, which can contribute to restore of the origin natural water regime and ecological system of Zelienka wetland.

Acknowledgements

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WAVELET ANALYSIS OF MONTHLY DISCHARGE AND SUSPENDED SEDIMENT LOAD ON THE RIVER SAVA

K. Potočki[1] , N. Kuspilić[2] and D. Oskoruš[3]

Abstract

Continuous Wavelet Transform (CWT) is a powerful technique that enables analyzing periodic localization of hydrological time series. In this study CWT is used to analyze multi-scale variations in monthly discharge and suspended sediment load on 3 Gauging Station on the Sava River for the period from January 1979 to December 1993. The River Sava is one of the most important rivers for water management in Croatia and better understanding of variation in discharge and suspended sediment has practical relevance in better understanding of anthropological impacts and climate variation on fluvial processes.

Keywords

Discharge, the Sava River, sediment load, wavelet transform

1 INTRODUCTION

Geophysical time series, such as hydrological time series from fluvial systems, are generated by complex natural processes. Investigation of predictable components of hydrological time series (e.g. trend and periodicity) is of great interest. Periodicity can be analysed in time and frequency domain. For analysis of periodicity in frequency domain traditionally is used Fourier transformation (FT) and lately also continues wavelet transform (CWT). FT is based on underlying assumption of stationarity in time of hydrological time series. However, CWT enables simultaneous time-scale modulation and consequently analysis of non-stationarity [1], [2]. Application of CWT in hydrological series is used for identification of seasonal and

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interannual variability in temperature, precipitation and discharge; water quality indicators and climate and anthropogenic impact on that variability [3], [4], [5],[6] ,[7], [8].

River Sava has a great importance in water management of Croatia and better understanding of hydro-meteorological time series of the Sava basin is of crucial importance. There has been considerable research on sediment load and runoff changes in the Sava river basin near Zagreb but it has to be noted that conventional statistical methods were been used which are insufficient in evaluating changes in periodical components [9]. Certain iinvestigations of climate changes in the Sava river basin have been made and some have showed decrease in trend in precipitation and discharge series [10],[11].

Anthropological impacts and climate changes in fluvial systems can result in changes in discharge variability [12],[13]. Modification of the river channel geomorphology is connected with increase and decrease in discharge and sediment load. Deepening of the Sava riverbed near Zagreb urbanised area has been documented. This process could result from the construction of HPP in upper reaches of Sava in Slovenia and extensive gravel excavation in aquifer near Zagreb [14]. The studies of anthropogenic and climatic influence on suspended load in the Yangtze River basin have shown that monthly maximum sediment load is more sensitive to external influencing factors than monthly minimum and mean sediment load [15]. In accordance with that, in this study maximum monthly data of discharge and sediment load has been analysed in terms of differences at the following Gauging stations: GS Podsused, GS Rugvica and GS Slavonski Brod.

The main objectives of this paper are: (a) to detect and describe changes in periodicity of the runoff and sediment load time series on GS Podsused with longest continuous measurement of sediment data (including the comparison of suspended sediment concentration and discharge and sediment load data – both on mean and maximum monthly data); (b) to compare changes in periodical patterns for maximum monthly discharge and sediment load on 2 GS in Zagreb urbanised area (GS Podsused and GS Rugvica) with patterns in lower reach of the Sava River, i.e. on GS Slavonski Brod - this has been done for common continuous time period without interruptions in measurement on all three GS; and (c) to discuss possible anthropogenic and climatic influences on the periodic patterns of runoff and sediment load. The specific contribution of this paper lies in the analysis of periodical changes in suspended sediment load.

2 DATA

Monthly suspended sediment load and discharge time series data has been taken from 3 GS on the River Sava: GS Podsused (GS PO), GS Rugvica (GS RU) and GS Slavonski Brod (GS SB) (**Fig. 1**). The data was provided by Meteorological and Hydrological Service of The Republic of Croatia. In some years there are interruptions in measurements of discharge and suspended sediment time series on GS downstream from Zagreb: GS RU and GS SB. On GS Podsused discharge, suspended sediment load, as well as suspended concentration data was analyzed for continuous time interval of 32 years i.e. from 1979 to 2010. Analysis was conducted both on monthly mean and monthly maximum data. Since suspended load is the product of discharge and suspended concentration, for comparison of all the three GSs (GS PO, GS RU and GS SB) the common time span with longest continuous measurement of both time series was taken – from January 1979 to December 1993 (**Tab.1**). The analysis and comparison of periodical patterns on all the three GSs was conducted on maximum monthly data, as was done in studies in China[15]. The sediment load in this study refers to suspended sediment only.

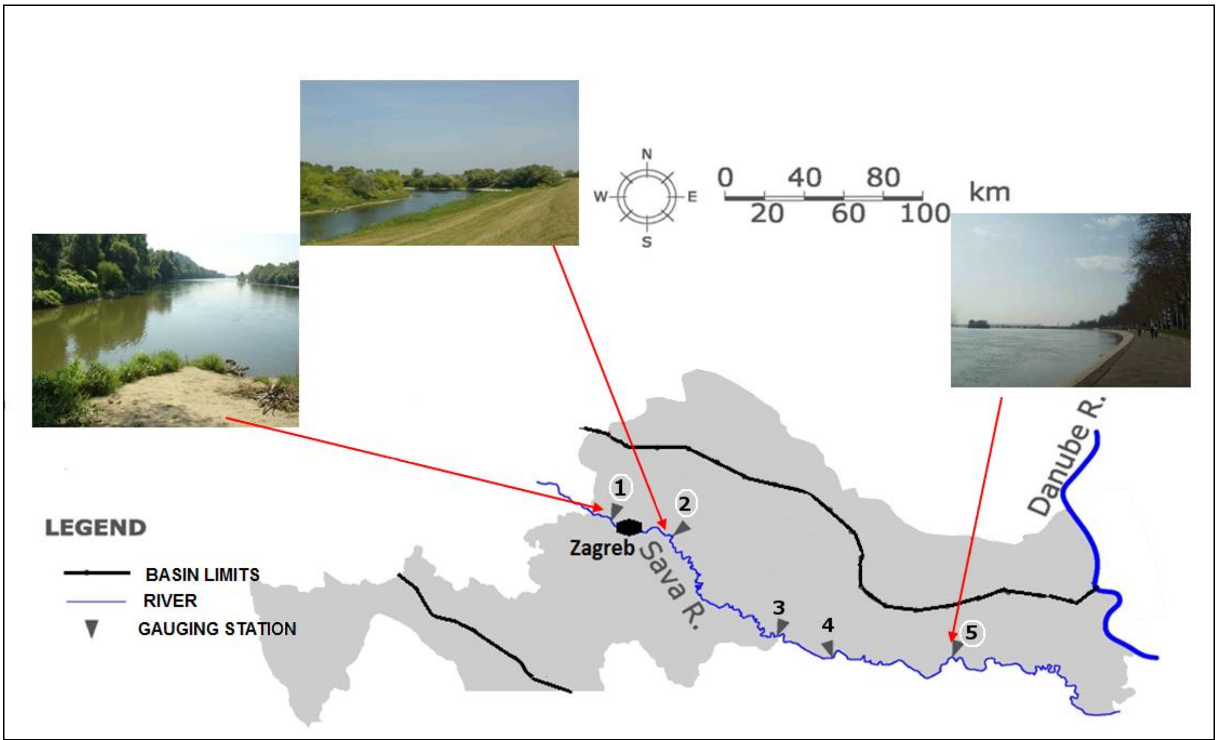

Fig. 1 Location of the analysed gauging stations on the River Sava

Tab. 1 Hydrological record on gauging stations (GS) on river Sava with suspended sediment load (SSL) monitoring. GSs with continuous common period from 1979 to 1993 are marked in bolded font.

Station numb.	Station	Location (rkm)	Drainage area (km2)	SSL measurement	SSL measurement interruptions	Analyzed time interval
6	**Podsused**	**675.4**	**12316**	**1979-2012**	**-**	**1979-2010 1979-1993**
7	**Rugvica**	**636.3**	**12730**	**1978-2012**	**1996-1999 2004; 2007**	**1979-1993**
8	Jasenovac	500.5	38953	1978-2012	1991-2007	-
9	Stara Gradiška	453.5	40262	1963-1991	1991-2000	-
10	**Slavonski Brod**	**360,0.**	**50858**	**1960-2012**	**1994-2003**	**1979-1993**

3 METHODOLOGY

3.1 Continuous wavelet transform (CWT)

There are two classes of wavelet transforms: the Continuose Wavelet Transform (CWT) – for better feature extraction purposes; and Dicrete Wavelet Transform (DWT) – useful for noise reduction and data compression [1].
Generally, for a time domain signal *x(t)*, wavelet transform is defined as follows:

$$X(\tau, a) = \frac{1}{\sqrt{|a|}} \int_{-\infty}^{+\infty} x(t) \cdot \psi^* \left(\frac{(\tau - t)}{a} \right) dt \tag{1}$$

where $\frac{1}{\sqrt{|a|}} \psi \left(\frac{t-\tau}{a} \right)$ is a wavelet function, ψ^* is a complex conjugate of the wavelet function, τ is a time shift and a is a scale parameter [16]. There are plenty different wavelet functions, but Morlet function is most widely used for analysis of hydrological time series.

The continuous wavelet transform (CWT) with Morlet wavelet function is applied in this study for analysing localised variations of power in time-scale space of hydrological series. Wavelet software that is used in this study is available at: http://paos.colorado.edu/research/wavelets/. Torrence and Compo (1998) provided corresponding detailed practical guide for its usage, where more details on CWT can be referred [1], [2]. In this study significance levels for wavelet power spectra (WPS) are found by comparison against red-noise background modeled as a univariate lag-1 autoregressive (AR-1) process. WPS in time-scale domain are color-mapped to indicate high wavelet power with red and oranges, and low power with blue and white and are analysed only inside cone of influence COI that is presented as U-shape line. Outside COI edge effects might distort the picture. The wavelet variance is indicated by Global Wavelet Spectrum (GWS), calculated by integration of the squared transform coefficients at different scales for all input data. Global Wavelet Spectrum (GWS) with 95% confidence level against red noise will be analyzed. Periodicities above it are significant [7].

4 DISSCUSSION

4.1 Sediment and discharge periodical changes on GS Podsused

The wavelet power spectra (WPS) for mean monthly discharge, sediment load and sediment concentration on GS Podsused from year 1979 to 2010 are shown in **Fig. 2a-c**. WPS for discharge is noisier than WPS for sediment load and concentration. Regions of high power spectra relative to noise background are noticed in discharge signal for semi-annual, annual and 2-year period for years: 1982-1985, 1990-1995 and in years 1998 and 2001. Annual period appears in the same years as semi-annual and 2-year period is stable from 1995-2005.

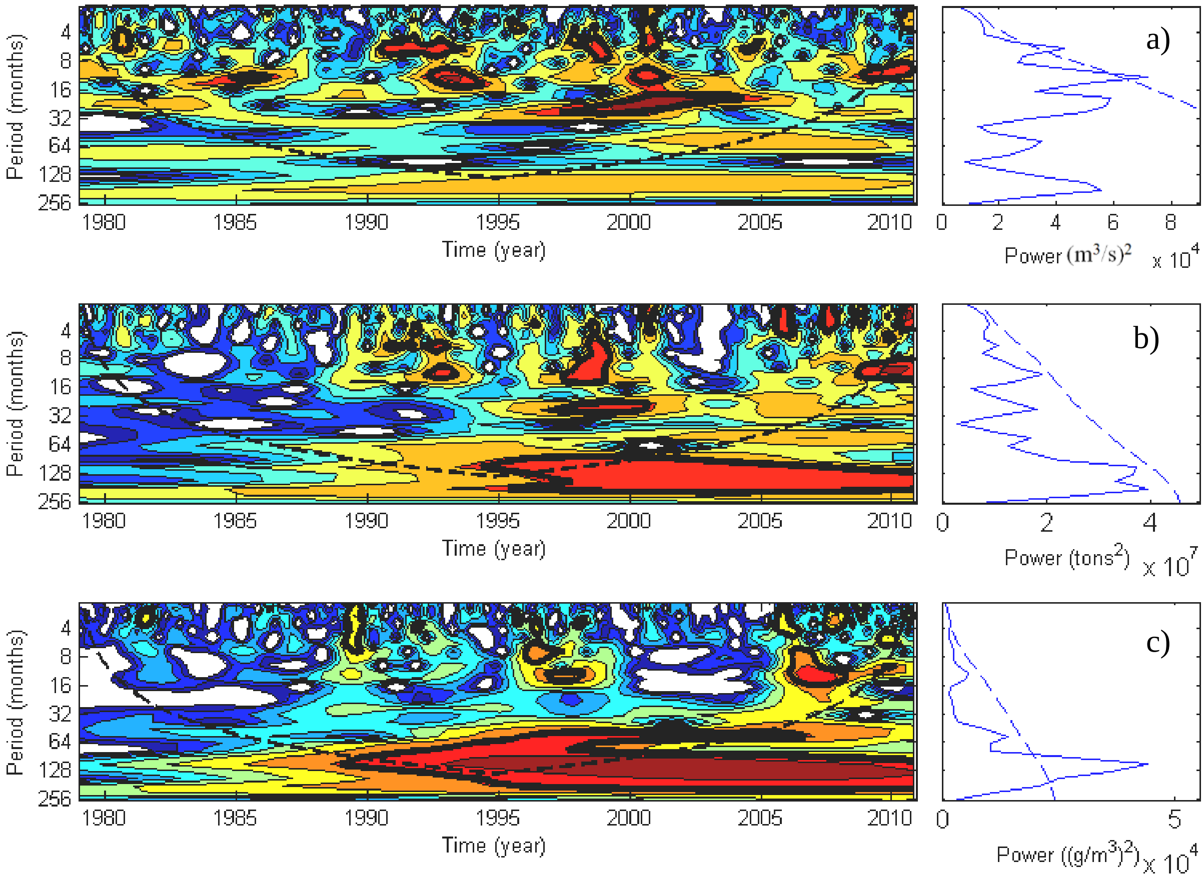

Fig. 2 On the left from top down, continuous wavelet power spectrum is shown for mean monthly time series: (a) discharge (m^3/s), (b) sediment load (tons) and sediment concentration (g/m^3) at GS Podsused. Significant wavelet spectra are shown with thick black contour within the cone of influence (COI) presented as U-shape line (dashed thick line). On the right at each figure is Global Wavelet Spectrum (GWS) with 95% confidence level against red noise (dashed blue line).

The strongest signal in WPS of sediment load and concentration is from 1990 to 2005. Beside 0.5, 1 and 2 year period there is strong common 7-year period (90 months) in both sediment load and concentration spectra. This could be an indicator of climate factors that influence the production of suspended sediment (flood and drought alternations). This is best seen in sediment concentration where 9-year period (100 months) is significant.

WPS for maximum monthly discharge, sediment load and sediment concentration on GS Podsused from year 1979 to 2010 are shown in **Fig. 3a-c**. In comparison with GPS of mean monthly data has more distinct difference between high and low power: that is way it is easier to notice common 0.5, 1 and 2-year period in years: 1990-1995.

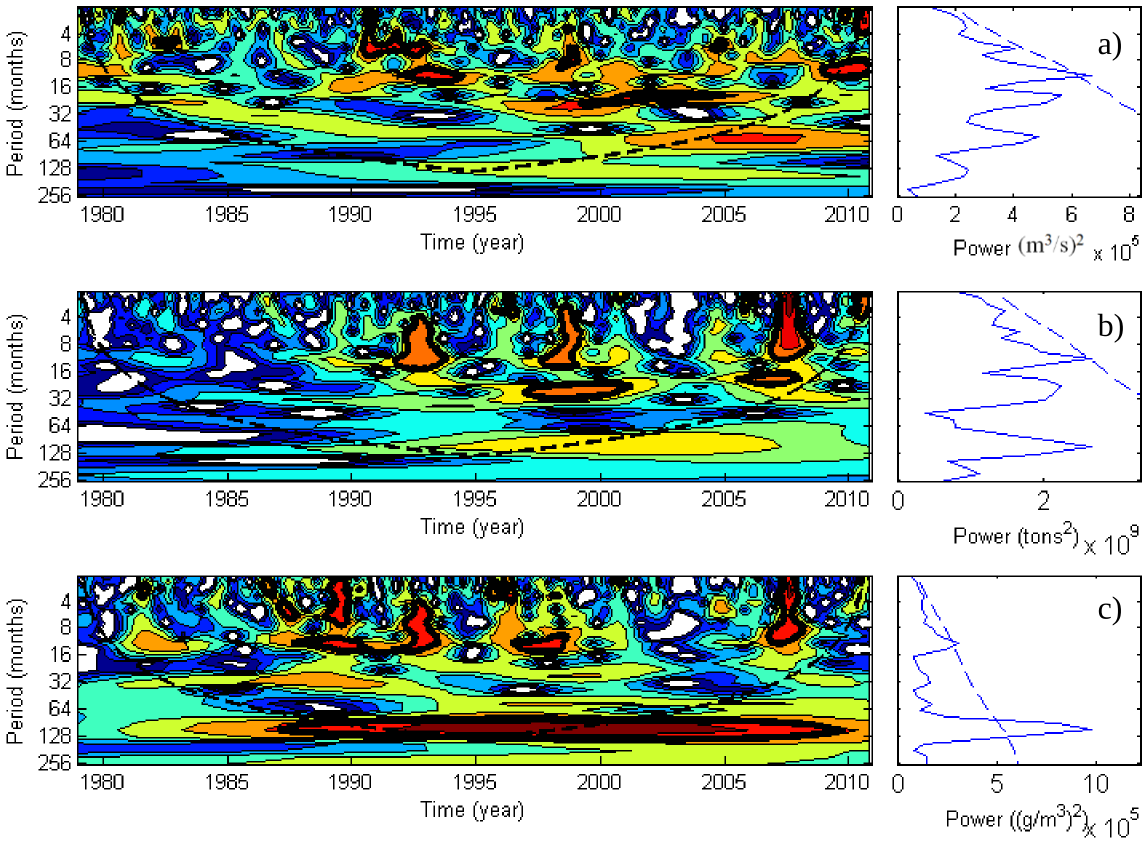

Fig. 3 On the left from top down, continuous wavelet power spectrum is shown for maximum monthly time series: (a) discharge (m^3/s), (b) sediment load (tons) and sediment concentration (g/m^3) at GS Podsused. Significant wavelet spectra are shown with thick black contour within the cone of influence (COI) presented as U-shape line (dashed thick line). On the right at each figure is Global Wavelet Spectrum (GWS) with 95% confidence level against red noise (dashed blue line).

4.2 Comparison of maximum sediment and discharge changes on the three GSs

WPS for maximum monthly discharge and sediment load on GS Podsused, GS Rugvica and GS Slavonski Brod is shown for common continuous time span of measured data – from the year 1979 to 1993 (**Fig4a-f**).

When comparing periodical pattern on GS Podsused and GS Rugvica, it can be seen common 3, 6 and 12 month period, both for maximum sediment load and discharge. This is especially prevalent after the year 1990. Removal of 3, 6 and 12-month periods is present in discharge and sediment data from 1983 to 1990 but there is a significant 3-month period for sediment load signal on GS PO in the year 1986. There is stronger signal on GS PO for sediment load then for discharge after the year 1990, where for GS RU the situation is reverse. This could be the result of the influence by the Kupa river backwater downstream from GS RU.

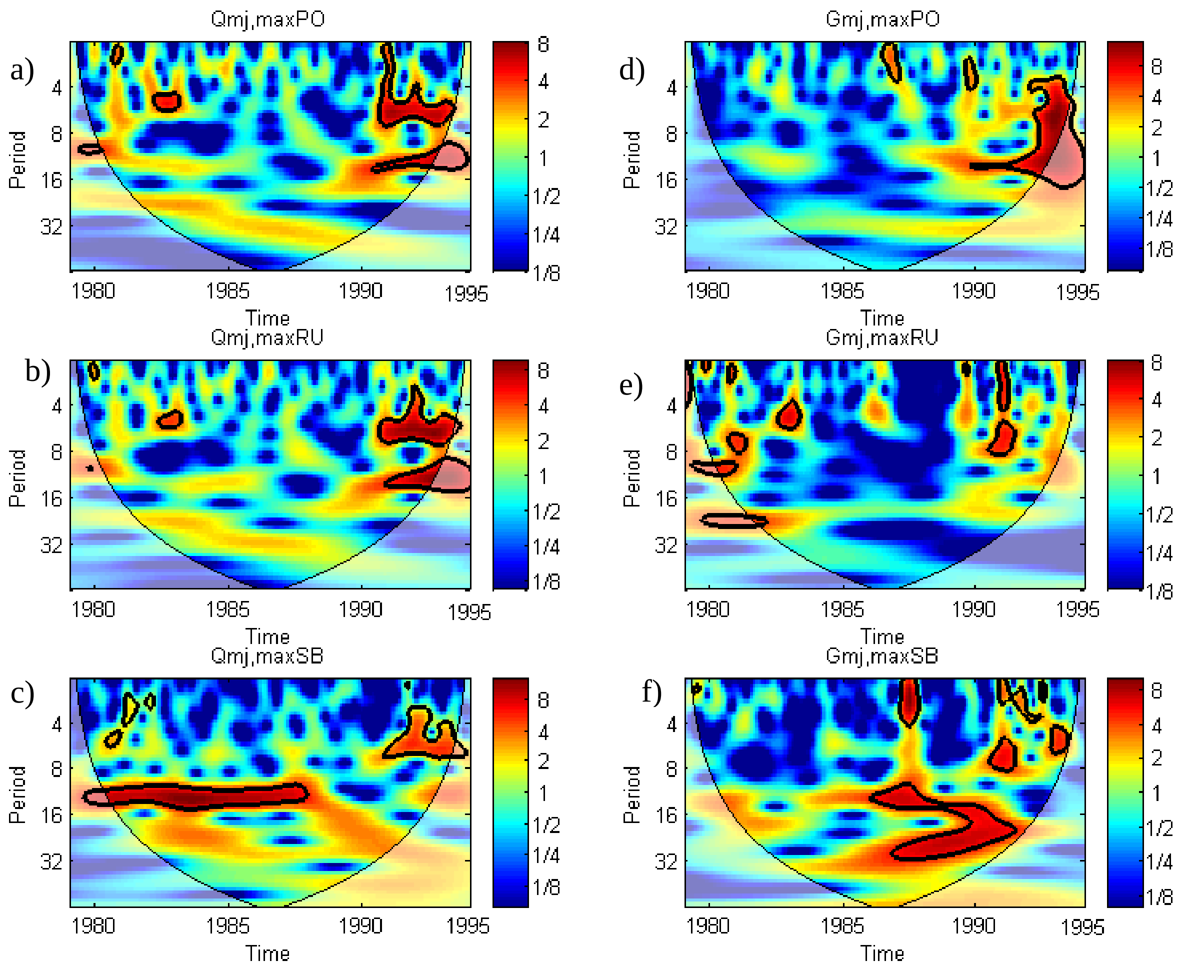

Fig. 4 On the left from top down, continuous wavelet power spectrum is shown for normalized time series of maximum monthly discharge at GS Podsused (a), GS Rugvica (b) and GS Slavonski Brod (c). On the right from top down, continuous wavelet power spectra for normalized time series of monthly maximum sediment load are shown at GS Podsused (d), GS Rugvica (e) and GS Slavonski Brod (f). 95% confidence level against red noise is marked with thick black contours and the cone of influence (COI) where edge effects might distort the picture is shown in thin U-shape line.

When comparing series on GS PO and GS RU with downstream time series on GS SB, there is a stabile annual period on GS SB in maximum discharge from the year 1980 to 1987. Also, common 3-6 month periods are seen on GS SB, GS PO and RU for the years 1990-1993. The appearance and stability of annual period agrees with the fact that strong decrease in the Sava riverbed downstream of GS RU causes less variability in discharge series in downstream parts of the Sava basin. In maximum discharge there is removal of annual period from 1988 to 1991 and that is consistent with 2 upstream GSs. This is probably caused by drought years on Sava from 1988 to 1990 [10]. It is interesting that for that drought years there is appearance of strong 0.5, 1 and 2-year period in maximum sediment load signal. This could be caused by stronger production of sediment load due to drought in the upstream parts and deposition in downstream parts of the basin. Noticed periodical component in discharge series on all the three GSs is consistent with previous CWT analysis of water-level series on middle reaches of the River Sava [17], [18].

5 CONCLUSION

In this paper wavelet analysis is used to identify periodical patterns in mean and maximum monthly discharge and suspended sediment load data on 3GS on the River Sava: Podsused, Rugvica and Slavonski Brod. When analyzing variability of maximum sediment load on all three GS, most variability is seen in periodical patterns on GS RU. Also there is noticed strong 0.5 and 1-2 year period for sediment load on GS SB in drought years which could be connected with climate influences on sediment production. With collection of longer time series data of sediment load better conclusions about connection between suspended sediment load and drought will be possible.

Although it is difficult to link changes in sediment loads and runoff series with climate and anthropogenic causes, future research with examination of wavelet cross correlation and wavelet coherence between hydrological and climate series could contribute to better understanding of these processes. The analysis of periodical patterns in discharge and sediment load series conducted in this paper should enable better management decisions in the Sava basin and possible improvement of black-box predictive models with periodical components. Finally, understanding of periodical changes is also connected with better understanding of changes in wetland flora and fauna habitat in the Sava basin.

Acknowledgements

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ECOHYDROLOGICAL SOLUTIONS TO RIVER CORRIDOR AND WETLAND RESTORATION – THE CONCEPT FOR THE GOLEMA REKA RIVER IN THE PRESPA LAKE BASIN

D. Sekovski[1], C. Popovska[2]

Abstract

The development pressures in the Prespa Lake Basin have caused considerable loss of valuable landscape elements (e.g., wetlands, riparian corridors) that used to provide critical ecological services for the watershed's integrity.

The combined influence of natural and human factors had a particularly harsh impact on the Ezerani wetland area – a natural high value ecosystem located along the northern shoreline of Lake Prespa.

The canalization of the river delta of Golema Reka – Lake Prespa's biggest tributary in the late 1990s is one of the most significant human interventions. Disconnecting the river channel from its natural floodplain dramatically reduced the wetland area and its filtering capacity. As a result, considerable quantities of pollutants and nutrients are being continuously discharged into the Lake.

The ongoing 'Restoration of the Prespa Lake Ecosystem' project intends to support significant wetland restoration/creation activities.

This paper elaborates on the concept for restoring the river corridor of Golema Reka and the related wetlands in the delta area by applying the ecohydrological approach.

Keywords: *Prespa Lake, Ecohydrology, River Corridor, Wetland Restoration*

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1 INTRODUCTION

The ‘Ezerani’ protected area is a natural high value wetland ecosystem located along the northern shoreline of Lake Prespa (Figure 1). The Lake Prespa (including Ezerani wetland) is both on the list of the most important ornithological locations of Europe and the list of the most important wetlands in the world – it was designated a Ramsar site in 1995.

With an age of over 5 million years, the Lake is placed on the list of the seventeen world’s ancient lakes [7]. Its basin is home to more than 2,000 species of animals, plants and birds, which puts it on the map of areas of global ecological significance.

Located at the river delta of Golema Reka and Istocka Reka rivers – the most degraded tributaries of Lake Prespa – the Ezerani wetland has been securing an important filtering function, essential for the Lake’s safeguarding against the upstream pressures. Human induced changes in Golema Reka and Istocka Reka have exacerbated the changes in the Ezerani wetland.

Fig. 1 Location of the Prespa Lake and Ezerani Wetland Area

Over the past years, the entire lake ecosystem has been facing with serious environmental challenges.

The Lake’s specific hydrology makes it a very vulnerable system. Because of a constant loss of water to the neighboring Lake Ohrid, through the karstic massif of Galicica mountain, the variation of the water level is very sensitive to precipitation and water use [10]. The excessive loss of freshwater over the past decades has not only severly affected the valuable shorline habitats but has also intensified major degradation processes (Figure 2).

Besides the anthropogenic discharges of nutrients and pollutants, the development pressures (primarily from agriculture) have caused considerable loss of valuable landscape elements that provided critical ecological services that contributed to the watershed’s ecological integrity (e.g. wetlands, riparian corridors).

The canalization of the river delta in 1998 was perhaps the single most significant human intervention [17]. Disconnecting the river channel from its natural floodplain dramatically reduced the wetland area and filtering capacity. As a result considerable quantities of pollutants and nutrients are being continuously discharged into the Lake.

The recent comprehensive investigations into the Lake’s ecological status revealed that one of the most significant problems that the ecosystem is currently facing is eutrophication, caused by the nutrient and organic inputs originating from agricultural runoff, watershed’s erosion processes, wastewaters and solid waste.

The pollution and eutrophication processes have not only affected the region's valuable biodiversity, but also key sectors such as tourism, water and fisheries, all of which have been imperative in ensuring the local population's well-being [19].

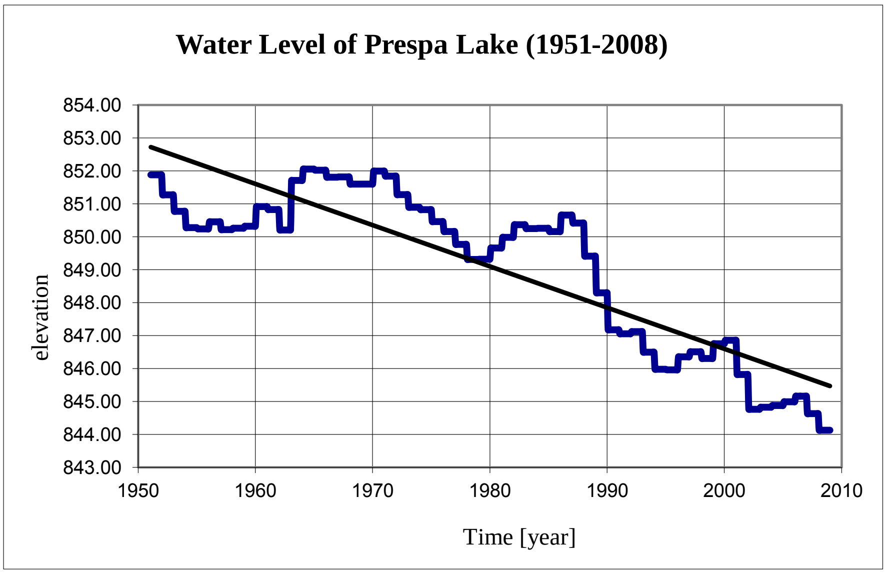

Fig. 2 Water level decline of Lake Prespa for the period 1950 – 2008
(*source*: Hydrometeorological Administration)

Figure 3 summarizes the recent findings of the analysis of nutrients from the five lake sampling points [5], [2]. Despite the absence of regular monitoring data, it can still be stated with a high degree of certainty that the Golema Reka River is the most important 'source' of nutrients for the Lake. The restoration of the Ezerani wetland's natural functions is a logical measure to combat the Lake's eutrophication processes through nutrient retention and filtering.

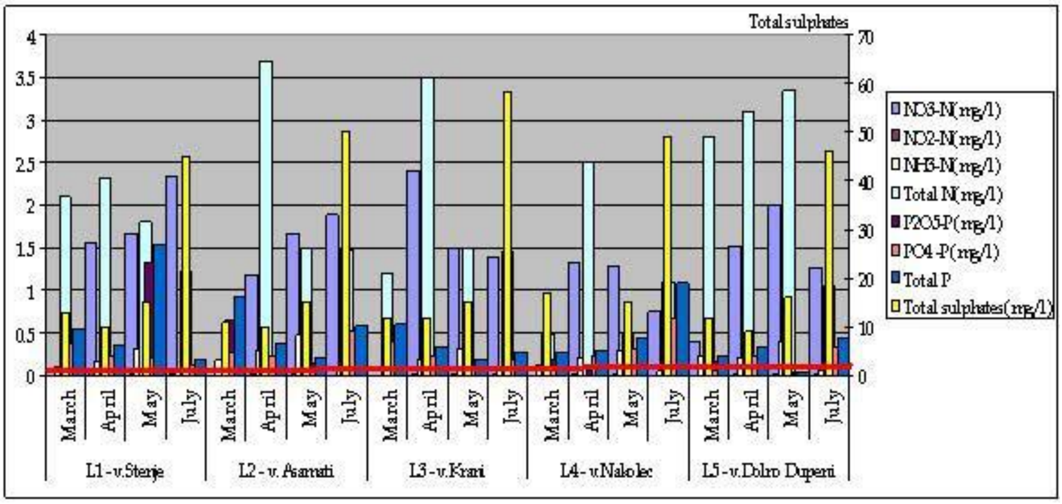

Fig. 3 Nutrient levels detected in Prespa in 2011 – an indicator of hyper-eutrophic state (the horizontal red line presents the threshold value of total phosphorus concentrations for hyper-eutrophic waters)

The purpose of this paper is to elaborate on the concept for restoring the river corridor of Golema Reka and the related wetlands in the delta area by applying the ecohydrological approach. It discusses possibilities, limitations and challenges for implementing restoration plans given the current local setting.

The restoration measures are being supported by the ongoing 'Restoration of the Prespa Lake Ecosystem' project implemented by the United Nations Development Programme (UNDP), the Municipality of Resen and the Ministry of Environment and Physical Planning, with the financial support of the Swiss Agency for Development and Cooperation.

2. METHODOLOGY

The methodology applied in developing the river corridor and the wetland restoration concept for the Ezerani river delta area is founded upon the principles of ecohydrology – a transdisciplinary science, inspired by the growing demands for alternative science-based, cost-efficient responses to sustainability and water issues [22].

The approach is based on the principle of using the ecosystem properties as management tools. The principle features three steps of implementation: a) 'dual regulation' – biota by hydrology and vice versa, hydrology by shaping biota or controlling interactions; b) integration at the basin scale of various types of biological and hydrological regulations towards achieving synergy to improve water quality, biodiversity and freshwater resources, and c) harmonization of the ecohydrological measures with the necessary / existing hydraulic structures (e.g. dams, irrigation systems, wastewater treatment systems) [23], [24].

The application of the ecohydrological controls should be based on a solid understanding of the hydrological features and ecological processes in the basin. The basin-scale ecohydrological programs normally aim at enhancing the landscape diversity. That is important not only for regulating the hydrological and ecological processes, but also for increasing the aesthetic and recreational values.

The agriculture dominated landscape of the Prespa basin need to be enriched with different non-productive elements which have proven effective in the control of diffuse pollution. These include: hedges, shelterbelts, stretches of meadows, riparian vegetation strips and small ponds [24]. A basin-scale approach needs to be applied prior to making any decision on the selection of the most appropriate wetland restoration scenario for the delta of Golema Reka.

The ecohydrological solutions cannot be considered as substitutes to the measures for elimination of threats such as the control of point and diffuse pollution (e.g. through wastewater treatment plants, or control of agrochemicals use). However, given the current state of the ecosystem, and its inherent vulnerability, the purely technical solutions aren't considered sufficient if the restoration of the ecosystem integrity is to be achieved.

Turning the abandoned fish ponds (Figure 6) in the vicinity of the former delta of Golema Reka into multi-functional wetlands complemented with the restoration of the former delta area present a typical ecohydrological solution. Besides the compensation for the formerly lost wetland area, 'activating' the fish ponds would provide an excellent opportunity for upgrading the existing municipal wastewater treatment plant (WWTP) for nutrient (nitrogen and phosphorus) removal.

This wetland restoration initiative must be viewed in the light of the efforts to operationalize the 'Ezerani' Nature Park. The methodology applied in this paper capitalizes on the achievement of these efforts that also involve a complete re-designation process.

The paper very briefly elaborates on the restoration concept without intending to provide a comprehensive ecohydrological system solution. For that purpose an in-depth analysis of the cause-effect relationships and comparative assessment is required [22].

3. ANALYSIS

One of the most significant parts of Lake Prespa is the Ezerani wetland. This part of the Lake is characterized by shallow water depth, frequent sandbars parallel to the beach and a marshy shore with a broad reed belt [21].

The ecological values and biodiversity richness of Ezerani give this area a special scientific and research significance. Because of its high ornithological significance, Ezerani has been placed on the list of the most important ornithological sites in Europe (IBA-Important Bird Area) and, from 1995, also on the Ramsar Convention List of the most important wetlands in the world (together with the whole of the Macedonian part of Lake Prespa). Recognizing the conservation significance of the area it was also designated a Strict Nature Reserve (IUCN I) in accordance to the national Law on Nature in 1996.

However, these legal instruments did not prevent the human-induced degradation processes and loss of wetland area. On the contrary – much of the degradation of the natural ecosystems happened after the official designation [21], [17]. The legal protection failed to prevent the uncontrolled expansion of agricultural land (mainly for apple production), at the expense of the natural areas (wetlands and riparian corridors). The prolonged dry period and the low river flows additionally amplified the expansion.

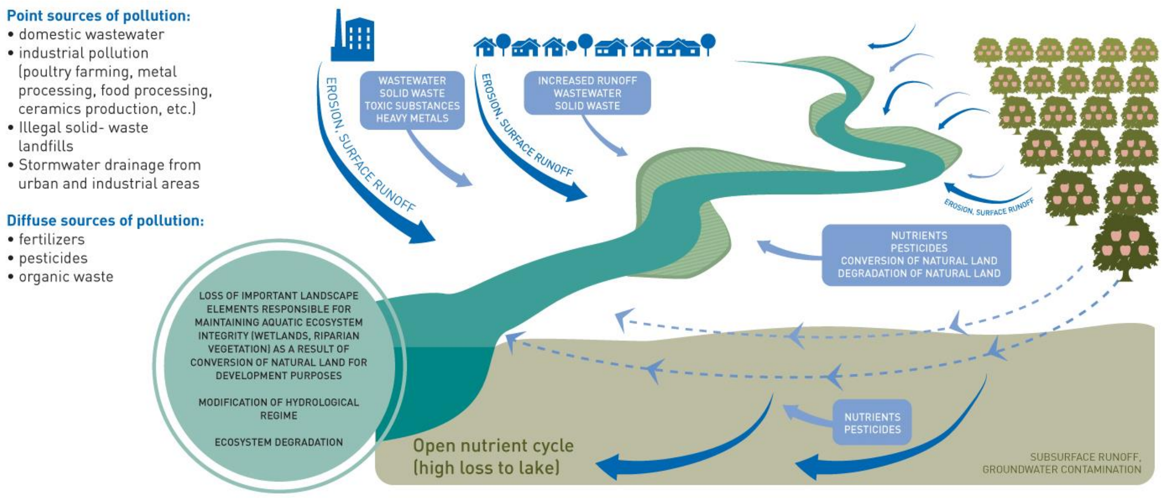

Fig. 4 Illustration of the sources and transport routes of pollution of Lake Prespa: the canalization of the river delta area of Golema Reka results in a high loss of nutrients and other substances from land to water (open nutrient cycle)

The unsustainable agricultural practices, illegal sand/gravel removal, illegal logging and fishing, human induced fires and pollution, are amongst the main types of pressures to the wetland. The canalization of the river delta in the late 1990s is considered to be the single most significant human intervention.

Disconnecting the river channel from its natural floodplain dramatically reduced the wetland area and its filtering capacity. As a result, considerable quantities of pollutants and nutrients are being continuously discharged into the Lake (Figure 4). The occasional flood events are the reason for individual or locally organized interventions in the river channel – its

straightening and deepening being the most common flood control practice. This disturbance allows the river forces to further incise the alluvial river channel causing loss of connectivity with its natural floodplain (Figure 5).
The previous and ongoing efforts to operationalize the 'Ezerani' protected area are particularly instrumental in creating the enabling environment for the planned restoration activities. These efforts resulted in a complete re-designation of the protected area that involves a few critical modifications of the management system.

Fig. 5 Examples of incised river channel sections within the boundaries of Ezerani wetland

Besides redesigning the area's boundaries (Figure 6), this process involved reclassification of the protected area from IUCN Category I (Strictly Nature Reserve) to IUCN Category IV (Habitat/Species Management Area or Nature Park according to the national legislation). The previous analyses showed that the operation of 'Ezerani' protected area does not fit the primary management objective of a Strict Nature Reserve at any level [21]. The new category turned out to be a more feasible option given the current local setting, also because it would legally allow carrying out the intended wetland restoration measures [17].

Fig. 6 Location of the wetland system (incised riverbed, fish ponds, WWTP) and boundaries of the 'Ezerani' protected area (old with blue and new with red color)

Overcoming the conflict with local interests was recognized as the key factor for better management. For this purpose a highly participatory approach was applied throughout all stages of the re-designation process, including the preparation of the new management plan and an ecosystem valuation study. The previous work provides the legal, scientific and technical basis for further elaborating and finally implementing the restoration plans.

4. RESTORATION CONCEPT

The conceptual solution proposed in this paper is based upon the key principles of ecohydrology. The restoration efforts have the potential to increase the absorbing capacity of the area against the pressures resulting from the landscape degradation and the associated diffuse pollution. If implemented and maintained properly the restored wetland area would help in regulating the hydrological regime (stabilizing the river discharge), providing nutrient and contaminant retention prior to their contact with the receiving lake waters.
The restoration efforts cannot be separated from the other basin-scale processes. This concept promotes the application of the integrated river basin management approach that will take into account the hydrological, geomorphological and ecological processes at the entire basins of rivers Golema and Istocka. This integrated conceptual ecohydrological solution is presented in Figure 7.

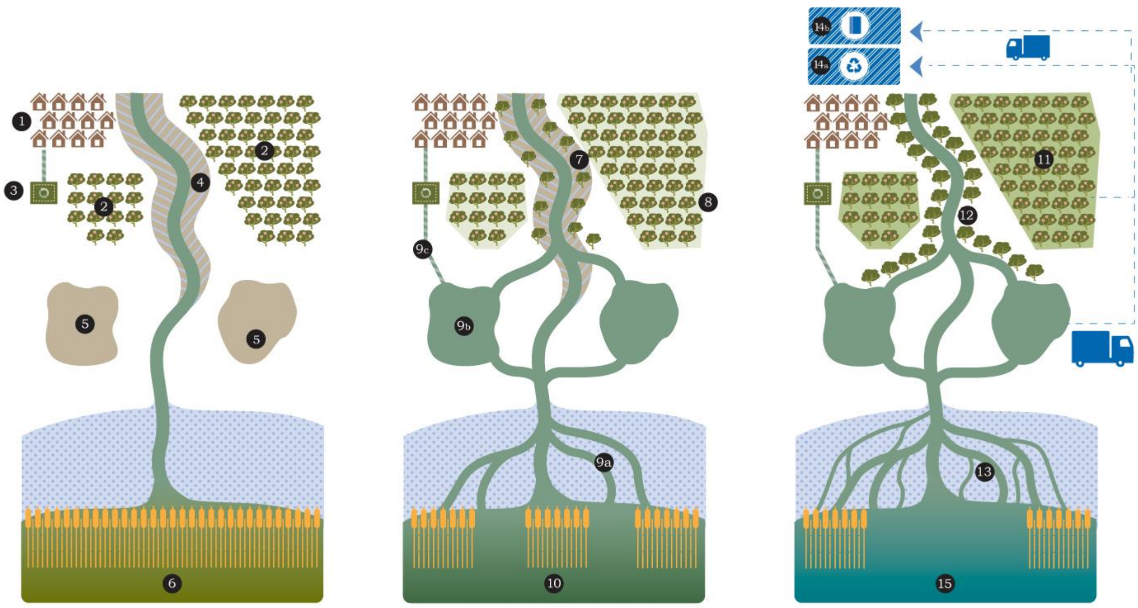

Fig. 7 The integrated basin-scale river corridor and wetland restoration concept

Legend: 1) Settlements; 2) Farmland; 3) WWTP; 4) Eroded land; 5) Abandoned fishponds; 6) High lake eutrophication levels; 7) Erosion control works – reforestation/afforestation; 8) Agroecological farming practices; 9) Restoration works: a. river delta restoration; b. turning fish ponds into viable wetlands; c. Tertiary treatment of wastewaters; 10) Improvement of the Lake's ecological status; 11) Agroecological and organic farming; 12) Regeneration of the riparian vegetation; 13) Functional wetland providing effective filtering of the upstream pressures; 14) Better management of organic and other types of waste; 15) Restoration of the Lake's ecological functions

The illustration shows a plan for gradual restoration of the deeply incised riverbed in order to re-connect it with its former floodplain. In addition, by applying adequate hydraulic structures, the fish ponds can be turned into wetlands, which together with the restored delta area would provide: a) tertiary treatment for the wastewaters from the adjacent municipal

WWTP; b) flood control by 'absorbing' water from Golema Reka during flood events; c) water retention of Golema Reka (filtering effect); d) retention / stabilization of untreated wastewaters in event of high hydraulic load or WWTP malfunction (when WWTP is bypassed); e) additional wetland-related benefits (wildlife habitat, recreational and aesthetic values, biomass generation).

The fish ponds would be replenished by the water of Golema Reka river especially during flood events.

In parallel to these efforts, various basin-scale measures are proposed that would result in minimizing the point and non-point pollution (e.g., better farming practices, erosion control works, restoration of the riparian corridor and better waste management). Since the use of agrochemicals and other harmful substances cannot be abandoned completely, this ecohydrological solution requires the use of different biogeochemical barriers that would prevent their discharge to the related water bodies (e.g., riparian vegetation, and various vegetation 'structures' that would increase the landscape diversity of the farmland, increasing the control of the diffuse pollution).

The effective functioning of the newly created wetland area would require active management of the excess biomass that would develop as a result of the continuous supply of nutrients to the system. The biomass can be added an economic value through composting or other treatment processes.

A detailed feasibility assessment process should build upon this concept to comparatively analyze various wetland restoration alternatives. Various combinations of wetland restoration measures would need to be analyzed following the preliminary evaluation of possible options (e.g., restoring only the delta, restoring only the fish ponds, a combined restoration of the delta and one or two fish ponds, and other possible options).

The proposed solutions must take into considerations the limitations imposed by the land-ownership structure (private land is marked with dark green in Figure 6) and balance the restoration goals with the social and political contraints. The solutions with no or very little expropriation requirements would have much better chance for implementation..

The study of the historical conditions/evolution of the wetland area (including river modifications of Golema and Istocka Reka) as a result of natural and human influences is of critical importance when defining the restoraiton goals. The review of historical documentation, field investigations, communication with the local communities and authorities andn old maps can be very useful in this regard.

A designated water quality monitoring programme may be necessary to study the nutrient and other loads, and to establish the nutrient balance. A geological/soil analysis would reveal the suitability of the local soil conditions for the retention of nutrients and other important substances. Groundwater analysis including the inflows and outflows to the wetlands are also necessary when defining the restoration targets.

In addition, a comprehensive vegetation assessment needs to be carried out with regards to the native species that can play role in the desired wetland functions. The hydrological and hydraulic modeling, including sediment and nutrient transport analysis of the current and restored situation is of critical importance.

The restoration alternatives would then be subject to detailed feasibility/cost-benefit assessment against a range of technical, environmental, social/stakeholder, economic and financial factors. These analyses would need to be complemented with hydraulic/flood risk analyses (focusing on the possible damage of private property (land and built structures) as a result of the wetland restoration actions.

The financial and economic viability of the proposed alternatives also needs to be considered. For this purpose the analysis would determine: initial capital costs (e.g., land acquisition, materials and works and other), operation and maintenance costs on annual basis (e.g. labor, transportation, possible harvesting costs) and other considerations.
The most feasible option will need to be elaborated to the necessary level of detail, so that is creates a basis for the physical implementation of the restoration works.

5. CONCLUSIONS

Overall, if implemented properly, the restored and newly created wetlands are expected to provide (Figure 8): a) water retention of Golema Reka, allowing the wetlands to slow and filter inflows from the watershed; b) tertiary treatment for the wastewaters from the municipal WWTP; c) flood control function; d) retention / stabilization of untreated wastewaters in event of high hydraulic load or WWTP malfunction; and, e) additional wetland-related benefits (wildlife habitat, recreational and aesthetic values, biomass generation).

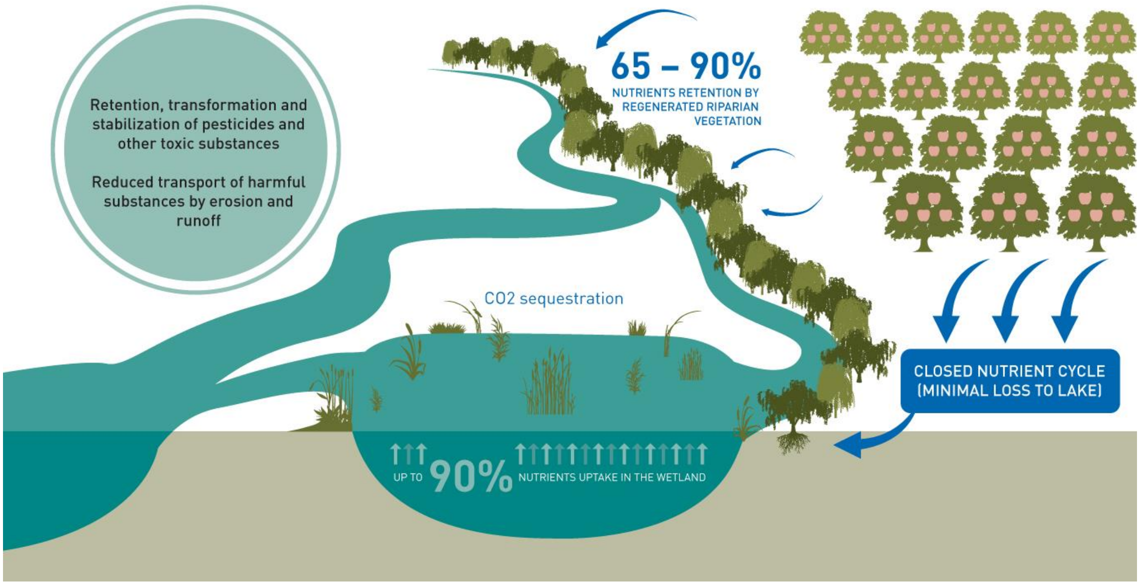

Fig. 8 Benefits of the restored wetland and riparian system – retention, transformation and stabilization of nutrients and other harmful substances resulting in minimal loss from land to the water bodies (closed nutrient cycle)

The presence of unresolved land ownership issues inside the fish ponds is expected to pose the highest risk to achieving these goals. However, a recent UNDP supported economic valuation study shows that the benefits of wetland restoration would bring over the next 20 years are 6.5 to 9 times higher than the costs when different discount rates are used, including purhasing the private land. The cost-benefit analysis shows that the restoration works could be a very cost-effective way to treat wastewaters [1].
In conclusion, this ecohydrology-based restoration approach has the potential of securing important protection of Lake Prespa, while providing at the same time numerous ecological and social benefits. However, besides the political will, the actual implementation of the restoration plan would also require continuous awareness raising and education of all stakeholders (primarily landowners and affected communities) on the multitude of tangible and intangible benefits of the restoration work.

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GIS BASED MONITORING DATABASE FOR DUBRAČINA RIVER CATCHMENT AREA AS A TOOL FOR MITIGATION AND PREVENTION OF FLASH FLOOD AND EROSION

Ivana Sušanj [1], Nevena Dragičević [2], Barbara Karleuša[3], Nevenka Ožanić[4]

Abstract

In this paper a GIS based monitoring database as a tool for mitigation and prevention of flash flood and erosion impact will be presented. The database is created for Dubračina River catchment area in Vinodol Valley (Croatia) that was chosen for its historical significance of potential hazard development. Information about built hydraulic structures (river network, river regulation..), geological (soil type, erosion and landslide affected areas), land use (types of vegetation coverage, areas used for agriculture..), anthropological (urban areas, traffic infrastructures, illegal waste disposals) and historical data (affected areas in the past, implemented structural and non-structural measures..) will be processed into an organized and correlated information database and map overview. For this purpose the software ArcGis 10.1. will be used.

Keywords

database, Dubračina river, erosion, flash flood, GIS, mitigation and prevention.

1 INTRODUCTION

Hazardous events, natural phenomena that occurs in a populated areas, with consequences such as losses of human life and/or significant material and infrastructure damages, are

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considered to be natural disasters. Same events in uninhabited areas and areas of no interest for people, are not considered to be disasters, and are rarely of any interest for detailed research and implementation of hazard mitigation strategy [1]. In populated areas, it is very hard to separate events as only natural events, in a way to exclude the impact of human activities. The occurrence of hazard phenomena cannot be prevented by humans, but its consequences can be minimized or even intensified depending on the human activities in the area prone to hazards [1]. Debris avalanches, expansive soils, landslides, rock falls, drought, erosion and sedimentation, river flooding, flash flood, mud flows and many more, are all considered to be hazardous events.

The first step in hazard mitigation and prevention strategy should be the determination of the area current condition and its comparison with historical information. The aim and objective of such monitoring database is its continuous development in order to act upon the hazard before its occurrence. In this paper, the emphasis will be given upon problems related with flash flood and erosion hazards, due to its significant impact on Dubračina River catchment area, that is already know as hazard risk area.

Flash flood and erosion can be influenced and initiated by many natural and anthropogenic factors, upon some itis possible to act and mitigate more than others. The occurrence of hazard is usually caused by a combination of different factors that could combined together potentially became triggering factors. Monitoring of changes on researched areas in order to achieve hazard mitigation and prevention is extremely important. Collected data itself is of no or little value if not used and transformed into information that is organized, presented, analysed and interpreted all in one integrated system. Only such a system can be used for different purposes: hazard risk assessment, hazard prediction, urban planning and future development of certain area, etc. [2, 3].

The research conducted on the river Dubračina catchment area within the bilateral Croatian-Japanese project “Risk identification on Land-Use Planning for Disaster Mitigation of Landslides and Floods in Croatia” will be presented in this paper. The chosen research area is of great significance, withal cultural, natural, geological, hydrological, as well as an area potentially endangered with erosion, landslide and flash flood hazard. For the purpose of further analysis a GIS database was made, with the use of available data from various sources and field research. Also, the worldwide usage of similar GIS databases is presented.

2 IMPORTANCE OF GIS TECHNOLOGY IN HAZARD MONITORING, PREDICTION, PREVENTION AND MITIGATION

In order to accomplish a comprehensive review of the area, what is the subject of this analysis, it is of crucial importance to gather and evaluate all available data and sources. Such data collection can then be called “Database”. Analysis and hazard risk assessments should be based upon such overall collected and organized data in order to achieve its accuracy.

The purposes of such database are future possibilities for hazard risk evaluation. For that to be able to accomplished, it is necessary to survey and analyse triggering factors that could lead to hazard event, as well as the elements at risk that could potentially be affected by the occurrence of such phenomena. There is no existing detailed regulation that consolidates these elements in one final list. To do so, it is almost impossible since these factors and elements can vary from one research area to another [2].

This type of database can also be used in the processes of planning, where it is essential that planners are familiar with the wide range of information that would often be dissipate pieces of information, if not put together and arranged properly. The Collection of data is the hardest and very complicated process [1]. Sometimes there are data that have been forgotten, or can only be obtained from older generation of inhabitants. Others are dissipated in many different profit and non-profit agencies that have done some of the research or projects on analysed area. Another very important source of information is the Local Government, but maybe the most important ones are national, regional and local archives [4, 5].

2.1. GIS database in theory

When dealing with the problem that needs to be solved, and with numerous information that as individual are of little value, techniques for managing the information have shown to be very useful in preventing planners mistakes. For such purposes, Geographic information systems (GIS), is often used. GIS is a systematic tool, used for geographic referencing of large number of “layers”, that are perceived as information, in order to facilitate the overlaying, quantification, and synthesis of data with the purpose to simplify and focus each specific decision-making task and its processes (Fig. 1) [1]. A part of Geographic information system is geographical information, that is a georeferenced data, processed into a form that is meaningful to the user and as such is of great importance in the decision making process [3].

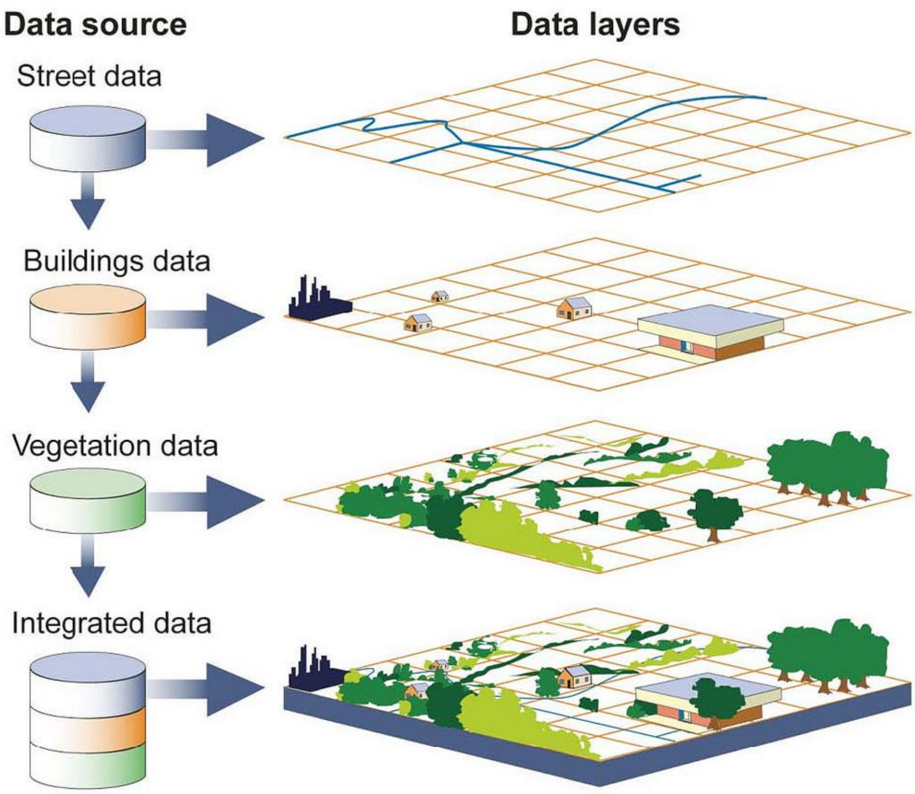


Fig. 1: Geographical information system - GIS „layers“ of information [6]

Last few decades, GIS technology has attracted the interest of many disciplines, among ones is civil engineering. Infrastructure management, transportation, land use planning, water resources management, environmental engineering, are just some areas of civil engineering where GIS was found to be very useful (Fig. 2), when implemented. Advantages, like lower costs, improvement of quality, supporting multi-discipline analysis for complex projects, have contributed to its further development, and today, its use is almost obligatory in the processes of solving various civil engineering problems [7].

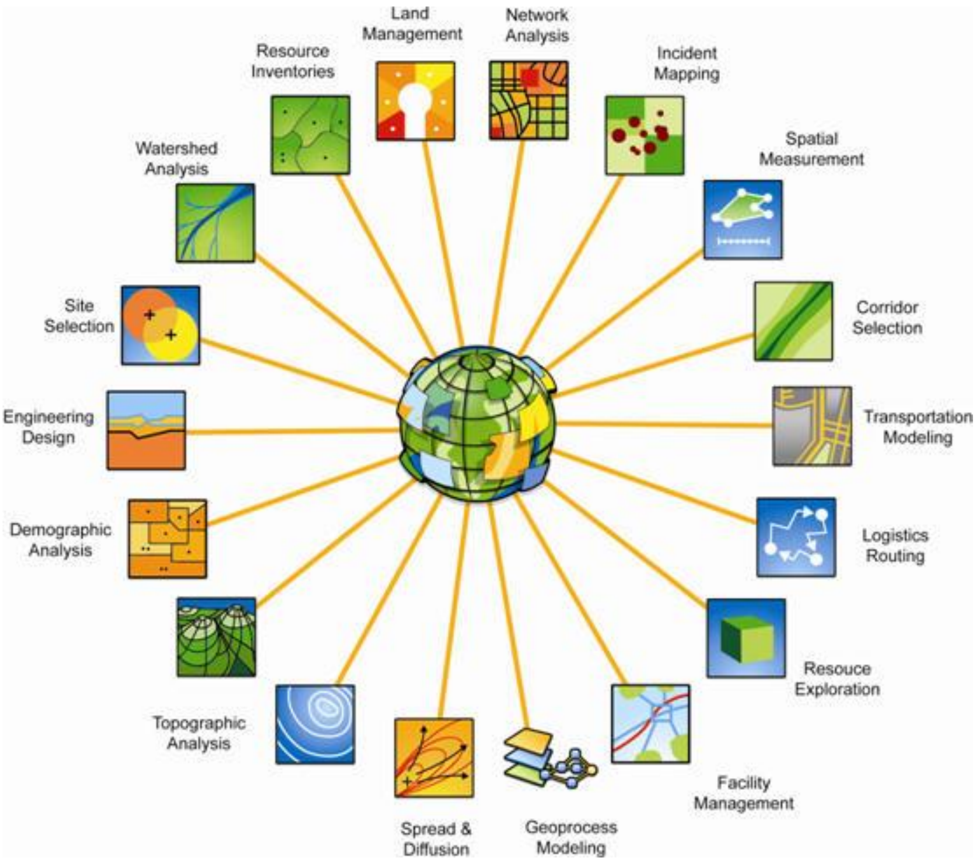
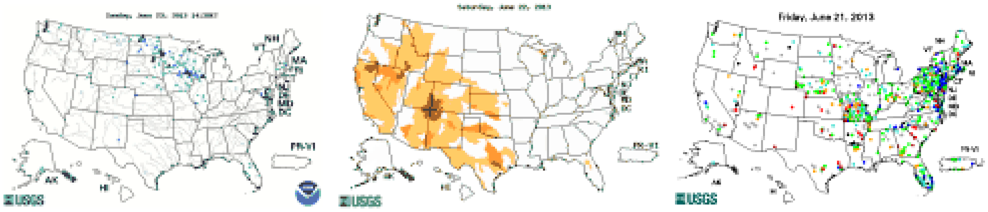


Fig. 2: Implementation of GIS [8]

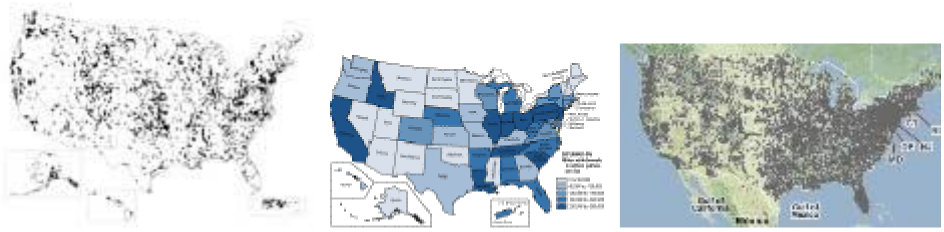
2.1. Worldwide implementation of GIS technology

There are various GIS computer software available that can be used for capturing, storing, checking, integrating, analysing and displaying spatially referenced data about the Earth. Nowadays, GIS is very much used for analyses of past hazard events and applications of disaster risk management measures [9].

An example of databases used for various analyses, projects and for one very important aim – information exchange and public information, is led by The U.S. Geological Survey (USGS) [10]. They collect and synthesize various different groups of information in several databases. For example, water spatial data, groundwater information, watershed maps and spatial data, surface water, water quality, water use, flood maps, etc. Such groups of data are later organized and presented into various forms: as real time water conditions (water level, streamflow, temperature), water alerts (high water levels), real-time streamflow (comparison with historical data), real time flood data, real time drought data, real time groundwater levels, recent groundwater levels, sediment data, National Water Quality Assessment, Water use in the United States, Annual water data reports, etc. (Fig. 3) [10]. Further use of this kind of data has no limits and it only depends upon the need of the society and the imagination of the researcher.



a) FLOOD DATA b) DROUGHT DATA c) GROUNDWATER LEVELS



d) SEDIMENT DATA e) WATER USE f) WATER ALERT

Fig. 3: Information as result of organized database by USGS [10]

Debris flow can be initiated by any factors, such as loose sediment, torrential rainfall and topographic conditions. Those data, among many others (topographic humidity index, sediment transport capacity index, elevation-relief ratio, form factor, effective basin area, slope gradient) were collected as input data for GIS database, used for identification of topographic features and conditions favourable for initiation of debris-flow. This analysis was made for the area of total 11 river catchment areas in northern and central Taiwan [11].

In Japan, the study whose aim is determination of the frequent distribution of knickzones in bedrock rivers (Fig. 4) throughout Japanese mountainous areas, was based on various topographic, climatic and geological data (topographic maps, water flow direction, drainage area, stream networks, slope angles, geological units, annual precipitation, elevation, etc.) and implemented in GIS surrounding [12].

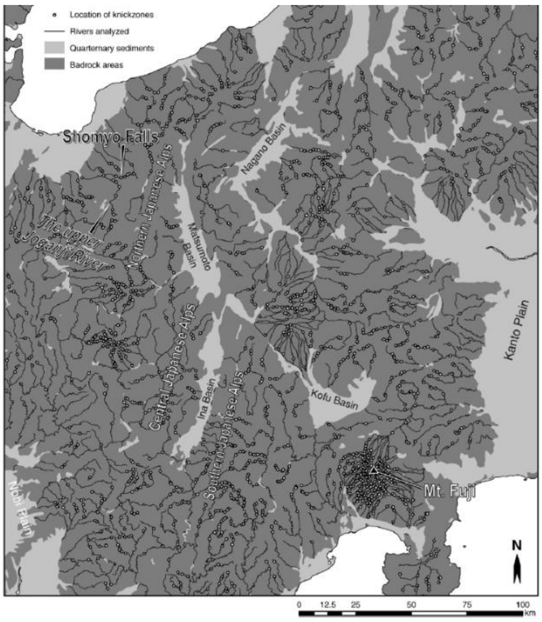


Fig. 4: Knickzone distribution in central Japan [12]

Another example of database was made for ACF Disaster Risk Reduction (DRR) and implemented in Philippines provinces: Camarines Sur and Catanduanes. Its final objective was to reduce population`s vulnerability to natural disasters. The main use of GIS in this project was based on multi-hazard mapping in two provinces that contains information like magnitude, frequency of hazard as well as affected areas [9].
Due to several severe storm flood events, that caused hundreds of people to evacuate from their homes in the Yerba Buena and Tucuman in Argentina, urban flood hazard assessment was made using GIS technology as a base for multi-criteria analysis [13].
In south-eastern Egypt, the development of mining activities and domestic infrastructures, caused the need for more precise knowledge about flash flood hazard in the area. GIS was used as a base database and latter for estimation about the amount of the surface runoff and the magnitude of flash floods [14].
There are numerous researches done around the world using GIS technology. Although, this technology exists for several decades, its full potential is jet to be discovered and implemented.

3 RESEARCH AREA: DUBRAČINA RIVER CHATCHMENT AREA

Vinodol Valley is a unique spatial entity situated between Križišće in the north-west, Novi Vinodolski in the south-east and the coast alongside the Vinodol channel (Fig. 5). It’s characterized by extremely asymmetrical cross section in the most part, with a north-eastern slope that is distinctly shorter and steeper than a south-western slope. Riverbeds of its two main watercourses Dubračina and Suha Ričina Novljanska are located at the lowest elevations of Vinodol Valley, but for this research Dubračina River catchment area is chosen.

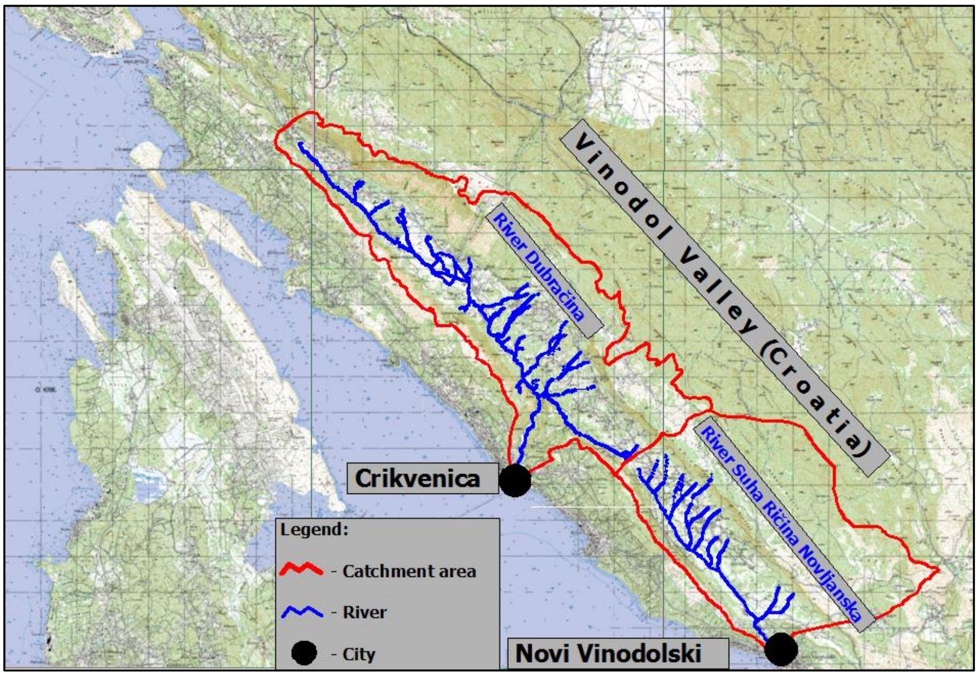


Fig. 5: Location of Dubračina and Suha Ričina Novljanska rivers catchment areas [15]

The composition of the soil in the area of the Vinodol Valley is very complex and consists of a many different lithological layers. Deposits from Cretaceous and Paleogene periods are litificated and they form carbonate sedimentary rocks of a clastic type (Fig. 6). Sediments that are partially or fully covering the older bedrocks are from younger Pliocene and Quaternary Period. Rocks from Cretaceous Period are from the Upper Cretaceous Epoch: Cenomanian-Turonian (K21, 2) and Turon-Senonian (K22, 3). The rest of the sedimentary rocks are from Paleogene Epoch: Paleocene to Lower Eocene (PCE1), Middle Eocene (E2), Middle-Upper Eocene (E2, 3) and the Upper Eocene-Lower Oligocene (E3O1) [16].

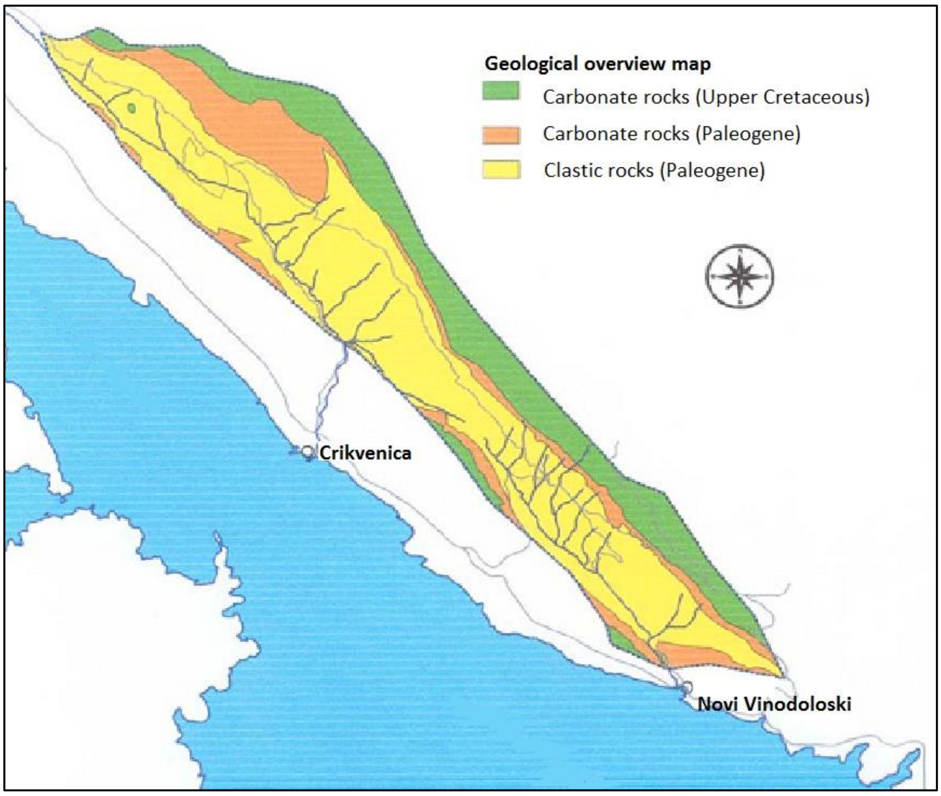


Fig. 6: Geological overview map of Vinodol Valley [17]

Because of its complex geological structure and special valley cross section with distinct steep slopes affected by erosion, local landslides and torrents this area has been known for many years as an area of potential hazard risk. Many elements in this area are considered to be of exceptional natural and cultural value, and as such, Vinodol Valley bears the title „Area of special significance for Primorsko – Goranska County“.

Dubračina River main watercourse in the central part of Vinodol Valley, it gathers waters from its many tributaries (Duboki, Bronac, Cigančica, Sušik, Slani potok, Mala Dubračina, etc.) of which most have torrential characteristics, and ends in the sea at the very centre of Town Crikvenica. Dubračina River catchment area is situated in the central part of Vinodol Valley. On the catchment area many springs and river tributaries form very well distributed river network on north-eastern slopes of Vinodol Valley. Because, the most affected areas with erosion, landslides and torrential flow, within the Vinodol Valley, are located on Dubračina River catchment area, this area has been chosen as a research area. Its sub-catchment areas of tributaries Slani Potok and neighbouring Mala Dubračina stand out as most affected ones (Fig. 7). Slani Potok catchment area is, as mentioned before, an example of the combined erosion, landslides and torrential impacts. Its surface is affected by excessive erosion that covers the area of 600 m in length and 250 m in width. The side effects, alongside the erosion affected areas are landslides, formed as a result of weathering of the flysh rock complex that is specific for this area [18, 19]. The overall size of affected area is approximately 3 km² causing potential hazard risk for its surrounding settlements Belgrad, Baretići, Grižane and Kamenjak as well as roads. Similar problems can also be found on the catchment area of river tributary Mala Dubračina. Unfortunately, roads and houses affected by local landslides and erosion processes can be found along the entire area of Vinodol Valley.

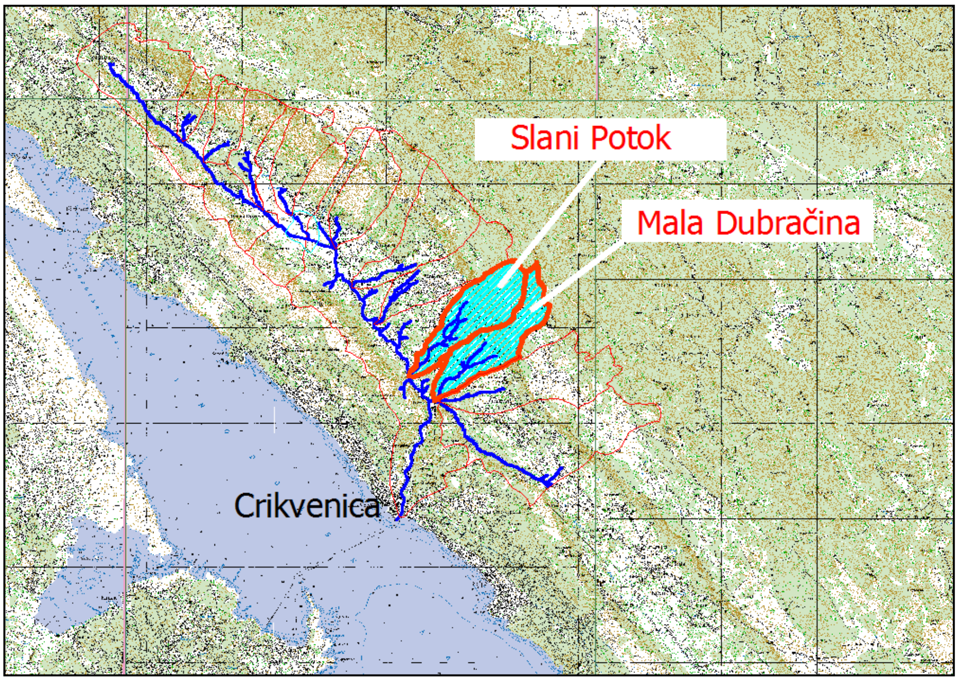


Fig. 7: Most affected areas: Catchment area of Slani Potok and Mala Dubračina [20]

For erosion affected areas, local landslides and torrents, reconstruction projects (roads, riverbed regulations, retaining walls, etc.), as well as geological and hydro-geological research have been performed many times in the last hundred years, especially the ones for the areas of Slani Potok and Mala Dubračina [21, 22]. Unfortunately, without positive outcome and with no prospect in finding the permanent solution of existing problems.
One of the main problems is the loss of some research documentations and projects along the years. Another problem is that data from existing documentation in some parts of the catchment does not correspond to existing data and state identified directly in the field. Also, maps showing roads, rivers, cities and villages are out of date and do not represent the current condition, but can be used for comparison with new data for progress of erosion, occurrence or expansion of landslides and torrent affected areas.
In this paper a GIS based monitoring database as a tool for monitoring, mitigation and prevention of flash flood and erosion impact is presented. The database is created for Dubračina River catchment area in Vinodol Valley (Croatia), which is included in the bilateral Croatian-Japanese research project "Risk Identification and Land-Use Planning for Disaster Mitigation of Landslides and Floods in Croatia" and was chosen for its historical significance of potential hazard development.

3.1. Methodology and data

The final aim of this research is to synthesize existing data and problems in the area of interest. Its first step was gathering of the available information and documentation. At the very early stage of the research, problem with data collection became obvious. Some of data from previous projects and documentation have been lost during the years, and also most of existing and available data is outdated. For this reason, as well as for dissipation of existing information sources, it was impossible to determine the current condition of the river tributaries, catchment areas and their risk of potential hazard. Base maps (Topographic maps, Digital Elevation maps, Ortophoto maps, etc.) are out of date and do not entirely present the current condition in the field (such as riverbed position, changes of road routes, spreading of cities and villages, etc.). For this reason, and in order to obtain more accurate data, the database was complemented with on-site field survey. For the purpose of determination of current area condition, steps of research were established and this paper provides the proposition of the methodology that was used for the determination of current area condition, developed on Dubračina River catchment area, that encompasses several steps of essential research work needed in order to achieve the final aim.

This most essential part of the research is divided into four main groups: collection of existing data, field research, classification and presentation of data which can be seen in Table 1.

Tab. 1: Steps of research divided into main four groups

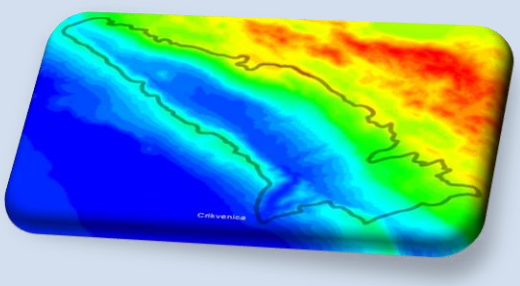
Collection of existing data
• Maps: (Scale 1:2500, Orthophoto, Regional Development Plan, Topographic, etc.) • Photography: Old photos • Reconstruction projects for damaged roads, riverbeds, culverts caused by erosion, landslides or torrents • Research studies: Geotechnical, Geological, Hydro-Geological, etc. • Interviewing people: local population involvement through interviews and public discussions • National, regional and local archives (old photos, newspapers, books, etc.)
Field research
• Filling the prepared form for descriptive evaluation for each river Dubračina tributary and its catchment area in the field. • Photo documentation: Photographing all the tributaries, hydraulic structures and catchment area conditions (April 2013.)
Classification of data
• Organizing into the groups (hydraulic, geological, land use, anthropological and historical data) • Collected data verification (comparison of the collected and existing data) • Overlapping of collected data
Presentation of data
• Database and map overview in the software ArcGis 10.1. • Map overview with photo documentation that has easy access to the all relevant information • Attribute table for every inputted object

Preparation of the database and map overview with photo documentation is a first step and probably the most important step towards the development of a detailed database, which requires constant growth and upgrade in the upcoming years.

3.2. Information GIS database for research area Dubračina river catchment area

Database with organized and correlated information is made in the software Arc Gis 10.1. Information such as hydraulic (river network, river regulation, etc.), geological (soil type, erosion and landslides affected areas), land use (types of vegetation coverage, areas used for agriculture, etc.), anthropological (urban areas, traffic infrastructures, illegal waste disposals) and historical data (affected areas in past, implemented structural and non-structural measures) are processed into a database for the purpose of the flash flood and erosion analysis.
The collected data can be divided into two main groups: existing data and new data (obtained by field overview) after which these data was classified into sub-groups (hydraulic, geological, land use, anthropological and historical data) and incorporated into the ArcGis database and map overview with photo documentation. Examples of incorporated data within layers in ArcGis are presented in tables (Tab. 2 and Tab. 3).

Tab. 2: Examples of existing data incorporated within database

	PRESENTATION OF DATA IN ArcGis	TYPE OF DATA	DESCRIPTION OF DATA
EXISTING DATA		Base map	Layer consisting of Croatian Topographic Maps with Dubračina river catchment area (orange area - research area) Scale 1:25000
		Base map	Layer consisting of Elevation raster map conducted from Digital Elevation Map with Dubračina river catchment area

Tab. 3: Examples of new data incorporated within database

	PRESENTATION OF DATA IN ArcGis	TYPE OF DATA	DESCRIPTION OF DATA
NEW DATA		Hydraulic	Layer consisting of springs (blue points), river network (blue lines) and lakes (light blue area) obtained from existing river network map that is revised to the current condition after field research (displacement of the riverbed, occurrence or disappearance of riverbeds…) with Dubračina river catchment area and tributaries catchment area
		Hydraulic	Layer consisting of river network (blue lines) and lakes (light blue area) obtained from existing river network map that is revised to the current condition and all hydraulic objects (riverbed regulations, embankments and culverts...) after field research with Dubračina river catchment area
		Geotechnical	Layer consisting of river network (blue lines) and lakes (light blue area) obtained from existing river network map that is revised to the current condition and all places where erosion or landslides caused road damage (red car symbols)

For the purpose of creating a detailed data base on which is possible to apply analyses such as assessment of current condition, hazards risks or their prediction, it was necessary to combine available data for each element, in a form of attributes arranged in multiple tables. Examples of such attributes for the group of objects are: Name, Length (m), Area (m^2), Building material, Geometrical shape, Condition of object, Photo documentation, etc. Most important attributes among mentioned ones, are Condition of the object and Photo documentation because they are showing current condition of the researched area. To provide uniform evaluation of condition for different groups of objects (for example: Culverts, Riverbed, Springs, etc.) simple descriptive attributes were assigned for each group element, along with its numeric value. An example of such ranking can be seen in Table (Tab.4).

Tab. 4: Example of descriptive evaluation with assigned number values for condition of culverts

NUMERIC VALUE	Description of the culvert condition
5	Excellent condition: no need of any reconstruction, the flow is not disturbed by any obstacles
4	Very good condition: traces of sediment transportation, the flow is not disturbed by described obstacles
3	Good condition: evident sediment transportation, the flow is potentially enabled by described obstacles
2	Poor condition: large amount of sediment transportation, vegetation inside of culvert, the flow is partially enabled by described obstacles
1	Very poor: culvert is full of sediment, the flow is fully enabled by described obstacles

Among all, during the field research, photo documentation of each structure was made and later implemented in the attribute table providing the user quick and easy insight of its condition. Example of attribute table for one culvert object is shown in Figure 8.

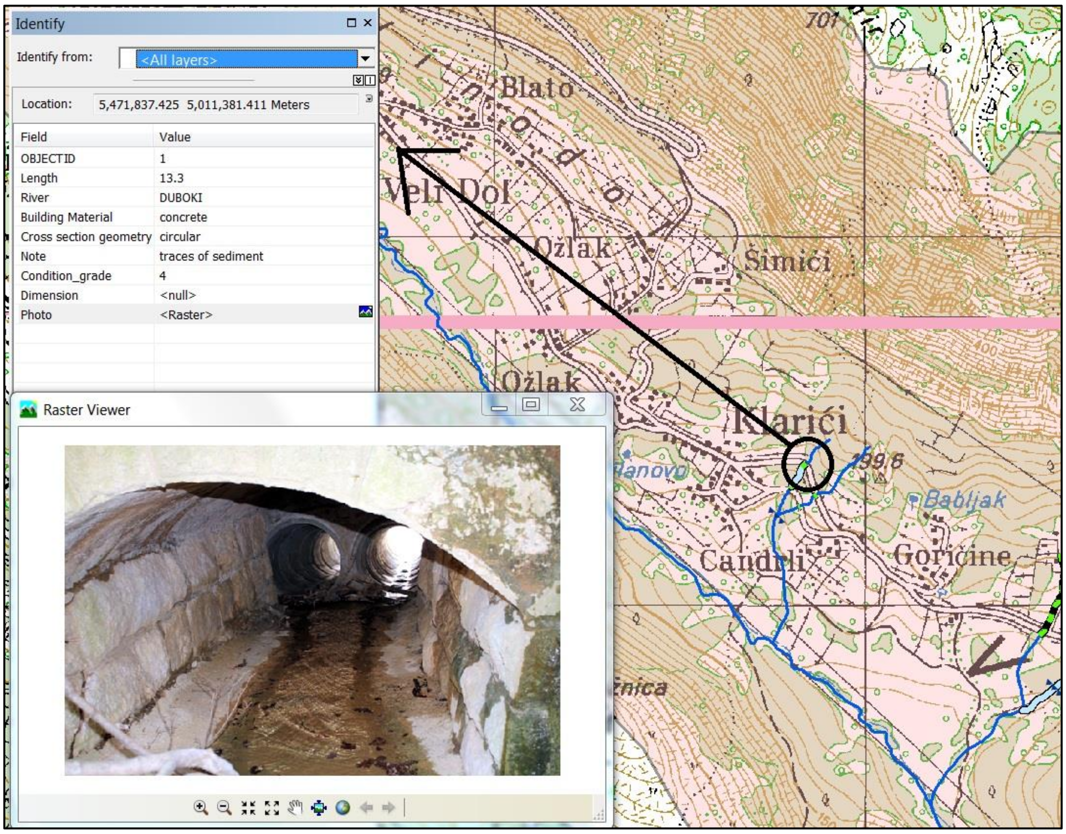


Fig. 8: Example of attribute table with photo documentation for culvert

4 DISCUSSION/CONCLUSION

Because of its complex geological structure and distinct steep slopes, significant natural and cultural values, as well as existing problems with erosion, landslides and torrential flow, Dubračina river catchment area was chosen as research area and area of potential risk hazard, upon which an integrated database was made.

In the past there were many existing data, but thought the years many of them had been scattered, forgotten or even lost, and some of remained data are not up to date, which is why all data integration was necessary. Among them, the most important ones could be divided into hydraulic, geological, land use, anthropological and historical groups of data. Preliminary research that was consisted of gathering existing information, projects from numerous sources (local government, archives, local and regional newspaper, field work and overview, information exchange and interview with local population, etc.) were all processed into an organized and correlated information database and map overview. This database as any other requires constant growth and upgrade in the upcoming years, in order for it to be up to date and available to the wider range of final users.

Long-term monitoring will lead to the improvement of hazard risk assessment, and future decision making processes regarding implementation of mitigation and prevention measures. Continuity in monitoring will enable future comparison between past, present and future on site conditions, and provide better evaluation of conducted measures and changes in the area, as well as its positive and even negative side effects. This will allow future decision makers experts and planners better evaluation and selection of planed actions for Dubračina River catchment area, whether it regards urban development (water supply system, transportation systems, city and village's growth, industrial infrastructure etc.), hazard (mitigation and prevention measures), agricultural development (selection of area that pose minimum risk of hazard, definition and recommendation of allowed actions toward sustainable agriculture and hazard prevention, selection of culture for future growth, etc.), or any other implementation depending on the final user's needs.

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FLOOD PROTECTION OF VILLAGE KRÍŽE, BARDEJOV DISTRICT

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Abstract

The aim of the paper is the study of flood protection of Slatvinec catchment situated in the north part of Slovakia. The first part of the paper is focused on the overview of the current state of knowledge in the field of flood protection and designing flood protection measures. The next part is devoted to the assessment of the contemporary state of the selected catchment. It include a methodological procedure that analyse the characteristic of the area taking into account the natural conditions of the area such as climate, the local geology, hydrology, water management, soil conditions and other related issues. The last part is oriented on the proposal of flood protection of Slatvinec catchment with focusing on technical and biotechnical flood protection.

Keywords

Flood protection, flood protection design, Slatvinec catchment

1 INTRODUCTION

Flood control is protection against the effects of high water. It is an extensive whole society activity, which belongs to the oldest wilful human activity. By the system measures, we seeks to prevent or, in case flood orignin to reduce or regulate the negative consequences of water.

This study is dealing with flood protection of Kríže village. It is settlement in the eastern part of Slovakia, laying in the Bardejov district. The village is sitated in the upper part of Slatvinec flow, which with its tributaries causes floods not only in this village, but also in the lower flow part situated villages. During the intensive rainfalls it is created the larger flow on Slatvinec stream and on its left-sided Solisko tributary. This cause that it is originated such

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water volume into channel, which is not possible to transform. Except of volume, water increases its force too, what cause that under water is transported consideralble amount of the different structure material. This material is transported and deposited along the stream and during the bridge construction crossing it can cause plugging and subsequent breakdown. The most suitable flood control of the village is construction of the non-overflow polders together with the construction of weirs in the river tributaries and the vertical alignment of the channel bottom modification.

Size of floods and the their process depends on many factors, the most important are rainfall and run their course, the amount of snow cover and air temperature, the shape of catchment and river network, geology, agricultural use and function of the forest, as well as the level of flood protection measures built in the catchment. The size and timing of floods affect unexpectedly. Flood control measures, however, can prevent and possibly alleviate the damage which can be done. Measures for flood control are carried out preventively during an emergency flood, during the flood and after the flood. The best result of flood protection can be achieved by building technical measures, land use planning, technical measures such as the construction of dams or river regulation. Furthermore, it is the construction of dams and polders which retain water. Troubleshooting water is no longer just a matter of water. It is a matter of social, cultural and existential, therefore, affects all.

The aim of this paper is the proposal of measures for flood protection of Kríže village in the Slatvinec catchment in Bardejov district.

2 MATERIAL AND METHODS

Measures for flood protection in the basin are intended to increase the retention capacity of the territory and increase its infiltration capacity and thus reduce flood risk to an acceptable level. Addressing flood protection is long and reaches far into the past [1].

The methodology of this work was to analyze the main factors of floods in the collection of information that is subsequently processed into the final study [2].

Work methodology consists of the following steps:

- Materials - maps in scale 1:50 000 and 1:10 000, climatological data.
- The survey of the area - photo documentation, field mapping necessary for further evaluation of the current situation.
- Evaluation of the current situation:
  - Climatic conditions
  - Hydrology
  - Geology
  - Pedology
  - Vegetation
  - Protected areas
  - Forestation
- Proposed measures for flood protection

2.1 Study area

There are three municipalities - Kružľov, Bogliarka and Kríže in the investigated Slatvinec catchment (Fig. 1).

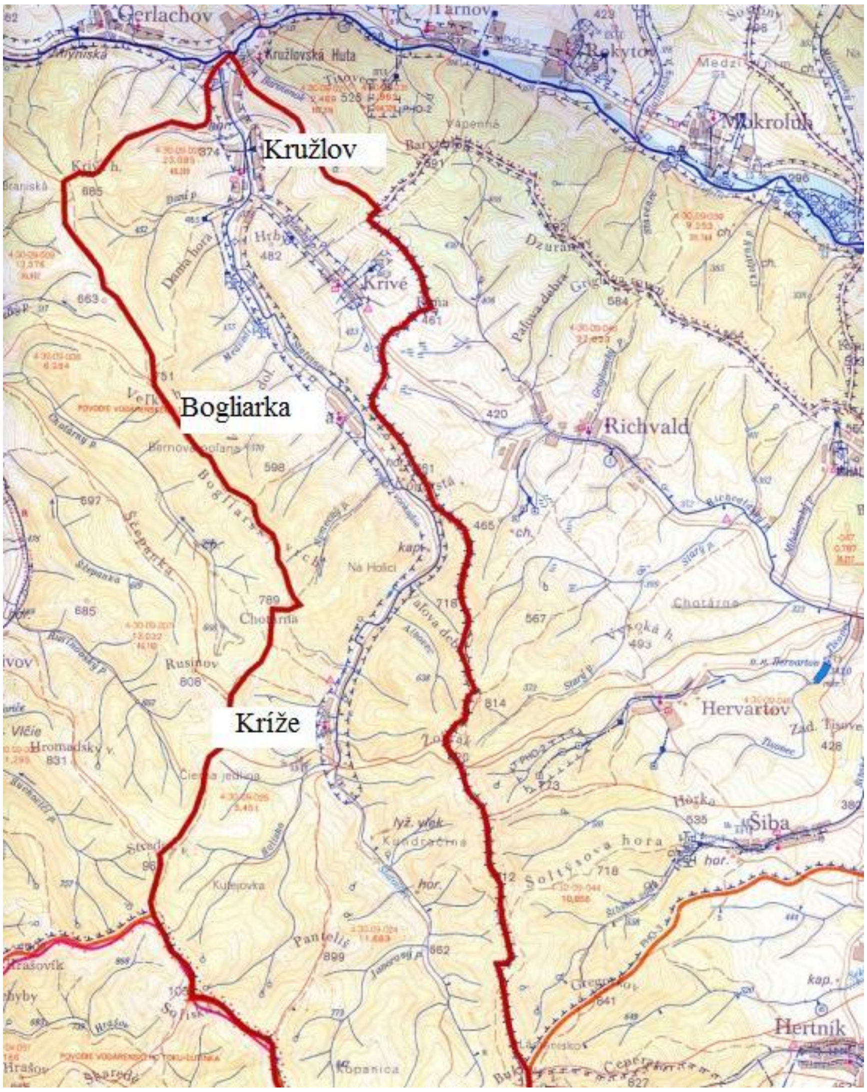

Fig. 1 Slatvinec river basin

Catchment area is 40.219 square kilometers. Village Kríže is located in the upper section of the water stream, at an altitude of 560 m above sea level. Slatvinec flows through built-up area of the village. It is a right tributary of river Topľa and has a length of 15.5 km. It rises in Čergov on the eastern slopes of the Veľká Javorina (1,098.7 m) at an altitude of about 900 m above sea level. Its tributaries are from the right under Bukový vrch (1,018.9 m), Albovec and Krivský potok, and from the left Javorový potok (1,028.9 m above sea level), Solisko and Nemecký potok, a tributary of Vápenná dolina, Medvedí potok, a tributary of the fields of the Dania hora.

3 RESULTS

3.1 Climatic conditions

Data of total precipitation are shown in Table 1 and Table 2. These data were obtained from Slovak Hydrometeorological Institute (SHMI) for years from 1980 to 2010.

Tab. 1 Total precipitation

Total precipitation		Kríže	Osikov	Malcov	Livovská Huta
Maximum	mm/day	80.0	55.7	82.0	97.2
Average	mm/day	5.7	4.9	4.2	5.9
Max. amount of precipitation	3 days	148.0	108.3	141.2	163.8
Max. amount of precipitation	5 days	187.0	138.0	167.0	193.8
Max. amount of precipitation	7 days	218.3	148.3	181.4	218.8
Max. amount of precipitation	15 days	317.3	251.5	225.2	311
Max. amount of precipitation	mm/year	846.8	637.9	718.5	855.3

Tab. 2 Climatic characteristics in station Bardejov

Bardejov	Temperature at 14.00	Daily average air temperature	Total atmospheric precipitation	Atmospheric precipitation duration - 3 days	Atmospheric precipitation duration - 5 days	Atmospheric precipitation duration - 7 days
	°C	°C	mm	hour	hour	hour
Maximum	35.1	28.6	63.2	66	84.7	97.7
Average	12.4	8.5	4.3	2.75	3.529167	4.070833

Basin belongs to the cooler area with an average annual air temperature of about 4 C, in January ranges temperatures from -4°C to -7°C, in July from 12°C to 16°C. The annual rainfall is around 800-900 mm. The morphology of the area affects rainfall conditions. Torrential rainfall events occur mostly in the summer months, and are crucial for runoff in the area of the interest. They cause severe flood situations occurence. Average number of days with snow cover is 80-120 days. The average maximum snow depth is 40-60 cm. Decline of runoff is obvious in winter because precipitation is falling mainly in the snow form. Losses and infiltration of the vapor at low temperatures and frozen soil are small.

3.2 Hydrology

Bodrog and Hornád river basin occupies 30 % of the area of the Slovak Republic. Bodrog occupies the eastern part of Slovakia. Its total catchment area in Slovakia is 7,272 km^2. River basin as a whole has a very low forestation with 37.6 %. It is within the Eastern Lowland catchment region with the lowest forest cover. The highest forests cover in the catchment basin shows Laborec river basin 45 % and Topľa river basin 41 % (into which Slatvinec river belogs). At this percentage forest cover involved streams in these basins, which have a 70 % forest cover. In the middle, hilly catchment areas, this percentage declines and lower basin are almost completely deforested. Geologically, the upper parts of these basins are largely made up of fragile layers of sandstone and claystone, known as the Carpathian flysch [3].

Slatvinec belog to International Danube basin (drainage area Black Sea) and to sub-basin Bodrog: the confluence Topľa with Ondava 4-30-09. Basic characteristics of Slatvinec stream are in Table 3 [4].

Tab. 3 Basic characteristics of Slatvinec stream

<table>
<tr><td colspan="2">Ranking</td><td rowspan="2">Village</td><td rowspan="2">County</td><td rowspan="2">District</td><td rowspan="2">Stream</td><td rowspan="2">Hydrology ranking</td></tr>
<tr><td>No.</td><td>SR</td></tr>
<tr><td>111</td><td>2011</td><td>Kríže</td><td>Prešovský kraj</td><td>Bardejov</td><td>Slatvinec</td><td>4-30-09-026</td></tr>
<tr><td colspan="3">Length of the stream in the urban area</td><td rowspan="2">Length of the proposed regulation in the urban area</td><td rowspan="2">Average annual flow</td><td rowspan="3">Enterprise</td><td rowspan="2">Basin area</td></tr>
<tr><td colspan="2" rowspan="2">Total (km)</td><td rowspan="2">Regulated (km)</td></tr>
<tr><td>(km)</td><td>($m^3.\ s^{-1}$)</td><td>km^2</td></tr>
<tr><td colspan="2">0,90</td><td>0</td><td>0,90</td><td>0,21</td><td>Bodrog river basin</td><td>19,09</td></tr>
<tr><td colspan="7">Channel capacity</td></tr>
<tr><td colspan="4">in a critical profile</td><td colspan="3">in a treated profile</td></tr>
<tr><td colspan="4">Flow, N-annual water ($m^3.s^{-1}$)</td><td colspan="3">Flow, N-annual water ($m^3.s^{-1}$)</td></tr>
<tr><td>29.70</td><td colspan="2">Q_{20}</td><td>31.50</td><td colspan="3">-</td></tr>
</table>

The largest historical floods often fall to the middle of the summer. The largest floods were in 1813, 1893 and 1913, and affected the whole Bodrog. It was in August [5]. Hydrological regime of the river and development of river network are influenced by geological composition of the basin.

3.3 Geological conditions

Solved Topľa upper river basin lies in subassembly Carpathians, which represent orographic units Čergov, Busov and Ondavská vrchovina [6].

Tab. 4 Regional geomorphological division

Geomorphological units	Orography
System	Alpine-Himalayan
Subsystem	Carpathians
Province	Eastern Carpathians
Subprovince	Outer Eastern Carpathians
Region	The Low Beskydy
Area	Ondavská vrchovina Čergov

Geological structure is built of flysch zone and quaternary sediments. Quaternary sediments are mainly represented by sediments with gravel, sandy gravel overlap by sandy loams. They built narrow alluvial strips along both sides of watercourses: Topľa, Kamenec, Rosucká voda, Šibská voda, Černinka, Kurimka and Hažlínka [4].
River basin is located on the outside of the Western Carpathians. The geological substrate consists mostly of resistant sandstones and claystones which grow on beech forests.

Morphology and soil conditions adversely affect the ratios of water basins, especially substitution of permeable sandstone layers and layers of impermeable clays. In the valley Slatvinec occur covert transfers involved in the drainage of the area. The area is rich in slope deformation, especially the formation of landslides in the stream area due to various rock complexes (sandstones, claystones).

3.4 Soil conditions

Land area solutions are represented mainly by cambisols and fluvial.
There is a great danger of soil destruction in the flysch areas, especially after the removal of vegetation. The soils are susceptible to erosion and landslides risk. It reduces the production capacity of forest land which has a direct negative impact on the ecological stability of the environment [7].
In a large part of the area are predominantly sandstone soil mixed with clay. These soils mainly on forest roads after rains created muddy sections which is navigating the mud slurry directly into the watercourse. The ability of soil to retain water or infiltration capacity is small, causing rapid evacuation of water from the surface of these soils. There is dominant runoff in the area, and infiltration of precipitation is small due to saturation of the soil. Steep slopes and valleys of large gradients from which water rapidly flows down by gravity due to the rapid saturation of the soil water also contribute to reduce the infiltration. In some places bedrock stands up above the stream bed, which means that in these areas the implementation of flood control is very difficult and challenging. The bottom stream is made up of mostly gravel, but the rocks of various shapes, sizes and weights.

3.5 Morphometric parameters relief

Morphometric parameters of the relief of the area can be derived from DMR (Digital Terrain Model). From the crated DMR [8] of the upper part of the basin (Fig. 2) is possible to derive vertical division of the territory and determine the value of $H_{max.}$ and $H_{min.}$ altitude of the area. For our territory is $H_{max.}$ = 1,095 m and $H_{min.}$ = 485 m above the sea level (Table 5).

Tab. 5 Altimetry of the area

Hypsography degrees	Area (km^2)	Area (%)
485-553	0.767	3.343
553-621	2.068	9.016
621-689	3.384	14.752
689-757	4.287	18.688
757-824	4.692	20.453
824-892	3.736	16.286
892-960	2.201	9.595
960-1027	1.397	6.089
1027-1095	0.408	1.778
Total	22.94	100

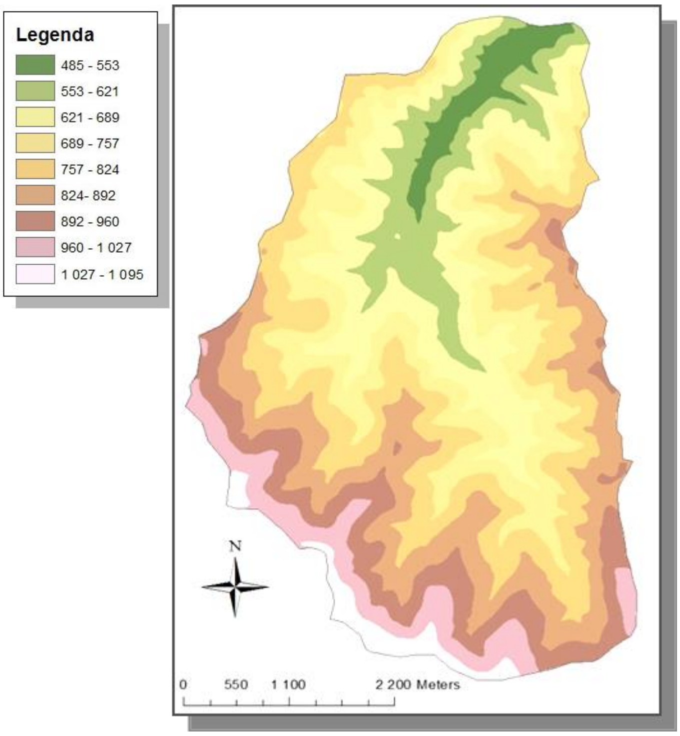

Fig. 2 Digital Terrain Model

3.5.1 The slope of the area

The slope of the area is the most important indicator for the evaluation of the velocity and thus the rate of flow of water. The slope of the area and the catchment area pertaining to individual slope values are found in Table 6. The slope topography has a great influence on potential runoff of the area. The table shows that most of the area is situated on the steep slopes from 12° to 25°.

Tab. 6 Slope of the area

Slope	Area (km^2)	Area (%)
0-7	1.211	5.279
7-12	2.131	9.289
12-16	3.462	15.091
16-19	4.406	19.207
19-22	4.253	18.539
22-25	3.348	14.595
25-28	2.293	9.996
28-33	1.344	5.859
33-45	0.492	2.145
Total	22.94	100

Steepest areas are located behind the village Kríže, where the slope exceeds 25 degrees. In these places there were sliding during the floods. It is necessary to stabilize them and reduce the risk of recurrence of the landslide. This stabilization can be used by gabions – wire structure filled by rocks – mm that is exported to the construction of the retaining walls.

3.5.2 Exposure

The next morphometric parameter is the orientation of the relief to the cardinals – exposure, which percentage of the distribution for each cardinal is given in Table 7.

Tab. 7 Exposure of the area

Slope		Area (km^2)	Area (%)
337.5-360	N	1.47	6.408
0-22.5	N	1.453	6.334
22.5-67.5	NE	4.516	19.686
67.5-112.5	E	3.710	16.173
112.5-157.5	SE	2.259	9.847
157.5-202.5	S	1.43	6.234
202.5-247.5	SW	1.68	7.323
247.5-292.5	W	2.902	12.65
292.5-337.5	NW	3.52	15.345
Total		22.94	100

Relief orientation to cardinal can be considered stable exposure of the relief operation to the sun. Exposure determines the direction of surface runoff, combined with a tendency to establish the amount of solar energy incident on the earth's surface, which is a factor underlying the processes of evapotranspiration and water circulation in the country. With the rising value of the slope increases the importance of focusing especially on sunny parts of the slopes, where there is a deterioration of soil properties and the weakening of the vegetation layer. Relative orientation plays a big role and to prevailing winds (aeolian processes, accumulation of snow, et al.) and snow melting.

3.6 Protected areas

There is protected area Čergov situated in the investigated area. Code of territory is SKUEV0332. The time period of validity of the conditions of protection is from 1st January to 12th December each year. Cadastral area: Bardejov District: Gerlachov, Malcov, Lenartov, Kružlov, Lukov, Richvald, Krivé, Venécia, Bogliarka, Livov, Livovská Huta, Hervartov, Kríže, Šiba, Hertník, Fričkovce, Osikov. The area is designed to protect the habitats of European importance.

Vegetation covering in the basin and around the stream is predominantly green liner and natural forests. Natural forests of different species composition have a high retention capacity. In addition, thez provide other irreplaceable landscape and ecological functions. The banks of the watercourse along the length of the tree covered islets, which is accompanied by shrubs and greenery. Vegetation in the basin contributes to soil protection and water protection, and significantly reduces the disturbance of soil erosion. Valley slopes are mostly covered by forests, but also permanent grassland.

Removing linear and scattered vegetation, compaction of forest roads heavy machinery reduces the inherent storage capability and contributes to accelerated runoff. As a result of this disturbing vegetation may occur to an increased risk of flash floods.

3.7 Forestation

Solved territory consists of 80 % forest. The purpose of 90 % of the forest is timber production and other forest products, while ensuring non-productive functions of forests. These forests are known as commercial forests. In addition to the economic functions of forests in the area are also protective forests (7 %), which main function is to protect the land bank line or low-lying vegetation, and special purpose forests, which are designed to provide the specific needs of companies, legal entities or natural persons whose security is significantly changing the way of the management.
Slatvinec catchment area is characterized by high forest cover. Forests are made up mainly of beech trees. Since the Slatvinec is located in the valleys, forests are planted on steep slopes. In the area of interest is active logging, which has a negative impact on the entire river basin. Forest has an important function in terms of hydrology and water purification during the floods. The logging influenced water runoff from the catchment significantly. This activity leads to compaction and subsequent solidification of forest roads. Paved roads cause that water from the slopes of the valley flows directly into the stream. The concentrated runoff occurs on forest roads. On certain stretches the forest roads pass directly through a watercourse, which results in clogging the flow of mud and stones.
Logging cause also the deforestation of the area, this results in formation of erosion phenomena. Result of logging is more water drained from the basin and consequently it increases the maximum flow rate in the stream. This activity is undermined of water regime function of the forest, and there is an increase aquosity flow, which can affect the more frequent occurrence of floods in the area.

4 PROPOSAL OF MEASURES FOR FLOOD PROTECTION

In addition to stabilizing the riverbed bottom vertical alignment, is necessary to ensure the passage of the flood wave deceleration. It is possible the construction of dams in the stream, which must be designed in several places in the stream Solisko. To build dams in the stream can be used by local sources [9].
In combination with the weir under consideration and dry retention tanks, called polders. It represents a place where they could be part of the water, if necessary, detain and then, after passing flood wave released. In terms of total area, it is necessary to respect natural resources and not to interfere with them, and therefore these technical measures must as far as possible be designed sensitively to the area. Both streams are located in the PLA Čergov, but the territory is valid only II. degree of protection. This means that before any intervention into the country and location of the watercourse shall require nature conservation authority (District Environmental Office), as specified in the Act No. 543/2002 Coll on nature and landscape protection, as amended. Before construction it is necessary to obtain the permission of any other relevant entities (municipalities Kríže and Hervartov, State Forests of Slovak Republic, Administrator flow - Slovak Water Management Enterprise).
Field surveys were identified suitable sites for the construction of the polder, these are shown in fig. 3 and fig. 4. It would be taking the overflow of polders. Before polder area is necessary to place space that would be used to capture large objects and to prevent sediment deposition.

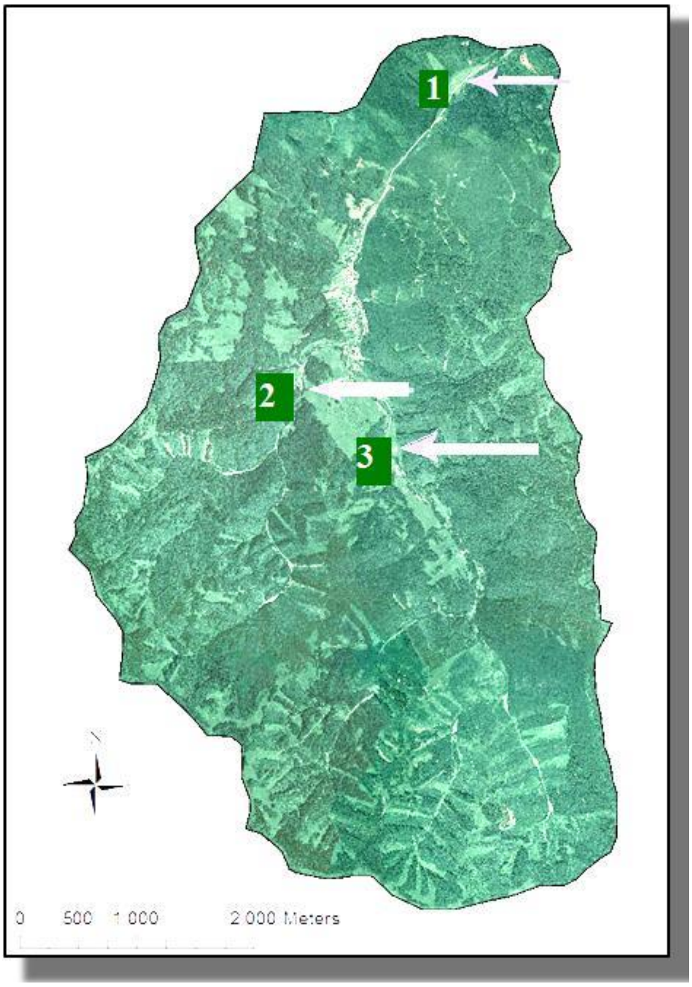

Fig. 3 Sites suitable for construction of polders

Fig. 4 Suitable place to build a polder

Polders would be filled only during the flood situation. In terms of technical solutions, care and maintenance is a better solution compared to overflowing valley polders.

5 DISCUSSION

Realistic calculations [8] confirmed the state, which took place in the village in recent years, and that the channel capacity in the village is not sufficient to catch flood waves. It is necessary to take appropriate action, as the space in the village is very limited (narrow) to be addressed flood protection before the arrival of the flood wave into the village. Such measures include the construction of accumulation weirs on tributaries. These would be in addition to the accumulation of material mounted fulfilling the function of reducing the flow velocity and deceleration of the flood wave. Since 80 % of the rural area consists of forest land and the small tributaries of Slatvinec and Solisko are in a state forest administration of Slovak Republic, needed their assistance.

The proposals of polders, which are included among water management works in terms of safety to III. category of water works. Its proposal proposal must be handled by qualified experts and reviewed by Slovak Water Management Enterprise. Sediments from river bed should be used either to build retarder across the stream or could be used for example for construction of gabions for the weir in forest streams. Other measures of the country, to be used in lowland areas such as infiltration strips to create agricultural land, cannot be used in a given locality.

6 CONCLUSION

The aim of the paper is an assessment of the possibility of flood protection for the village of Kríže. Kríže lies in the east of Slovakia and has a stream – the Slatvinec – flowing through it. After heavy rainfall the Slatvinec tends to burst its banks and threaten the local community with flooding. The paper includes an analysis of a characteristic area taking into account the natural conditions of the area such as climate, the local geology, hydrology, water management, soil conditions and other related issues. In conclusion, this work has evaluated all the information available for producing the final record. In this case it is a solution to the flood protection issues at the village of Kríže by ensuring a reduction in the kinetic rate and diminishing flood waves at the confluence of the Slatvinec and Solisko streams. Flow conditions would be also improved by the removal of sediment from the riverbed.

The course of Slatvinec and Solisko allows building side polders, which would be filled up only during the flood situation, which is in terms of their technical solution, care and maintenance of a better deal compared to overflowing valley polders.

Acknowledgements

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WATER AS IMPORTANT GEOLOGICAL FACTOR OF THE ENVIRONMENT IN SLOVAKIA

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ABSTRACT

Water is generally an essential component of the environment that creates natural conditions for the existence of organisms, including humans. At the same time it is a component of the geological environment, which directly affects the conditions for the existence and development of the society that man uses and changes.

Groundwater is one of the most important components of the geological environment. Represents either geological potential i.e. a natural resource which is favourable to development of a society or geological barrier, i.e. factor that limits or prevents the positive development of a society. The article lists overview of geological factors of environment, mainly focused on water, either as geological potential or geological barrier in the geological environment of Slovakia.

Keywords

groundwater, environment, geological environment, geohazards, geological factors, geo-potential, geo-barriers

1 INTRODUCTION

The water is in general the most important element of the environment. It creates natural condition for existence of living organisms including the man kind. It is also part of the geological environment, which directly influences conditions for existence and development of human society, which man use and change.

Ground water represents intersection of hydrosphere with lithosphere; it significantly influences properties and behaviour of rock and soil mass and it is consider as specific type of raw material conditioning the life of man kind [1].

The water is either geo-potential i.e. natural resource positively influencing development of society or geo-barrier i.e. barrier which limits or blocks the society development.

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In the article we are presenting review of geological factors of the environment, with respect to the water, either as geo-potential or geo-barrier in the geological environment of Slovakia. In more detail we will discus influence of water on development of slope movement and their deformations, what is considered as the most important geo-barrier in Slovakia.

2 WATER AS GEOLOGICAL FACTOR OF THE ENVIRONMENT

Geological factors of the environment are those geological factors (i.e. objects and phenomena), which significantly influences the environment of the society, either negatively of positively and are becoming limiting factor of the development. The geological factors, which are influencing the quality and efficient development of the environment are briefly summarized in the table 1 [1].

Tab. 1 Review of geological factors by Matula [1]

GEO-POTENTIALS (resources and possibilities of use of the geological environment)	GEO-BARRIERS (geological barriers and limits unfavourable limiting rational usage of the geological environment)		
1. Geological factors supporting favourable society development	2. Geological factors jeopardizing life and human activities	3. Geological factors decreasing efficiency of construction and operation of engineering works	4. Geological factors deteriorating the environment due to negative man activity
- mineral resources (coal, crude oil, geothermal energy etc.) - building materials, - potable and healing groundwater resources, - good quality agricultural soils - good quality, foundation soils, - suitable areas for waste disposal and landfills;	- volcanic eruptions, - earthquakes, - disastrous slope failures, - inundation, flooding (by rivers, by coast due to tectonic subsidence), - toxic, radiation and other hazards jeopardizing health of population;	- foundation soils with high compressibility and low bearing capacity, - quickly weathering rocks, strongly karstic rocks, - slopes with low stability, - high groundwater level and waterlogged foundation soils, - corrosive groundwater, - seismic areas;	- subsidence of undermined land, - subsidence of land after exploitation of water, crude oil or natural gas, - devastation of the land due to open pit mining and waste dumping, - waterlogged or dry land due to constructions (i.e. water works), - groundwater and soil contamination, due to unsuitable waste disposal, agricultural activity and industrial production;

Geological factors, which are directly effected by water of all kind are in the table 1 underlined. As can be seen from the table, water is dominant geological factor either as geo-potential or as geo-barrier.

2.1 Water as geo-potential of the environment

Geo-potentials are various natural resources and possibilities, which the geological environment is capable to offer for positive development of human society. In to this group of geological factors also belong not only traditional natural resources and water, but also fertile soils, good foundation soils and building materials which were base condition for development of civilization since ancient ages.
From the above mentioned it is obvious that potable and healing groundwater are from the point of view of the water management the most important geo-potentials. Quantity and quality of these waters is closely tied with geological environment in which the waters are accumulated and circulating. The geological environment of Slovakia, as part of Western Carpathian, provides rather favourable conditions for their occurrence. There are resources of good pore, fissure and karst water. Regarding the regional point of view the rich resources of groundwater are present in all engineering geological regions of Slovakia, with exception of some regions with claystone development of flysch zone.
Regarding usage of water and refinement of volume of usable resources of the groundwater, improving efficiency of groundwater collection and distribution and creation of resources of groundwater by regulated infiltration is an important task for geologists and water managers. Ground water protection against polluting as well as optimalization of water usage in agriculture and for spa purposes is especially actual task [1].

2.2 Water as geo-barrier of the environment

In general the geo-barriers are various barriers and restrictions of geological character, which significantly limits or totally blocks useful usage of nature for prosperous development of the human society. Such barrier effects (sometime only hardly reducible) can cause:

- geological factors threatening life and work of man,
- geological factors invoking negative interactions between geological environment and engineering works and thus significantly decreasing effectiveness, durability and safe operation of engineering works,
- geological factors representing reverse negative effects of engineering works and interferences, which seriously damages the environment and create a need for its protection and recultivation [1].

From the characteristics of the geo-barriers it is also obvious, that water has its important place in every group of factors.

2.2.1 Water as factor threatening life and work of man

In Slovakia, as well as in this group of geo-barriers, the water plays crucial role in threatening life and man made works, especially as factor developing slope deformations of all kinds and causing river flooding.
In Slovakia, there are very good conditions for occurrence of slope movements which are resulting in slope deformations. This conditions are good especially thank to geological – tectonic setting suitable for development of slope deformations (flysch rocks, thick deluvial deposits and so on.), which are subjected to climatic conditions (high long-lasting rainfalls, frequent extreme rainfalls, snow melting and so on.) and have suitable hydrogeological

conditions (groundwater changing state and properties of rock and soil, increase of hydraulic pressure and so on.).
In geological conditions of Slovakia the slope deformations thus represent the most serious geo-barrier and are the most dangerous geodynamic process which affects civil engineering activities of any kind. For this reason an engineering geological project called Atlas of slope stability maps of Slovakia in scale 1:50 000 [2] was solved.
During this project in the territory of Slovakia there was registered 21 190 slope deformations which are disrupting overall 257 591,2 what is 5,25 % of total area of Slovakia (fig. 1).

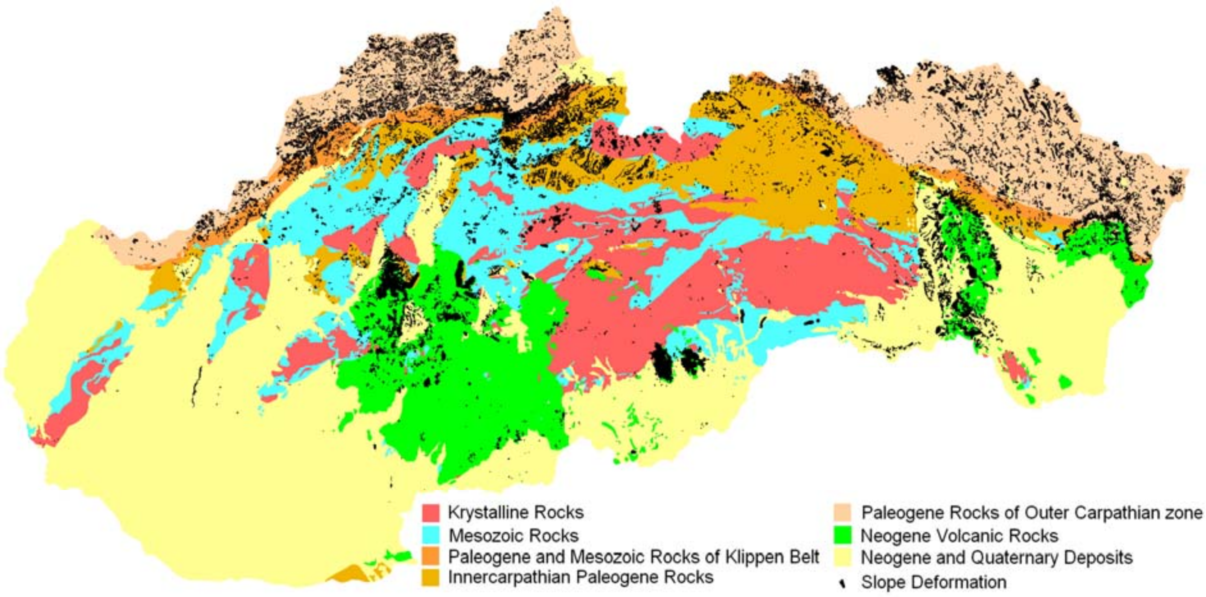

Fig. 1 Schematic geological map of Slovakia with distribution of registered slope deformations [3]

Of all geological units present on the territory of Slovakia (fig. 1) the most damaged by slope deformation are Paleogene and Mesosoic sediments of Klippen belt where 14,8% of their area are damaged by slope deformations. The next most damaged geological unit are Paleogene deposits of outer flysch zone (12,7%). The following units are Neogene volcanic rocks (9,3%) and Inner Carpathian Paleogene sedimentary rocks (7,2%). The least damaged units are Mesosoic rocks (2,4%), Neogene and Quaternary sediments (1,5%) and crystalline rocks (1,5%).
Importance of surface or ground water and its geological activity as important factor is proved also by results of evaluation of causes of development or reactivation of slope deformations in Slovakia. Altogether 21 natural and 9 man made causes of slope deformations were evaluated. Of the natural reasons of occurrence of slope deformation the climate with erosion have the greatest influence (45,2%), follows climate in combination with seeps of groundwater (17%), climate alone (17%), seeps of groundwater and buoyancy of groundwater in combination with erosion of rivers (9,8%). The most common man activities causing occurrence of slope movement are undercutting of slope (49%), undermining (18%) and overloading of slopes (16,3%).
After all it is important to mention, that in Slovakia the slope deformations caused especially by water are threatening large amount of water works. To the most threatened objects belong

water tanks (38,5%), water reservoirs, ponds, weirs (6,3%), water supply pipes (4,2%) and other.
Flooding by river is further factor threatening life and works of man by activity of water in Slovakia. Causes of flooding occurrence are mainly flow rate increase in river bed after heavy rainfalls, snow and ice melting, decrease of river bed flow capacity by ice or sediments and catastrophic events like landslide, water work dam collapse and so on.
Also these phenomena are closely related to geological environment of the country. For example, existence of impermeable subsoil (clayey development in flysch zone where surface water runoff is prevailing over infiltration), large thickness of eluvial or deluvial sediments on slopes which are eroded and transported in to river beds. Catastrophic phenomena like giant landslides or rock falls are rare in Slovakia. However, events related to collapse of protective river side dikes caused by hydrodynamic influences of water (sufosion, liquefaction) has considerable scope For example, Danube river dike was destroyed by suffusion near village Kľočovce in 1965 [4].

2.2.2 Water as factor decreasing effectives of construction and operation of engineering works

In this group of geo-barriers the water plays important role in all factors. Rock state (consistency, volume) and properties (carrying capacity, strength) are being changed by activity of water, especially in the case of soils, rocks containing clay minerals and soft rocks. The changes have influence on quality of foundation soil.
Groundwater is common factor intensively effecting occurrence and development many recent geodynamic processes (weathering, slope deformations, mechanical and chemical suffusion, karst and so on.).
Constructions often change natural regime of groundwater and the crucial task is prediction of these changes along with proposal of measures against negative impact of the changes.
Hydrogeological conditions dictate conditions for performance of construction works (foundation pit drainage, foundation under the water table, measures against breaking of floor of the foundation pit by groundwater buoyancy). Occurrence of the groundwater in the zone of foundation requires number of measures in form of foundation protection against water saturation, damage of the construction by buoyancy etc. In our conditions common corrocive properties of groundwater requires special measures and construction procedure for ensuring durability and safety.
Unfavourable hydrogeological conditions mutilate buildings´ hygiene and health conditions (wet environment with biological consequences) [1].

2.2.3 Water as factor mutilating environment by negative antropogene effects

Negative antropogene effects related to water are causing also subsidence of terrain as consequence of extreme pumping of groundwater. Beside subsidence of terrain the land can be exposed also to excessive drying, which has significantly negative effects on geological environment of the area (change of soil and rock humidity, disruption of natural regime of groundwater, change of quality of agricultural land and so on.).
Drying of land is closely related also to swamping the land, especially as result of construction of water works.
Groundwater polluted by man activities is special geo-barrier. It can be created from high quality groundwater by its pollution i.e. the water as geo-potential is turned by man activity on geo-barrier.

3 CONCLUSION

Water is outstanding geological factor of the environment. It is either geo-potential, i.e. it is natural resource that positively effects the development of human society, or geo-barrier, i.e. barrier which limits or blocks positive development of the society.

Water as geological factor has crucial effect on development and protection of the environment. It follows that we all have to help with protection of water as irretrievable geo-potential. In relation to water as geo-barrier there occurs a task of protection of human society against disruptive effect of water.

Acknowledgements

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REMEDIAL MEASURES ON THE LEFT SIDE FLOOD PROTECTION DIKE OF VÁH RIVER IN KM 23.040 – 27.075

D. Grambličková[1], E. Bednárová[2], M. Minárik[3], J. Babečka[4]

Abstract

The present article concentrates on review of the filtration stability of the left side dike of Váh, its subsoil and the adjacent territory between village Komoča and town Kolárovo at the extreme hydrodynamic loading. At the flood in 2006, here appeared waterlogged areas near downstream toe, outflows with movement of fine particles from subsoil and indications of the lifting of overburden layers. Presence and location of observed negative effects indicated a high probability of excessive seepage or occurrence of preferred paths in the subsoil of dike. Assuming the confirmed existence of risk factors threatening the stability of the dike it was necessary to design appropriate remedial measures and review their impact on the parameters of the filtration flow in the study area. Analysis was performed by finite element method as a transient flow task in a vertical plane using software Seftrans.

Keywords

filtration stability, filtration velocity, finite element method, flood protection dike, remediation measures, subsoil, uplift

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1 INTRODUCTION

There is no doubt, that reliably operated flood protection dikes have positive impact on the environment. To ensure their reliable operation it is necessary to monitor their safety, analyze the risk factors and, where necessary, design appropriate remedial measures. Primary signals of threatening the stability of flood protection dikes can be often detected by monitoring of dikes and their surrounding areas during extreme hydrodynamic loading during floods. This issue was more broadly analyzed in paper [1], where are presented the most often observed concomitant circumstances threatening the safety of flood dikes during floods, causes of their occurrence, corresponding risk factors and possibilities of their remediation. Confirmation or refutation of risk factors should be analyzed by suitable methodological procedure. One of the possibilities is a finite element method (FEM) numerical modeling, which was also applied on selected sections of the left side dike of Váh between municipalities Komoča and Kolárovo in km 23.040 – 27.075 (Fig. 1). Accumulation of observed negative effects testified threat to the stability of dike. A detailed analysis of the development of filtration parameters in the body and subsoil of a dike under extreme hydrodynamic loading may clarify this suspicion, and if necessary appropriate remedial measures may be adopted. Analysis was performed by FEM as a transient flow task using software Seftrans.

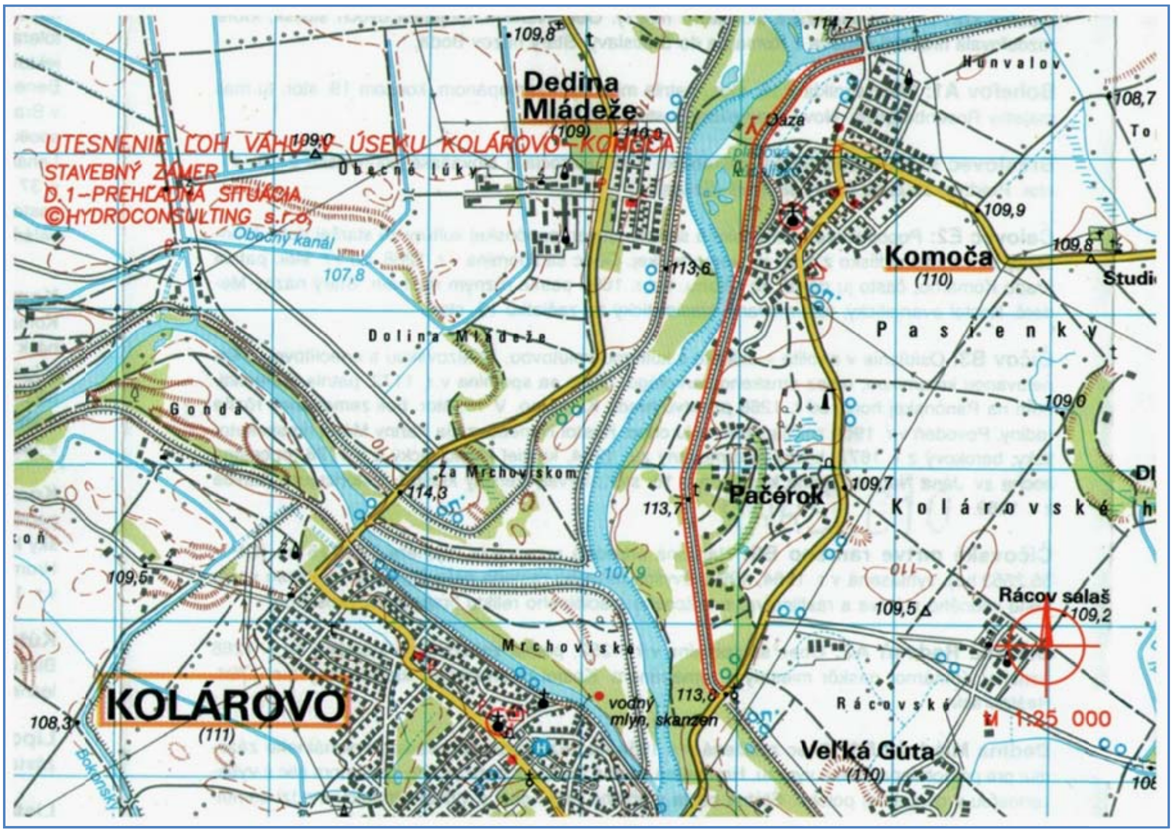


Fig. 1 Situation of scoped area [7]

2 PRINCIPLE OF THE METHOD

The mathematical solution of the transient flow of underground and seepage water utilizes the continuity relation and Darcy’s law [3].

$$\frac{\partial}{\partial x}\left(k_x \frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(k_y \frac{\partial h}{\partial y}\right)+\frac{\partial}{\partial z}\left(k_z \frac{\partial h}{\partial z}\right)=S_S.\frac{\partial h}{\partial t}-q \tag{1}$$

where*:* h je is the pressure head *(m)*, k_X, k_Y, k_Z - filtration coefficients in the direction of the axes *x, y, z (m.* s^{-1}*)*, t - time *(s)*, S_S – specific storage *(*m^{-1}*)*, q - sink or source term *(*$m^3 . s^{-1}$*)*.

Three-dimensional numerical models are very demanding from the point of view of creating model alone as well as very time consuming mainly solving multiple alternative tasks. Therefore is often substituted by solution in vertical or horizontal plane. By plane model approximation formula (1) is integrated over thickness *b* of permeable layer:

$$\frac{\partial}{\partial x}\left(k_x b \frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(k_y\, b \frac{\partial h}{\partial y}\right)=(S_V + b\, S_S).\frac{\partial h}{\partial t}-Q \tag{2}$$

where *b* is thickness of permeable layer at pressure flow. For the flow with free groundwater level is S_v the specific yield for unconfined aquifer and *b* is thickness of saturated permeable layer.

The use of numerical methods without appropriate software is not realistic. In our case, was applied the program Seftrans [2], which allows solving transient filtration flow problems using FEM in the horizontal and vertical plane with a water level under pressure as well as with free surface. Simple rectangular 4-node elements were applied. The discretization of equations was applied according to the Galerkin method. The Gauss elimination technique was used to arrive at a solution. The equivocalness of the equations solution is determined by the boundary conditions on the border of the area investigated. In the case of transient flow, it is necessary to extend them with the initial conditions.

3 LEFT-SIDE FLOOD PROTECTION DIKE OF VÁH RIVER IN KM 23.040 – 27.075

Analyzed section of the left side flood protection dike of Váh River in km 23.040 – 27.075 provides flood protection of the village Komoča and the adjacent area. Flood discharges in on Váh River are here combined with flood discharges in the Danube River [4]. For the occurrence of discharge Q_{100} in the Dunube and the Váh correspondent water level at km 23 is on the elevation of 113.88 m a.s.l. During the flood in year 2006 the water level reached elevation 112.71 m a.s.l. Even at this water level occurred on the left side flood protection dike waterlogged areas along its toe, outflows with movement of fine particles from subsoil and signs of lifting of the overburden layers. Presence and location of observed negative effects indicated a high probability of excessive seepage or occurrence of preferred paths in the subsoil of dike. Analysis of the filtration flow parameters under extreme hydrodynamic loading may review stability of flood protection dike and eventually also design of the remedial measures

3.1. Morphology of the territory and engineering-geological composition

Effect of hydrodynamic loading on the development of parameters of the filtration flow is very demanding issue due to numerous parameters significantly affect changes in filtration regime. Besides hydrological data (flood water level, duration of flood, steady-state water level), engineering-geological data, the sufficient attention should be paid to the morphology of the territory (height of the dike, the terrain level, the length of the foundation base and other). In the case of flood protection dikes is due to their line nature this problem particularly essential. To represent investigated territory with sufficient accuracy the computational model should be therefore carefully designed. Often it is necessary to execute analysis in several cross sections.

The elevation of the crest of the left-side flood protection dike of Váh River between village Komoča and town Kolárovo range from 114.75 m a.s.l. in km 23.1 to 115.23 m a.s.l. in km 27.05, with the average value 115.0 m a.s.l. Its height above terrain is variable, range from approx. 4.4 m to approx. 6.2 m. The width of the dam's crest is mostly 4 m, locally more. Inclination of the upstream slope of dike is in upper part approx. 1:3, in lower part approx. 1:6. Inclination of the downstream slope is approx. 1:3. In the body of flood dike are generally inbuilt clay soils with permeability 10^{-7} m. s^{-1} occasionally - in km 27 were proved at the top of dike position of loamy sands ($k_{f,p}$ = 1. 10^{-5} m. s^{-1}). In Fig. 2 are with thin lines plotted cross sections of investigated segment of dike [4]. Given the relatively acceptable differences (dike's height, the terrain level, inclinations of slope), it was possible to choose for analysis one representative computational model. Its cross section is imaged in Fig. 2 with thick red line. It takes into account the various parameters on a given section of dike so that the results of the analysis are with sufficient reliability transferable to the whole examined section [5].

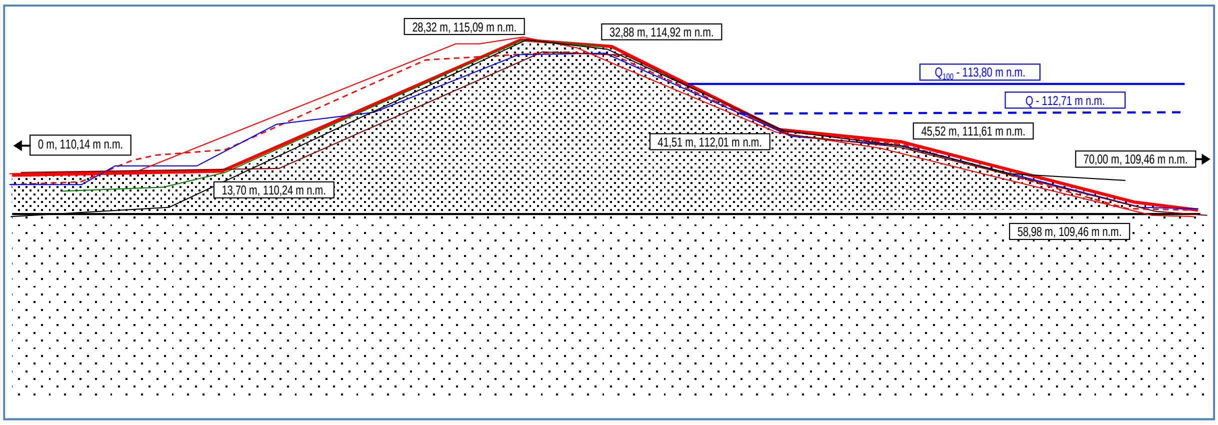


Fig. 2 Cross sections of left side flood protection dike in km 23.040 – 27.075

According to the available engineering-geological survey consisting of the 11 exploratory works [6] subsoil of a dike at a given section (approx. 4 km) has varied geological composition. The overburden layers are mainly composed of fine-grained cohesive soils, locally also cohesionless soils. The vast majority of these are clayey soils, respectively sandy loams with a thickness from 1.5 m to 2 m, locally up to 3 m. Their permeability varies on the order of 10^{-9} m. s^{-1} to 10^{-7} m. s^{-1}. Under the overburden layers to a minimum depth of 50 m was in a relatively strong representation demonstrated occurrence of the poorly graded sands (SP). On their contact with overburden layers and locally also in higher depths around 9 –10 m are poorly graded sands interspersed with sands with admixture of fine particles (S-F). The average values of the coefficient of filtration of sandy soils in the subsoil ranging from approximately 1.10^{-5} m. s^{-1} to approximately 5. 10^{-4} m. s^{-1}. Territory in a given section is flat. On the downstream toe of the dike and in surrounding area local terrain depressions are situated up to depth 2.0 m. Average recorded groundwater elevation is 106.5 m a.s.l.

3.2. Assumptions of the calculation

Geological environment in given section is characterized by the occurrence of low permeable overburden layers with relatively high permeable sandy soils beneath them. In such geological composition determines the intensity of the filtration flow permeability of the aquifer. In the calculation model, we consider its permeability of 3. 10^{-4} m. s^{-1}, which resulted from a set of 20 samples, corresponded to permeability with approximately 75% probability of occurrence. The results of penetration tests confirm in upper parts of the aquifer occurrence of mostly loose sands to a depth 5 m, while at greater depths than 5 m were proved predominantly medium dense sands. This fact was therefore taken into account in alternative solution with permeability of loose sands to a depth 5 m 5.5 .10^{-4} m. s^{-1}, to medium dense sand has been assigned permeability 7.5 .10^{-5} m. s^{-1}. Subsoil permeability was chosen to consider each extreme value of risk factors. Assumptions of calculations with alternative values of permeability of the subsoil as well as the considered thickness of subsoil are apparent from Fig. 3.

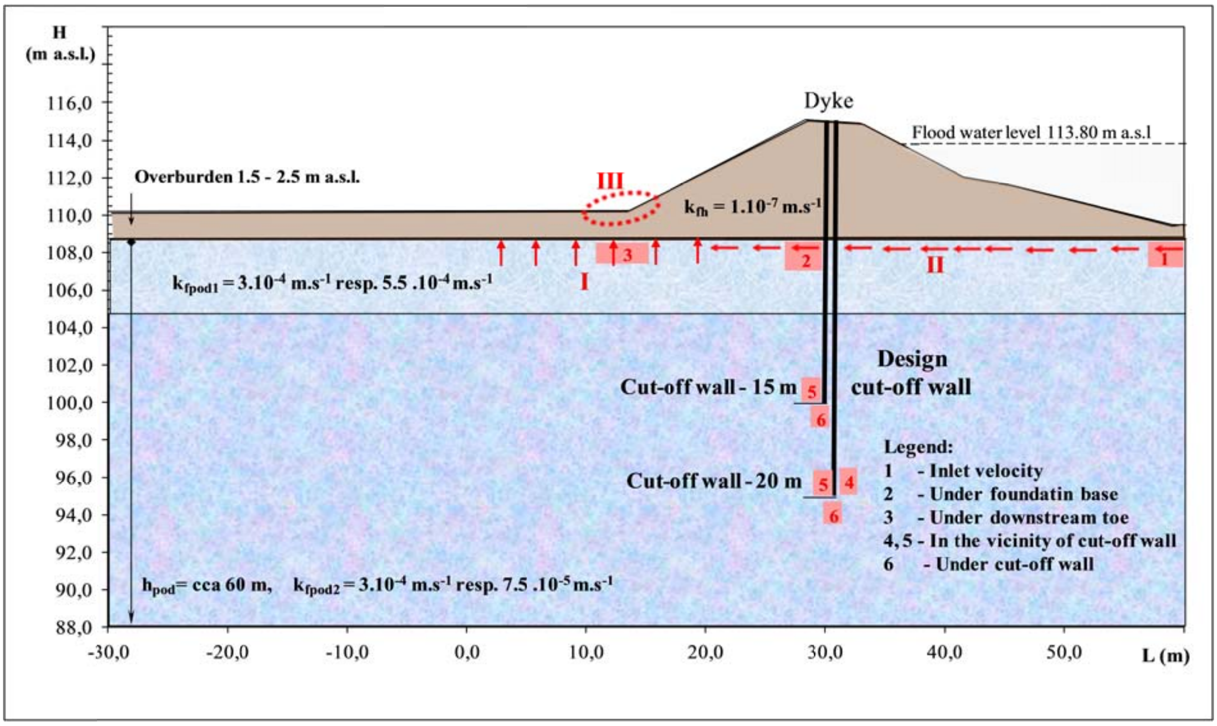


Fig. 3 Assumptions of the calculations and risk factors

Besides the permeability of aquifer, on the development of pressure conditions in the subsoil has also important effect the lower edge of overburden layer. Based on engineering-geological survey its position range mainly from 107.60 m a.s.l to 188.6 m a.s.l. and this range were respected in numerical model in various alternative solutions.

For breaking of the low permeable up to quasi-impermeable overburden layers with uplift and waterlogging of the downstream toe of dike has a dominant influence thickness of overburden layers and thus position of the terrain. In the numerical model has been considered terrain elevation 110.14 m a.s.l., what respects the average value determined from the survey.

Assumed the position of the lower edge of the overburden layer its thickness was chosen within the limits 1.5 - 2.5 m. The results gained from such designed alternatives of solution can be relatively easily applied to other terrain levels.
Parameters of extreme loading entering the numerical model - water levels in Váh River at an elevation of 113.8 meters a.s.l. with duration of 12 days - were derived from the hydrological data of flood discharge Q_{100} at a given section. Flood water level is increased above the average groundwater level 106.5 m a.s.l. about 7 m.

3.3. Analysis of risk factors

Given the geology of the subsoil as well as the observed negative effects in the monitoring of adjacent areas during floods the analysis of the filtration flow under extreme hydrodynamic loading was concentrated on the development of pressure conditions in the subsoil (loading of the overburden caused by uplift – on Fig. 3 marked as "I"), on the development of filtration velocities respectively gradients in areas of sand susceptible to suffosion (Fig. 3 – "II") and to risk of waterlogging of the downstream slope and toe of dike (Fig. 3 – "III"). Analysis was carried out in different geological subsoil's conditions, reflecting the results of engineering-geological survey. The results of the calculations at extreme hydrodynamic loading with the water level corresponding to Q_{100} confirmed in all cases risk of:

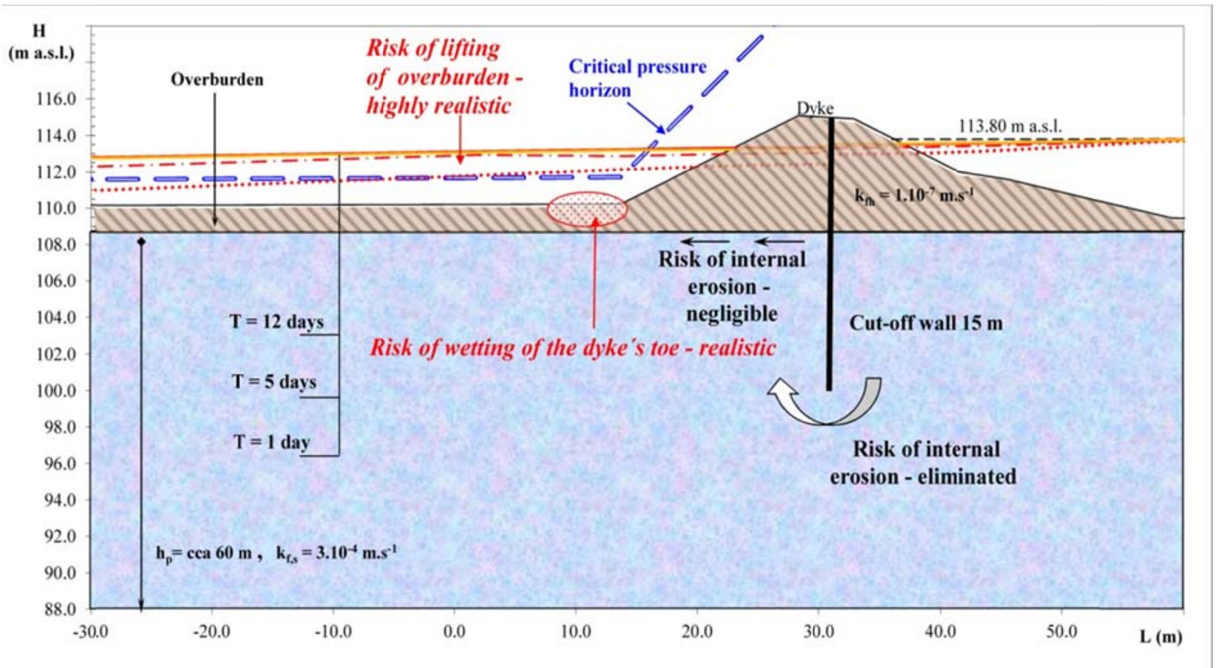
- waterlogging of the downstream toe and slope of the dike. When the ground surface is lower than about 110 m a.s.l. and there is the occurrence of thinner layers of overburden (approx. 1.5 m) can be expected a relatively rapid flooding of adjacent areas. For thicker overburden layers in the subsoil (about 2.5 m), water seeps to the ground surface after about 5 days of flood discharges. The result is adequate to the flood water level Q_{100}. For the flood in 2006, the flood level was at 112.71 m and the waterlogged areas were recorded only on sections km 23.1, 23.9, 24.5 - 24.9 and 26.5 - 27.5;
- breaking of the overburden caused by uplift was for considered boundary conditions proven in all analyzed alternatives. But based on the results it may be concluded that in the sections with overburden layer greater than 2.5 m, the risk is minimal. According to E-G survey such sections are at the site rare;
- exceeding of the critical hydraulic gradient was due to the proven presence of suffosive sands below overburden layers observed in the inlet areas below upstream toe of the dike, in the subsoil below dike's crest and near downstream toe. Besides the permeability of the subsoil, gradients in the subsoil below the crest of the dam are significantly affected by the position of the lower edge of an overburden layer (considering steady-state average groundwater level 106.5 m a.s.l. Effect of terrain elevation is negligible). Highly exceeded gradient with significant duration was therefore particularly recorded in the subsoil with overburden of thickness 1.5 m, what is an interface of overburden on elevation approx. 108.5. Upon the occurrence of impermeable overburden layers that go into greater depth than 107.5 m a.s.l. (overburden thickness 2.5 m) was not recorded exceeded gradient below the dam's crest. But such regions are in the investigated area rare (Fig. 5). Short-term exceeding of the critical gradient near downstream toe was also registered.

Outlined results confirmed the need of realization of appropriate remedial measures to minimize risk factors to acceptable levels.

4 DESIGN AND ASSESMENT OF REMEDIAL MEASURES

Numerical calculations confirm the possible presence of all the observed risk factor under extreme hydrodynamic stress Q_{100} in most of the regions, confirming the need for realization of appropriate remedial measures. Exceeded hydraulic gradient in the subsoil of flood protection dike with the presence of a suffosive sands indicate the possibility of internal suffosion respectively longitudinal contact erosion with the risk of washout of the fine-grained fraction from soil. The elimination of this negative phenomenon to acceptable level can be ensured by the cut-off wall (COW). In the present section, where the depth of neogene is more than 60 m, was designed suspended COW from the crest of the dike (to eliminate seepages through the body of dike with possible occurrence of permeable positions). Its length was considered in the two alternatives - 15 and 20 m below the crest of the dike. The effect of proposed measures (reduction of hydraulic gradients, changes of pressure conditions in the subsoil, waterlogging of the downstream slope of the dike) on the changes in filtration regime in the subsoil of dike under extreme hydrodynamic load with the duration of 12 days was the subject of optimization design. Detailed numerical analysis proved following:

- The planned COW with the length 15 m respectively 20 m reduces to the acceptable level hydraulic gradient in the subsoil and near the downstream toe of the dike at the expected geological composition of the subsoil, determined on the basis of available E-G survey. Attention has been focused not only on the immediate area under the overburden layers (Fig. 3 - zone 1, 2, 3) but also to the surrounding area of COW (Fig. 3 - zone 4, 5, 6). Exceeding of the critical hydraulic gradient was not confirmed under the COW as well as in its immediate vicinity.
- In relatively permeable environment with permeability 10^{-4} m.s^{-1} with prolonged (more than 1 day) duration of flood discharges is the effectiveness of a suspended COW in terms of reducing uplift and waterlogging of the downstream slope of dike very apparent. The trend in the development of uplifts is at an early stage reduced, but by growing the



duration of flood, the reduction of the uplift on overburden layers gradually decreases. The impact of the position of interface of overburden is weak, in given conditions reduction of uplift on overburden is about 0.2 m. Extension of the COW (as well as reduction) has due to the high permeability of subsoil in the final stage of extreme loading minimal impact on changes in pressure conditions. As evidence may be quoted the development of uplift on impervious overburden layers for assumptions of the geological composition of subsoil shown in Fig. 4. Uplift on the overburden in case of realization of COW with the length of 15 m at time T = 1, 5, 12 days after the beginning of the floods is presented with dotted, dashed and solid line (red color). Critical pressure horizon (double blue line) is exceeded even after the first day of flood, indicating the possibility of breaking of the overburden layers by uplift. In the case of realization the COW with the length of 20 m the pressure line at T = 12 days marked in the figure by dotted line (yellow color) nearly coincident with the development of the uplift for the COW length = 15 m. Assumed the above it can be concluded that the reduction of pressure conditions in the subsoil cannot be provided by suspended COW to acceptable level. Effective would be its binding to the impermeable subsoil, what is in given geological and morphological environment not realistic.

Detailed analysis indicates that for the reduction of gradients in the subsoil in the given E-G conditions it is sufficient implementation of the COW with a length of 15 m. The shorter COW is in this respect deficient, with the regard to the lack of reduction in filtration velocities below overburden layers. However, it should be also noted that design of suspended COW does not annul all risk factors threatening the stability of dike (Fig. 4). Its realization requires additional remedial action. Proven are stabilizing backfills on the toe of the downstream slope of the dike, extending to the so-called protection zone. The height of the stabilizing backfill is apparent from Fig. 5, where a line of varying thickness (blue-red) shows a terrain at the toe of the downstream slope. Thick (red zones) characterizes regions where terrain was flooded during flood in 2006. With the brown line is plotted position of the terrain considered in the calculations. With the continuous red line is plotted calculated position of pressure horizons

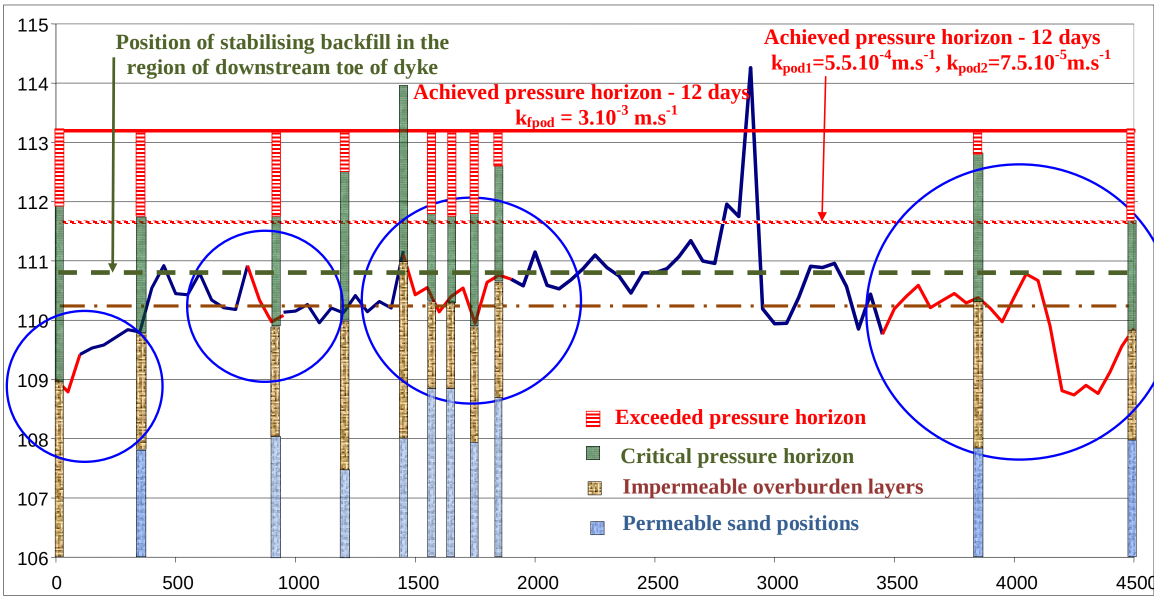


Fig.5 Breaking of the overburden caused by uplift

for the duration of flood discharges 12 days in the case of presence of sandy soils in the subsoil with permeability 3.10^{-4} m. s^{-1}. Column diagram indicates position of available exploration boreholes of E-G survey. In these with darker brown color are marked widths of overburden layers, with diagonally striped green hatch patterns to them assigned levels of

critical pressure horizon and with red horizontal hatch patterns exceeded critical pressure horizon. Its thickness is maximum 1.2 m, which means that for the elimination of the risk of breaking the overburden by uplift can effectively contribute stabilizing backfill with thickness approximately 60 cm. This represents an increase in the terrain level up to a height of 110.80 m a.s.l. near the downstream toe of dike, locally without increasing - according to the exceeding of the critical horizon (Fig. 5). The length of the stabilizing backfill should be specified on the base stability analysis of the downstream slope of the dike.
The proposed combined remedial measures reduce all risk factors threatening the stability of the downstream slope of the dike. While COW reduces the maximum gradients in the subsoil to acceptable levels, stabilizing backfill eliminates breaking of the overburden layers caused by uplift.

5 REVIEW OF THE FILTRATION STABILITY AFTER THE IMPLEMENTATION OF REMEDIAL MEASURES

Remediation of the left side flood protection dike of Váh River in km 23.040 - 27.075 is currently proceeding. It is realized the suspended cut-off wall from dike's crest to a depth of 15 m below the crest to elevation 100 m a.s.l. across the whole analyzed section of dike. Its minimum thickness is 30 cm and the estimated maximum permeability 1.10^{-7} $m.s^{-1}$ [7].
From the numerical calculation resulted that COW with a length of 15 m sufficiently reduces the maximum filtration velocities in the subsoil and near the downstream toe of the dike. Exceeding of the critical hydraulic gradient was not proved nor in the area under the COW nor in its immediate vicinity. This suggests that the implementation of COW eliminates the risk of internal suffosion in the subsoil and ensure its filtration stability. At the same time by its realization from the dike's crest are eliminated possible preferred seepage path through the body of the dike.
However, with the calculation confirmed risk factors are excessive uplifts at the overburden layers and flooding of adjacent areas. Although suspended COW extend seepage path and adequately seepage amounts, the risk of waterlogging of the downstream slope and toe is still real. Due to the nature of the adjacent protected area may be this fact accepted. It should be noted that the risk of breaking of the overburden layers by uplift remains, which entails a possibility of occurrence of outflows with the risk of washout of the fine-grained fraction from subsoil (due to confirmed presence of a suffosive sands under overburden layers). It may be concluded that in the sections with the overburden layers with thickness more than 2.5 m, the risk is minimal. Based on the knowledge gained from the E-G survey such dangerous sections on the site occur rarely.

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LANDSLIDES ON THE SLOPES OF WATER RESERVOIRS IN THE ENVIRONMENT OF CARPATHIAN FLYSCH

Miloslav Kopecký[1], Martin Ondrášik[2], Jana Frankovská[3] and Darina Antolová[4]

Abstract

The article evaluates a stability of landslides next to two major water reservoirs in Slovakia. The first is a landslide by the water reservoirs Liptovska Mara, which with its volume of water 360 mil. m^3 is the largest reservoir in Slovakia put into operation in 1975. On the right side of the dam body there is landslide displacing about 5mil. m^3, which strikes the reservoir with its toe. The second landslide is a landslide on the water reservoir Nová Bystrica, located over the reservoir water level, but its possible instability is threatening the water discharge object. Both landslides are located in the geological environment of Carpathian flysch and the article assesses a current stability of landslides, remediation measures and results of a long-term monitoring.

Keywords

landslides, monitoring, slope stability, water reservoir

1. INTRODUCTION

Water reservoirs (WR) Nová Bystrica and Liptovská Mara are located at northern part of Slovakia (fig. 1). Both reservoirs are build in the environment of Carpathian Flysch, while the reservoir Liptovská Mara in not folded, the water reservoir Nová Bystrica in folded flysch environment. On both water works the slopes above the reservoirs are affected by landslides, which are threatening their safety and operating. In the next text we present the nature of the landslides, hazard for the dams, the way of their remediation and proposal of their monitoring.

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Fig. 1 Localization of the water reservoirs in the map of Slovakia

2. LANDSLIDES ON THE DAM LIPTOVSKÁ MARA

On the right-side of the dam body of WR Liptovská Mara there are located two landslides. On the water-side bank it is Veľkomarský landslide and on the downstream face of the dam it is Malý Vlaštiansky landslide (fig. 2).

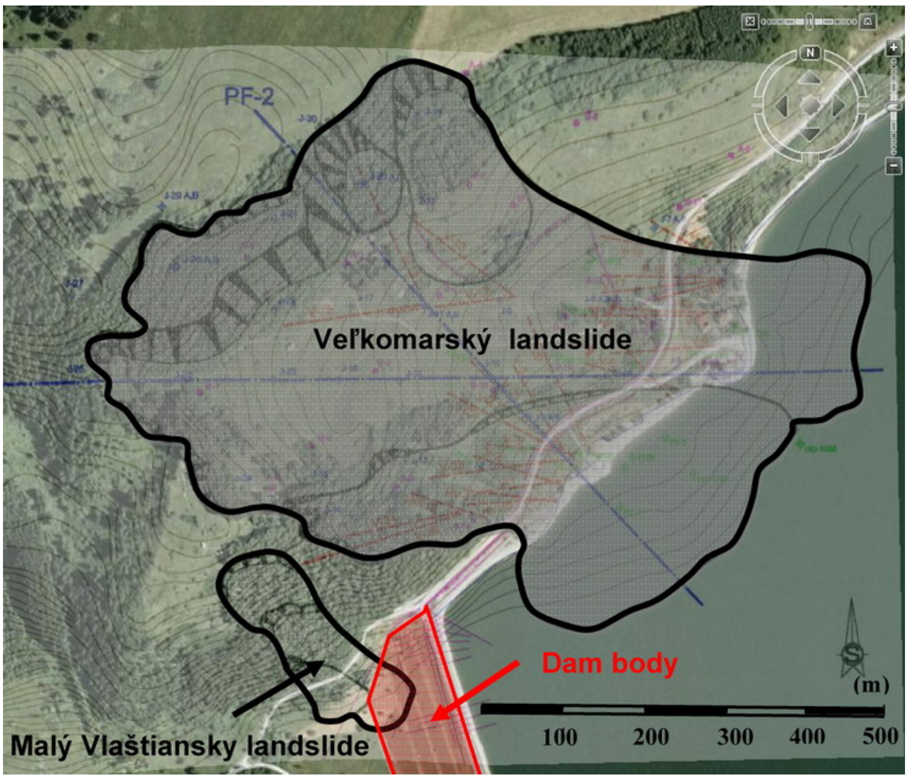

Fig.2 Localization of the dam profile between Veľkomarský and Malý Vlaštiansky landslides (satellite photo by Google Earth)

Both landslides were developed in the area consisting of unfolded Paleogene flysch strata. The area of Veľkomarský landslide extends with length 900 m, width 550 m and the maximum thickness of the sliding material in the accumulation zone of the landslide exceeds 30 m [1]. The whole landslide is composed of several partial landslides of various ages. The supposed volume of the sliding mass exceeds 5 mil. m³ [2].

2.1 Remediation measure at Veľkomarský landslide

The stability calculation of Veľkomarský landslide performed prior to the WR construction had appointed to low stability of the slope. The calculation result meant concern about significant instability of the landslide after finishing the dam construction due to water buoyancy acting on the accumulation zone of the landslide.
To improve the Veľkomarský landslide stability after finishing the construction of the WR the following remediation measures were taken (fig. 4):

- building of stabilization anti-abrasion embankments consisting of gravel and sand on the landslide toe (in 1974-75) – the thickness of the embankments was 7 m (volume 700 000 m³),
- realization of horizontal drainage boreholes (HDB), (4 stages in 1974-77),
- creating a system of surface drainage gutters (in 1976-1978).

2.2 Recent activities on the landslides

Although on the Veľkomarský landslide there is network of geodetic points, the results of the measurements of position changes of the fixed observation points on the landslide cannot be used for qualified assessment of the activity of the landslide (movement of "solid" points). For overview of activities of Veľkomarský landslide therefore changes of altitude of geodetic points are used only.
In late March 2006, the sudden warming and rapid melting of very thick snow cover led to infiltration of water from the melted snow into groundwater what resulted in increase of groundwater table levels in the sliding slope to maximum ever observed during the entire observation period. These high groundwater levels were synchronous with changes of elevation of the survey points. A significant decrease was recorded on points from B-1 to B-6, which are located in the head of the landslide area (from 6,9 to 12,2 mm – 9,8 mm in average! fig. 3), what indirectly pointed to a partial activation of the landslide area.

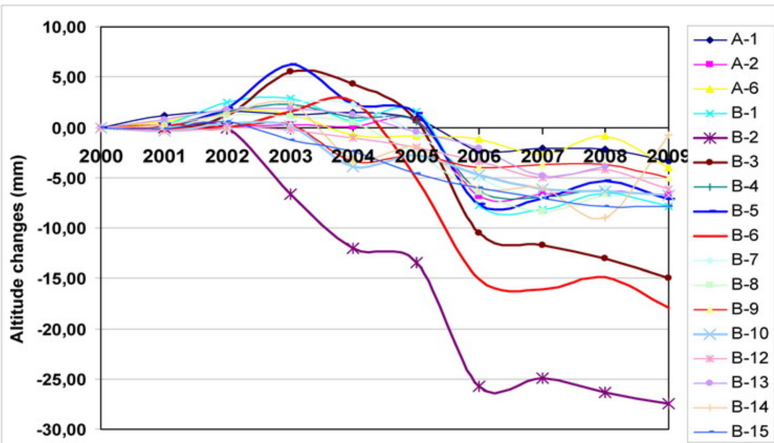

Fig. 3 Cumulative curves of altitude changes (mm) of geodetic observation points during years 2000-2009

2.3 Status of the current monitoring network on the landslides

The monitoring network (fig. 4) of both landslides was built 39 years ago. On the Veľkomarský landslide in 1974-1975 there were built sites for observation of ground water levels (observation wells - 30 pcs). As remedial measures also 28 horizontal drainage boreholes (HDB) with the total length of 3 800 m were built in 4 stages.

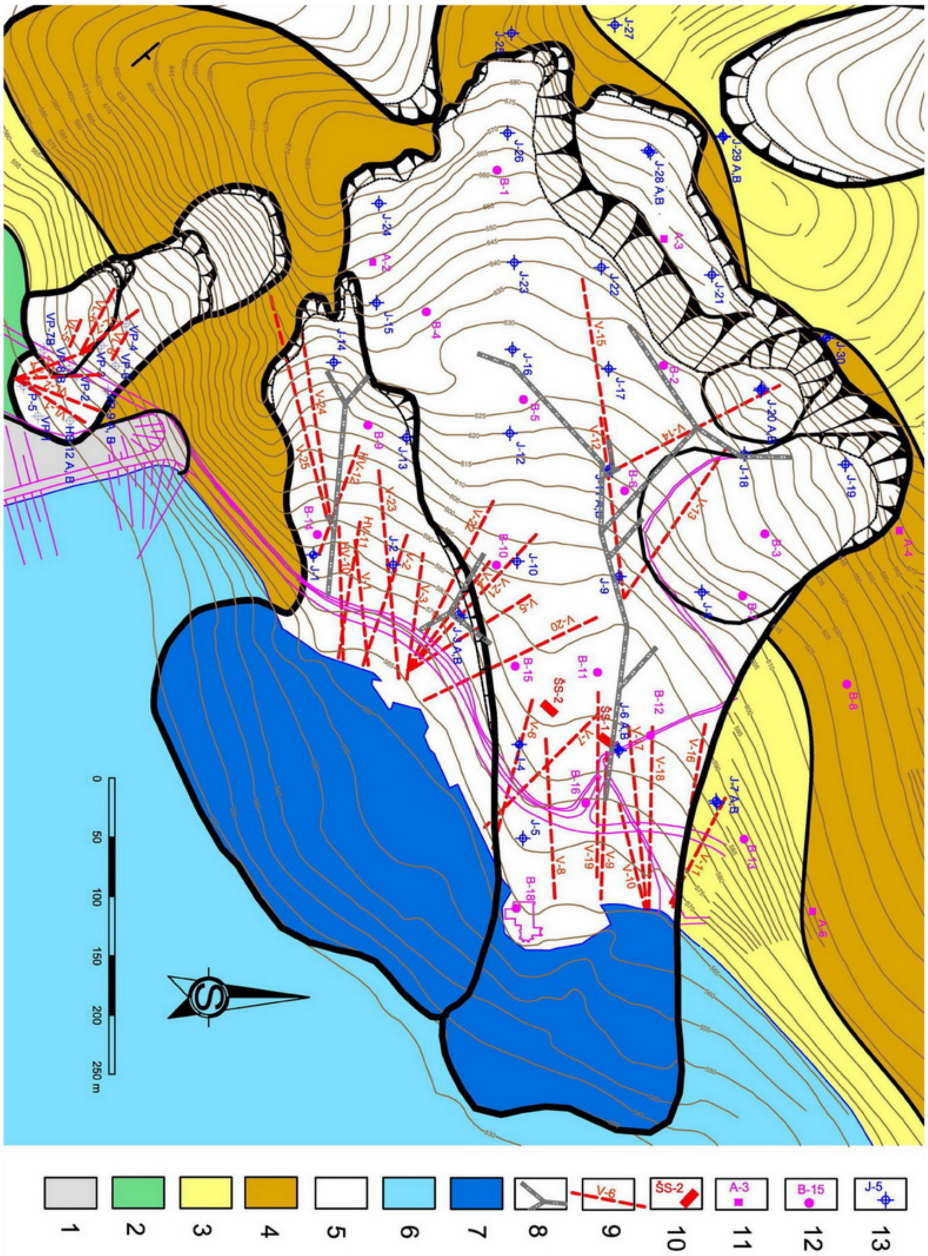

Fig.4 Scheme of existing monitoring points and remediation measures on Veľkomarský and Malý Vlaštiansky landslides [3]

1 - earth dam, 2 – Vah river fluvial sediments, 3 – deluvial sediments, 4 - Paleogene strata, 5 - landslide bodies, 6 - water surface, 7- part of the Veľkomarský landslide under the water of the reservoir, 8 - surface drainage gutters, 9 - horizontal drainage boreholes, 10 - gravel walls, 11 - fixed geodetic points, 12 - geodetic observation points, 13 – observation wells

The groundwater level and discharge rate of HDB are monitored once per 14 days. In the period 2003-2010 sixteen observation wells were equipped with automatic piezometers. Monitoring is carried out in order to assess the effectiveness of the remediation measures. For tracking of movements of the landslide a network of geodetic control points, consisting of 6 fixed points and 17 observation points was built in the landslide area. The geodetic measurements are performed once a year.

With regard to the long term monitoring of the HDB discharge, it is possible to observe decrease of the total volume of the water being removed from the landslide (fig.5).

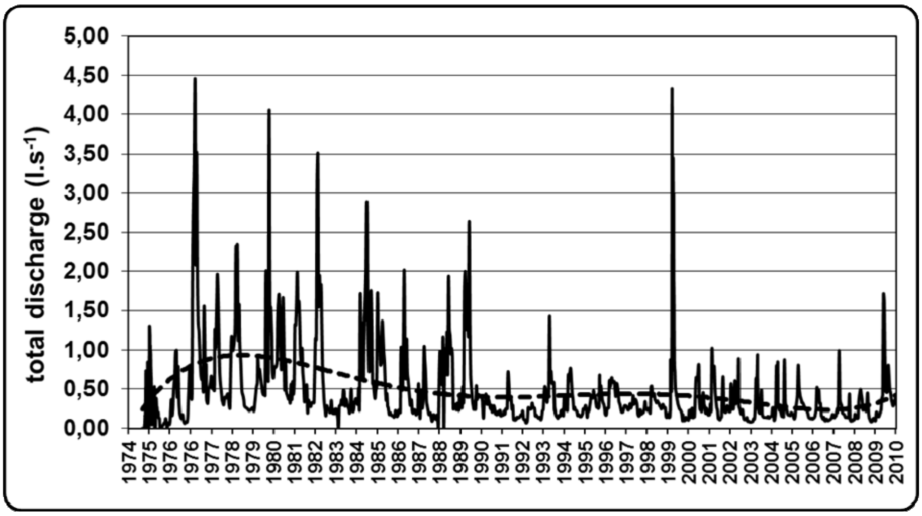

Fig. 5 Downward trend of the total discharge from all HDB on Veľkomarský landslide

However, this overall groundwater level (GWL) decline does not have to play a role in reduction of the local slope stability, unless due to decrease of the water discharge from HDB the groundwater level is not increased in the nearby observation wells. The negative impact of the discharge decline in HDB from V-12 to V-15 located in the landslide head area is quite obvious. In this area (fig.4), in observation wells J-16, J-17 (fig. 6), J-18, J-11B, J-11A the groundwater level is increasing for long time period, in the case of J-11A the groundwater freely flows out of the well as from artesian well. There was made inspection of the horizontal drainage boreholes with use of camera (fig.6). The largest throughput was observed only up to 30 m. In most cases the camera got only a few meters away from the HDB mouth. The inspection results have pointed out that the broken HDBs should be either cleaned or replaced by new HDBs.

Fig. 6 State of the HDB casting after 38 years in operation at the camera inspection

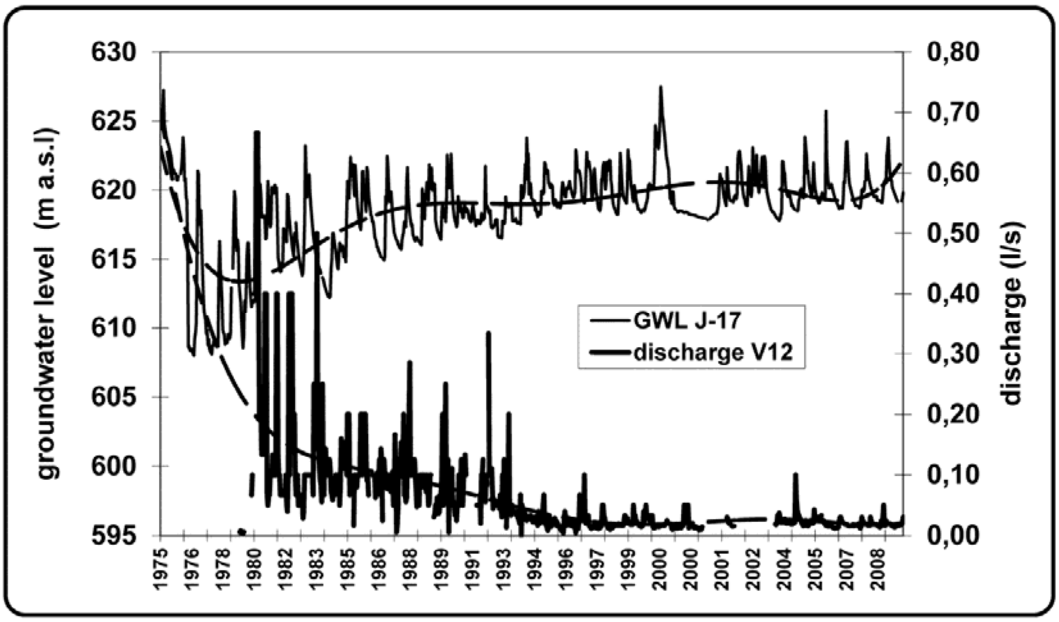

Fig. 7 Decrease of functionality of HDB V-12 and GWL rise in observation well J-17

2.4 Proposal of design and maintenance of the monitoring network of the Veľkomarský landslide

The proposal of the complex monitoring of the landslides on the right-side of the dam body of WR Liptovská Mara was designed such a way that its result was:

- determination of the landslide recent activity,
- forecasting the future development of the stability,
- setting the critical values for need of realization additional remediation measures.

The proposed measures were split in to 3 stages – tab. 1. Emphasis is placed especially on the building of inclinometer boreholes and reconstruction of network of geodetic points, because stability (or better dynamics) of any slope can be judged the best way by combination of geodetic measurements of the surface movement with inclinometric measurements of movement under the surface. It will be also necessary to clean the existing HDB. If such measures prove ineffective, it will be necessary to proceed with a construction of new HDB.

Tab.1 Proposed measures on the Veľkomarský landslide in 3. stages

Proposed measures	***Purpose and output of the measures***
1. stage - specifying and obtaining additional information on landslide area	
a) Geodetic survey of existing monitoring elements and important elements of the landslide areas	Creating a representative model of slope deformations + positioning of the network elements
b) Geophysical measurements – 4 profiles with total length 2 380 m	Determination of surface and depth extent of the landslide area + precising the location of new inclinometers
2. stage – building of new elements of the monitoring network	

a) Building inclinometer boreholes – total 7 pc – 200 m	Observation of movements in depths of the massif. Determination of residual shear strength parameters of soils from samples taken during the drilling.
b) building new piezometers - total 4 pc – 100 m	Measurement of GWL in the vicinity of inclinometer boreholes + installation of automatic piezometers.d
c) new geodetic points – 5 pc	Measurement of surface movements of landslide the area – movement of blocks in the upper part of the landslide area.
3. stage – reconstruction of the current monitoring network elements	
a) geodetic points – rebuilding all 22 pcs	Rebuild geodetic points in their original places and adjusting their surroundings for measurements by methods of Very accurate leveling and GPS.
b) piezometer - reconstruction of approximately 11 pcs – 265 m	Reconstruction of the broken wells
c) horizontal drainage boreholes – about 2 000 m	Cleaning those HDB which show long-term decline in discharge rate and those in which there is a rise of GWL (fig. 5) In the case of inefficient cleaning new HDB will be necessary to build.

Even during the renovation and reconstruction of new elements of the monitoring network it is necessary to continue with monitoring measurements on both landslides. However, gradually it is necessary to upgrade the system to automatic data acquisition in order to determine the threshold conditions for the possible activation of the landslides. Only then it will be possible to implement necessary measures in time and to provide a safe operation of the WR Liptovská Mara.

3. LANDSLIDES ON THE DAM NOVÁ BYSTRICA

The reservoir Nová Bystrica is located in the northern part of Slovakia, about 6 km southward of the border with Poland (fig. 1). It is used to supply the population with drinking water and was put into operation in 1989 [4].

Even before the construction of the water reservoir Nová Bystrica, during the implementation of a detailed engineering geological survey [5], the presence of slope deformations was observed at the slopes on left side from the designed dam (fig. 8), which are located above the maximum shoreline. Two slope deformations of varying activity are allocated on the slope. In the area between the reservoir shoreline and the road there are active landslides with overall area about 120 x 60 m. The landslides were probably caused by improper intervention during construction the road. Active landslides are bound only to the slope debris containing coarse fragments from 40 to 80 % and the shear surfaces were detected by the inclinometric measurements at depths from 3,5 to 5,0 m. In the higher part of the slope (in the range of about 630-690 m above sea level) there is a block deformation (fig.8). Based on initial investigation works [6] its dimensions were estimated to approximately 100 x 140 m. According to our latest knowledge, this could be up to 220 x 140 m (dotted line in fig.8). Block deformation is result of gravitational movement of relatively rigid sandstones over the surface of plastic claystone. During the movement from the main scarp zone the sandstones were disintegrated to individual blocks. The maximum depth of the shear surfaces was detected by inclinometric measurements at a depth of 34.0 m below the surface (fig.11).

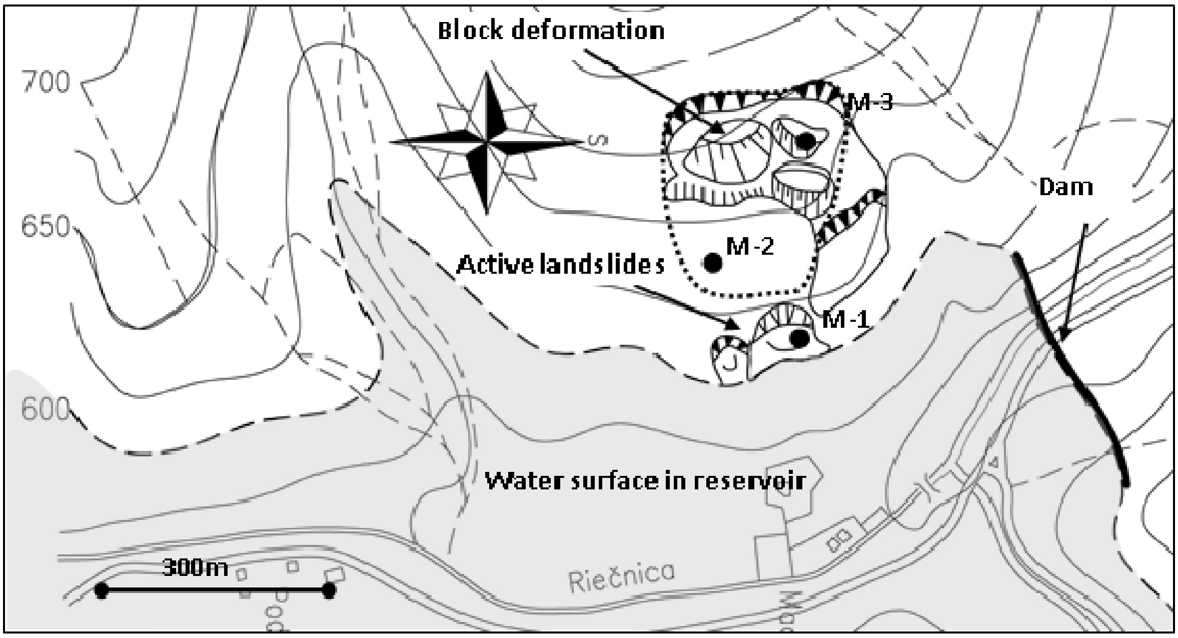

Fig. 8 Scheme of the landslides at the slope located on the dam left-hand side

3.1 Landslide activity - monitoring and its results

In 1993, exploratory boreholes [6] were equipped for the combined observations of groundwater levels (GWL) and movements on the shear surfaces (7 pcs.). Four of them, which were placed in the active landslides, become inoperative since 2001. In 2002 there was made 6 new inclinometric boreholes (without possibility of measuring GWL) in the active landslide on the road. Furthermore, in 1995 there were built 4 fans (each with 3 horizontal boreholes) of drainage boreholes, and their discharge is monitored. These drainage wells are the only remediation measure performed on the landslide up to now.

At present, GWL can be measured in 3 wells and movements on the shear surfaces in 7 inclinometric boreholes (of which 5 are in active landslide and 2 in the block deformation).

The analysis of measurements of movements in the inclinometric boreholes are the most important. Due to the nature and depth of the measured movements it was necessary to assessed independently the active landslides with the road and the block deformation.

<u>Active (little) landslides</u>

In the landslide area with the road it has been confirmed that the activity of the movement still persist. The maximum deformation observed from 1993 to 2006 is 300 mm (sum of deformations from two inclinometers – the second one replacing the damaged first one). The movement is probably not a continuous and uniform, it occurs only under extreme climatic conditions (rainfall and snow melting). The depth of the shear surface of the active landslide on the road was found 3,0 -5,5 m below the surface (fig.9).

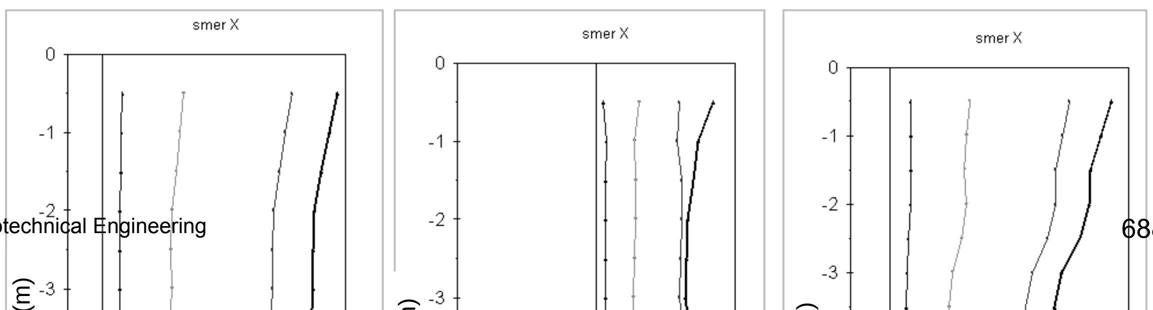

Fig. 9 Movements in the inclinometer located in the active landslide on the road

Based on the analysis of the inclinometric measurements it was found in 2 inclinometric boreholes located outside the presumed area of the landslide active in 1993 that since 1999 there was gradual expansion of the landslide and significant distortion of the road (fig.10).

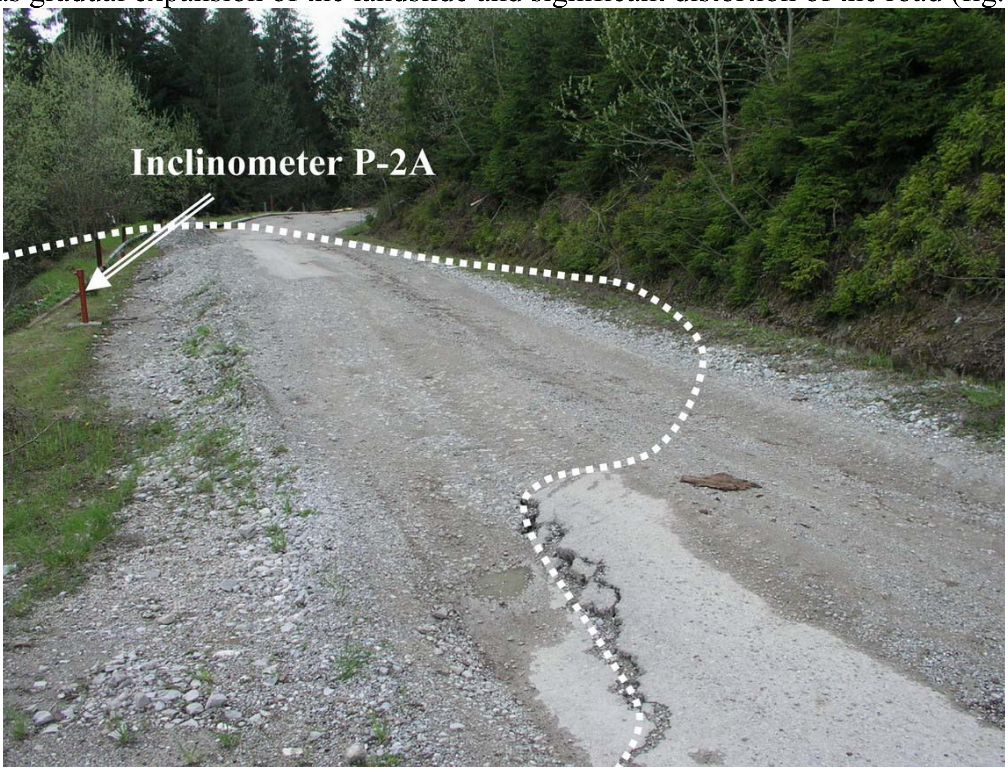

Fig. 10 Main scarp area of the active landslide in the body of the road

<u>Block (larger) deformation</u>

The block deformation, which is situated in the slope above the road, there is a gravitational movement of relatively rigid sandstones over the surface of a plastic claystone. Inclinometric measurements under the main scarp zone of the block deformation (borehole M-3) confirmed the existence of a relatively thick shear zone (about 7 meters), along which there is a movement of the block of sandstone up to 24 meters thick (fig.11).

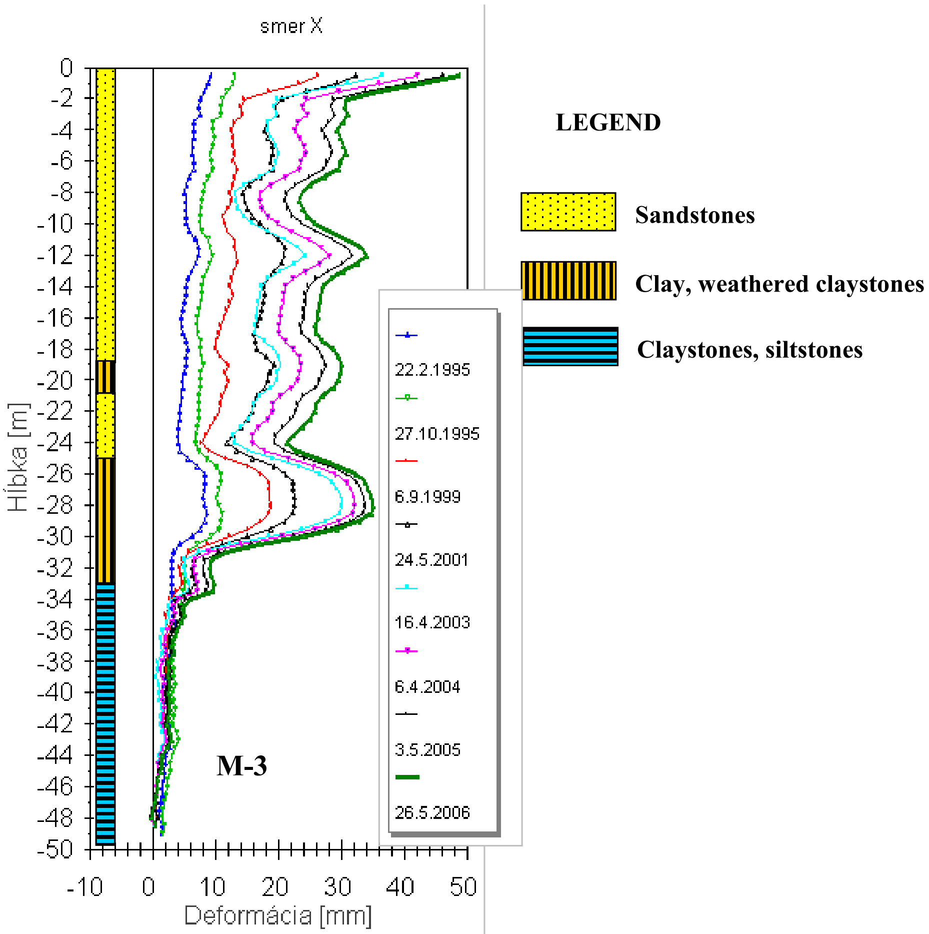

Fig. 11 Inclinometric measurements in the borehole M-3 located in the block deformation

Total movement of 44 mm during 13 years was detected. However it is important that the movement at a depth of about 28.0 meters relatively uniform is up to now (fig.12).

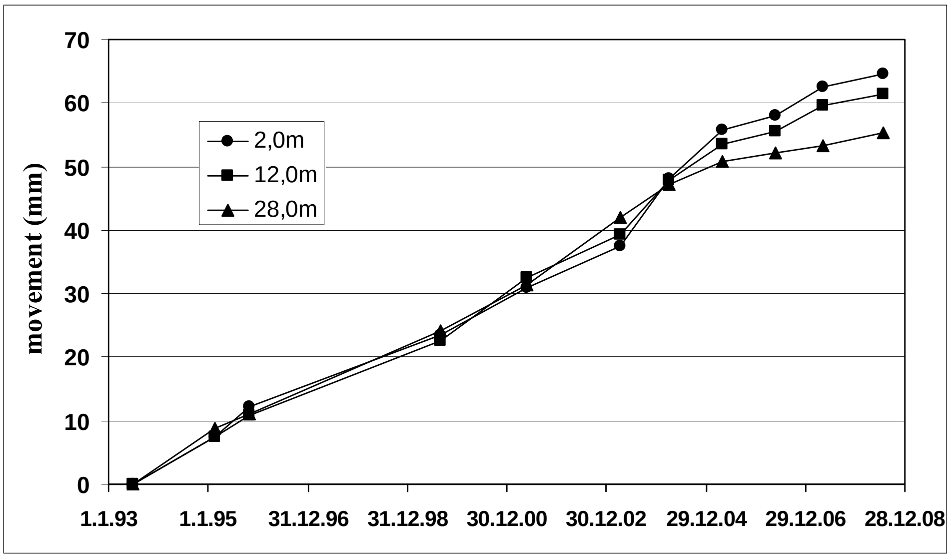

Fig.12 The time course of movements in different depths in the borehole M-3

Borehole M-2 (fig.8) was situated on morphological platform about 60 meters above the road, and it was assumed [6], that the platform was not the result of the slope movement. However, subsequently realized inclinometric measurements showed quite a clear movement on the contact between sandstone and claystone (13.0 meters below the surface). The total resultant movement in this depth is almost 63 mm for 13 years. These findings led us to an opinion that the areal and depth extent of the block deformation is greater than it was originally anticipated.

3.2 Forecast of development of the slope deformations and their influence on the operation of the water reservoir

Development of the existing slope deformations and their potential impact on the operation of the water reservoir can be forecasted based on the measurements of the monitoring network (especially in inclinometric boreholes), and also calculations of slope stability [7]. Results of the calculations of the slope stability showed that the most important factor affecting the stability of the slope deformation is the groundwater level in the slope. This fact will have to be taken into account when designing remediation and stabilization measures.

Active (small) landslides

It is obvious that, if no remediation works will be made on the active landslide area with the road [7], than in some time there will be an absolute destruction of the slope and sliding of the deluvial sediments to its lower part and partially into the water reservoir. About 55 000 m^3 of debris can get into the movement. However we assume, that in this process there is no threat to the inflow waterworks facility remote about 150 m from the accumulation of the active landslides. On the other hand, removal of rock masses from this area can cause acceleration of movements of the block deformation situated above the road.

Block (larger) deformation

Basing on the inclinometric measurements a link between deformations in an area of borehole M-2 with the deformations around the borehole M-3 cannot be ruled out. The knowledge of the fact whether there is accelerated motion or movement is steady is crucial to the prognosis of further development of the movements (fig. 12). However, if the inclinometric

measurements are carried out only once a year, it is not possible to determine the nature of the movement reliably.
If there are demonstrably accelerated movements in the area of the block deformation, it will be necessary to proceed to remedial or other measures, because at certain acceleration it will not be possible to stop the given movement. The subsequent movement of the rocks would likely jeopardize the operation of the water reservoir.

3.3 Recommendation for the following monitorings

Assessment of the stability of the slope on left side from the dam Nová Bystrica and forecast of its future development with respect to the operation of the water reservoir were based only on pre-existing knowledge, where the inclinometric measurements have the critical role. Assumptions obtained by calculations and analyses corresponds the accuracy of all existing data.
For clarification of existing knowledge (especially in the area of the block deformation) the recommended further works are summarized in Table. 1.

Tab.2 Recommended remediation works on the Nová Bystrica landslide

RECOMANDED WORKS	PURPOS AND OUTPUT OF THE WORKS
Geodetic survey of the terrain morphology	Creating a representative model of slope deformations (active landslides and block fields)
Mounting surface geodetic points and measuring their movement	Understanding the relative speed and direction of the movement of blocks on surface (forecast their future development)
Inclinometric measurements (2 times a year) Construction of 2 pieces of new inclinometer boreholes in the block deformation Restore functionality of inclinometers P-5 and M-1	Finding the depth and size of the deformation in the rock environment – prediction of further development of the movements with identification of critical values of their velocities
Groundwater level (3 new piezometers with continuous measurement of GWL), discharge of horizontal drainage boreholes	Optimization of boundary conditions for stability calculations and determination of the critical levels of GWL
Geophysical measurements	Determination of areal and depth extent of the block deformation + specification of location of new inclinometers

CONCLUSION

Based on the monitoring we assume that the landslide Liptovská Mara is temporarily calming; the movement has slow to creep character. The only activation was recorded in the spring 2006, when there was a downturn in the crown zone of the landslides. The stability of the

landslide slope described in this paper is primarily a function of groundwater levels and water level in the tank.
On the WR Nová Bystrica the stability calculations demonstrated instability of the shallower landslides threatening the road, what was subsequently proven by measurements of the movements in boreholes equipped with inclinometer. The movement was more than 30 cm in 15 years. More dangerous for the operation of the water reservoir may be movement in the block deformation above the shallow landslides. The movement of the blocks occurs along shear surfaces in greater depth (34 m and 12 m bellow the surface). The movement is uniform and is about 3-5 mm per year. Acceleration of this movement could have disastrous effects on the safe operation of the water reservoir.
On the WR Liptovská Mara, the only representative measurements proving the actual movement of the landslide are measurements of elevation changes on geodetic points. Positional changes of geodetic points are irrelevant because unstable are even the fixed points. It is necessary to rebuild a network of the measured geodetic points. It is also necessary to build several boreholes to monitor movement within the soil mass (inclinometer boreholes). Only then we can determine the critical groundwater levels and predict the actual development of the landslide slope. With respect to poor condition of the monitoring elements after 39 years it is necessary to build new or refurbished the old monitoring elements.
In turn, on the WR Nová Bystrica there are quite good data on the movement of the landslide slopes, however less satisfactory there are data on groundwater levels and particularly there is a lack of data on the extent (area and depth) of the block deformation.
The output of the article is therefore not only information about the current stability of landslide slopes, but especially the proposal for their continuing monitoring, enabling reliable prediction of their further development and hence it is also the proposal of measures ensuring long-term trouble-free operation of the referred water reservoir.

Acknowledgements
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USING GPR FOR DETECTING ANOMALIES IN EMBANKMENTS

D. Marčić[1], M. Bačić[2] and L. Librić[3]

Abstract

Application of Ground Penetrating Radar (GPR) for investigation of anomalies in embankment is discussed in this paper. Short overview of factors which cause embankment instability is given (such as soil parameters of embankment and foundation soil, water effect, man-made structures and biological factors), as well as GPR theoretical background. GPR was proven as reliable, non-invasive, tool for detection of embankment voids, surface cracks, shallow and deep cavities, inhomogeneities etc. Some successful case studies of GPR application on embankments are also presented.

Keywords

antenna frequency, dielectric constant, embankment, Ground Penetrating Radar, resolution

1 INTRODUCTION

ICOLD (International Commission on Large Dams) [1] divides dams into embankments, concrete dams and masonry dams where approximately 70% of total number are embankments. Embankment represents a man-made 'ridge of earth or stone that carries a road or railway or confines a waterway' [2]. It is constructed by using earth materials such as clays, silt, sand, gravel, or crushed rock. Type of material which will be used for construction is primarily defined by the purpose which embankment needs to fulfil. Two types of embankments can be constructed. First one is for transportation purposes where they are constructed in order to 'follow' designed road/railway level. Main problems associated with this type of embankment is its excessively settlement and its instability. Therefore, for this kind of application compressibility and strength parameters of embankment fill and

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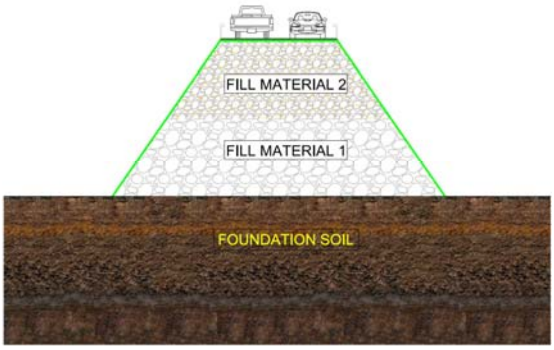
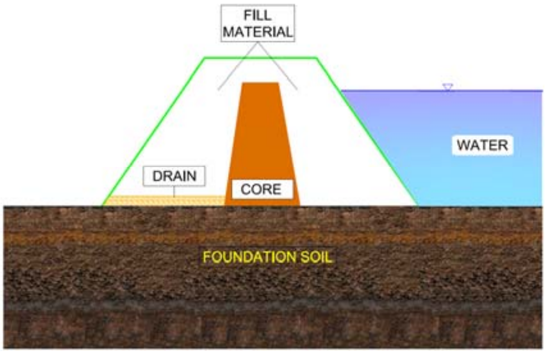
foundation soil are of importance. Settlement rate must be controlled with attention, especially for railway embankments, due to fact that rail tracks are highly sensitive to differential settlements, which are combined result of traffic loading and compressibility characteristics of embankment fill and foundation soil. Second type of embankment, analysed in detail in this paper, is embankment for water management purposes, which are mostly built as flood mitigation measures [3]. Construction materials, in this application, must provide stability and impermeabilization of the embankment. Typical cross section of transportation embankment is shown in Figure 1a, while Figure 1b shows typical cross section of embankment constructed for water management purposes. As it can be seen, embankment on Figure 1b is a complex structure divided in zones, where every zone has different characteristics and functionality [4]. Embankment can also be constructed to fulfil both water management and transportation needs at same time.



(a)



(b)

Fig. 1 Typical cross section of embankment constructed for transportation purpose (a) and of embankment constructed for water management purpose (b)

During their lifetime, embankments are subjected to series of factors which can cause its instability and degradation, leading to fatalities and immense material damages. Therefore, periodic maintenance and upgrades are necessity. According to Niederleithinger et al. [5] there are four groups of factors which can cause instability of embankment system. They include:

Category A – soil parameters of embankment and foundation soil
Category B – water effect
Category C – man-made structures
Category D – biological factors

Category A implies geotechnical parameters of embankment fill and foundation soil such as strength parameters, density, granulation, permeability etc. Factors associated with water activities, category B, can lead to instabilities due to processes of percolation (very slow water leakage through embankment), fluation (fast leakage of water), overflowing (due to water overflow the crest of embankment) and erosion (quick flow above small portion of embankment) [3]. Water effect is the most frequent cause of embankment instability. This fact is confirmed on example of 220 embankment failures, occurred in USA in period from 1850 to 1950, where 40% of all failures were consequence of problems linked with seepage, piping or internal erosion phenomena [6]. Additionally, construction of buildings and roads on crest of embankment, or presence of pipes inside body of embankment increase instability potential and those factors are part of category C. Factors in category D include animal digging and plant roots which have negative effect on embankment stability. Mention factors are shown in Figure 2a.

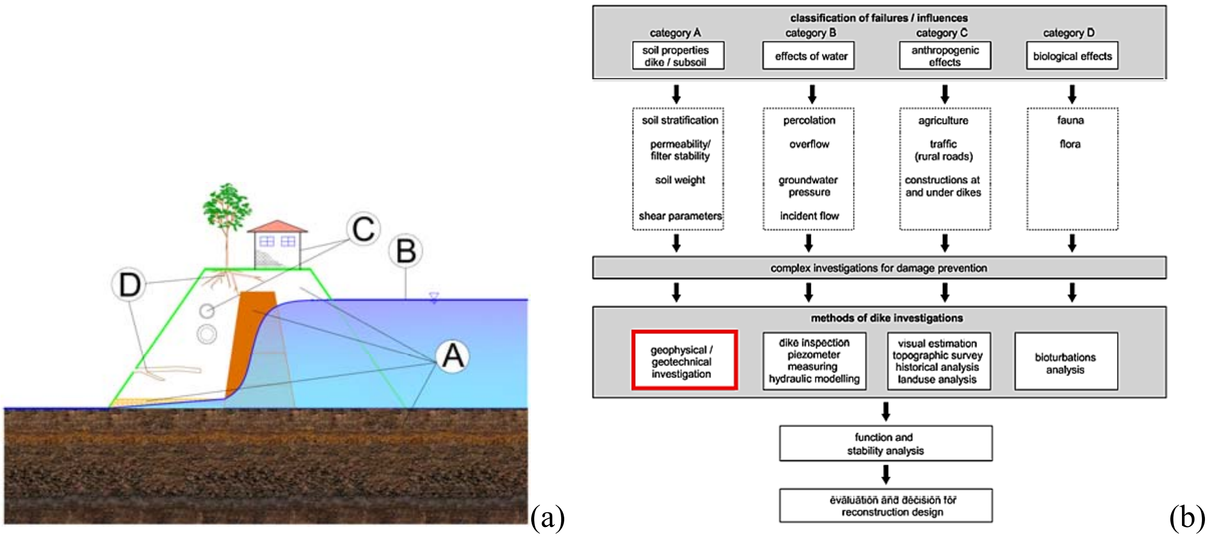


Fig. 2 Scheme of factors causing embankment instabilities (a) and classification of embankment failures/influences and methods of embankment investigations [5] (b)

In order to detect anomalies and to prevent, by timely interventions, any potential unfavourable situation, it is necessary to conduct investigation works which will give answer about whether there are anomalies and to what degree do they affect embankment stability. Along with factors which cause instabilities, Niederleithinger et al. [5] presented methods of embankment investigations, which are shown in Figure 1b. Investigation works provide basis for decision making process on necessary remediation measures. Traditionally, the most widely used tool for investigation of embankments in order to detect surface cracks, shallow and deep cavities and inhomogeneities was visual detection. The problem with visual inspection is that hidden effects can be overlooked and that visual inspection is very susceptible to subjective assessment. Besides visual inspection, an invasive method of borehole drilling was used for detection of anomalies as well as for defining geotechnical characteristics of embankment construction material. Borehole drilling is expensive and although it gives unique information about geotechnical parameters, those information are obtained in discrete area (point information). Because of latter, drilling works are proven as ineffective tool for detection of extent and scale of embankment anomalies. All mentioned led to development and application of non-invasive geophysical methods for embankment condition monitoring. In last few decades an application of geophysical methods has expanded from original application in exploration of natural resources to use in civil engineering, in prevention of natural hazards, for environmental protection, in mining, in archaeology etc. However, there is still a lack of information on selection of suitable methods and measurement parameters when using geophysical methods for embankment investigation [7]. Geophysical methods offer considerable savings of time and financial resources and although most of them require application of complex methodology and advanced mathematics for interpretation of results, many information can be estimated on-site based on collected data [8]. Further interpretation of geophysical data requires knowledge and experience, because collected data set may not indicate to specific condition in embankment (not unique solution) [9]. However, geophysical methods can not directly (without correlation) provide information about strength, compressibility and permeability parameters of materials, so borehole investigation works cannot be excluded, but their optimization can be made based on results of geophysical survey. Non-invasive geophysical methods can be divided to geoelectric, seismic and electromagnetic methods. Most frequently used among electromagnetic methods is Ground Penetrating Radar (GPR). It is mostly applicable for detection of man-made objects [10], but it is also proven as reliable in identification of embankment structure and layer boundaries, as well as for detection of groundwater level. GPR survey includes rapid site coverage, where on-site interpretation is possible due to instant graphic display of collected data. All mentioned, along with low survey cost relative to other methods, recommend GPR as a first order survey strategy for assessing embankments [11]. However other authors, such as Morris [7], consider geoelectric methods as most reliable for assessment of embankment condition, but indicate GPT methods as suitable for the same task.

2 GROUND PENETRATING RADAR (GPR)

As mentioned earlier, in last few decades GPR has found its application in civil engineering (underground engineering, investigation of bridge decks, asphalt pavements, concrete pavements, concrete structures etc.). This rapid development and extensive usage led to preparation of number of guidelines and standards regarding GPR application (for example *ASTM D6432-11-Standard Guide for Using the Surface Ground Penetrating Radar Method for Subsurface Investigation* [12]). GPR method is based on emission of high frequency electromagnetic pulses in subsurface (or structure) by using suitable antennas. In subsurface, waves can be attenuated, reflected or refracted. After reflection, which is produced at boundary between two materials with different dielectric characteristics, wave returns to surface where it is received by second antenna, Figure 3. This kind of system, when different antennas emit and receive wave, is called bistatic system. Opposite system is called monostatic, in which same antenna is used for emission and receiving of wave. Depending on antenna position during GPR survey, GPR systems are classified as air-coupled or ground-coupled system.

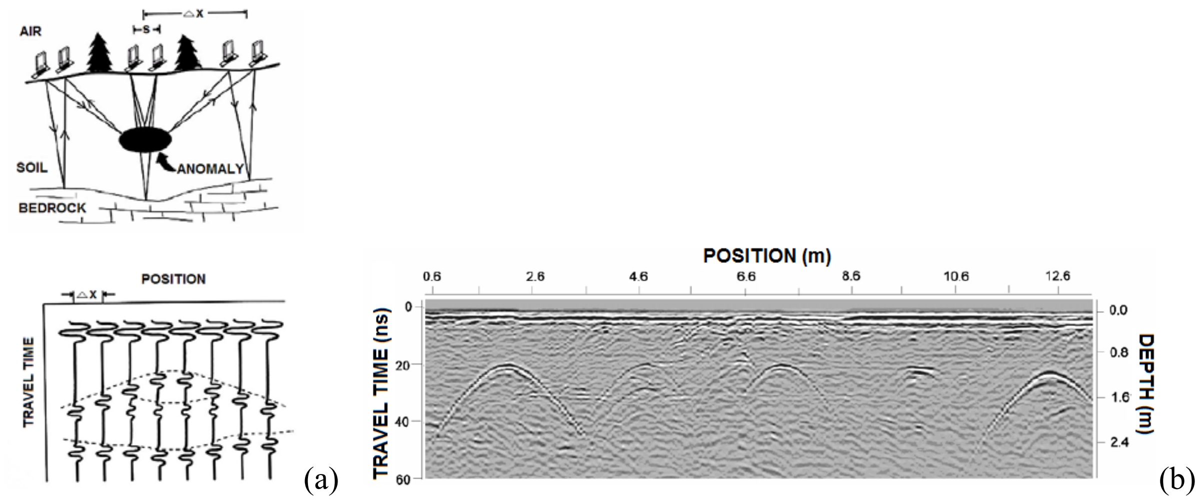


Fig. 3 Principle of GPR investigation (a) and typical GPR image (b) [13]

In order to evaluate geophysical method as useful, a change in certain physical characteristic must exist. In case of GPR survey, that physical characteristic is relative permittivity. The relative permittivity of a material for a frequency of zero is known as its dielectric constant. Higher dielectric contrast between two materials will result in stronger reflection in GPR image [14]. Example for this can be seen on Figure 3b, where series of hyperboles in GPR image represent position of underground pipes. Dielectric constant influences velocities of the electromagnetic waves and because of that, velocity analysis conducted in post-processing phase can be used for dielectric constant calibration. Typical electromagnetic properties of some materials, among which is dielectric constant, are given by Morey [15] and are presented in Table 1.

Tab. 1 Typical electromagnetic properties of some materials [15]

Property / Material	Dielectric Constant, ε_r	Electrical Conducitivity (mS/m)	Velocity (m/s), v	Attenuation (dB/m), A
Air	1	0	0.30	0
Fresh Water	81	0.05	0.033	0.1
Sea Water	80	$3 \cdot 10^4$	0.015	10^3
Dry Sand	3-5	0.01	0.15	0.01
Saturated Sand	20-30	0.1-1.0	0.06	0.03-0.3
Silts	5-30	1-100	0.07	1-100
Clays	5-40	2-1000	0.06	1-300
Limestone	4-8	0.5-2	1.12	0.4-1
Granite	4-6	0.01-1	0.13	0.01-1
Bituminous Concrete	3-6	0.5-1.5	0.12	0.05-0.5
Concrete (cured)	6-11	1-3	10	0.5-1.5

Frequency of GPR determines two crucial survey parameters – investigation depth and resolution. According to Annan [13], who gives a comprehensive overview on GPR theory and application areas, GPR plateau usually occurs in the 1 MHz to 1000 MHz frequency range. When higher frequencies are used, lower investigation depth can be achieved, but with higher resolution. Using lower frequencies, subsurface investigation will result in lower resolution, but larger depths can be investigated. There are two types of resolution when conducting GPR survey, a vertical and horizontal one. Vertical resolution is, simply put, smallest distance in vertical direction at which two phenomena can be apart in order to see and distinguish them as separate phenomena, while horizontal resolution is the minimum horizontal distance between two phenomena at the same depth before the radar merges them out into one single event [16]. High conductivity material, such as clays, may absorb and attenuate signals leading to significant decrease in penetration depth. On the other hand, in low conductivity materials such as sand, penetration depth can reach up to tens of meters. Dielectric constant of granular sediments is dominantly governed by water content in that particular sediment. Higher water content usually means higher dielectric constant (due to fact that dielectric constant of water has high value of 81). Van Overmeeren et al. [17] presented a relation between soil water content, dielectric constant and propagation velocity of radar waves in unsaturated sands, shown in Figure 4. Salinity of water does not have large influence on dielectric permittivity, but it strongly influences conductivity and attenuation of electromagnetic waves [11].

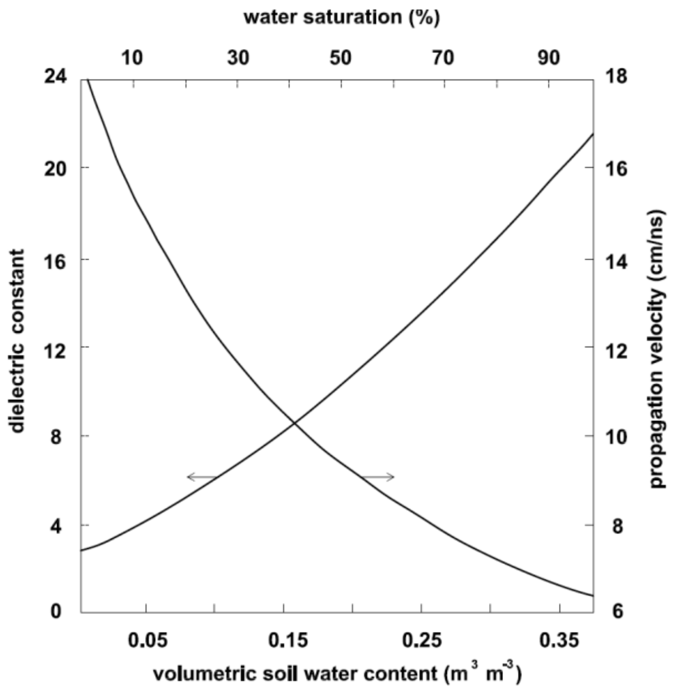


Fig. 4 A relation between soil water content, dielectric constant and propagation velocity of radar waves in unsaturated sands [17]

System for GPR survey is very simple and easy to operate with. It is composed of three parts: an antenna, a control unit (which is 'responsible' for generation of the radar signal and for detection of received signals as a function of time [4]), and the processing system (a laptop), Figure 5.

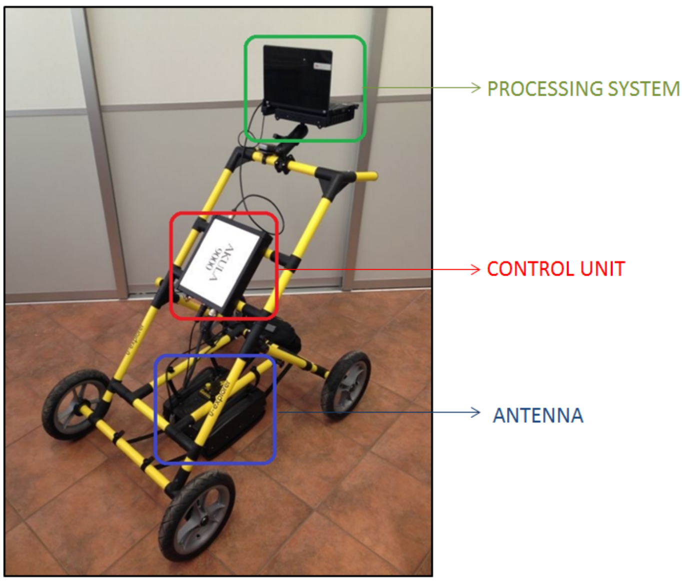


Fig. 5 Typical system for GPR survey

GPR survey can be divided into two phases: a pre-processing phase and post-processing phase. A flowchart for both phases was given by Kang et al. [14] and is shown on Figure 6. Post-processing phase is used for interpretation of collected data form pre-processing phase which include certain noises (can be reduced by averaging) and interferences. Still, in many cases it is possible to use the results from a GPR survey with very modest processing [4].

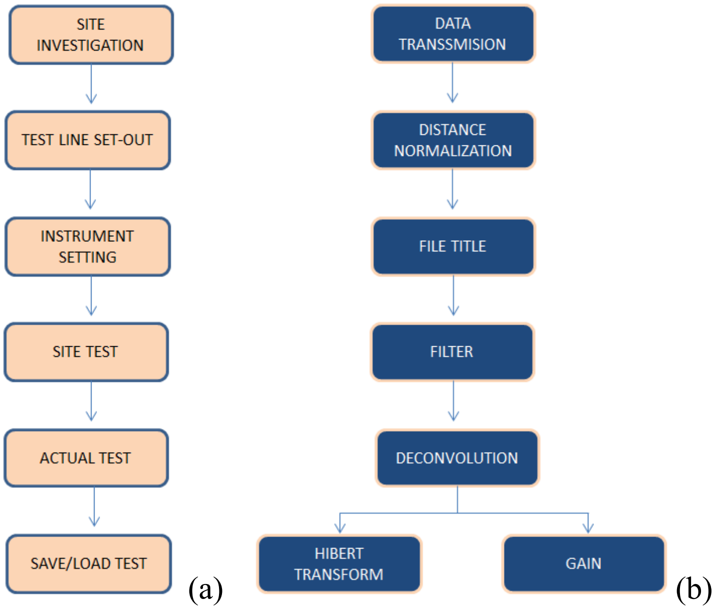


Fig. 6 Flowchart of pre-processing phase (a), and post-processing phase (b) for GPR survey [14]

3 CASE STUDIES OF GPR APPLICATION ON EMBANKMENTS

This chapter presents some case studies where GPR was proven as reliable and highly effective tool for locating anomaly zones in transportation embankments and embankments constructed for water management purpose, as well as in foundation soil under embankments. When conducting a GPR survey on river embankment, it is recommended to position measurement profiles along the rim of an embankment, along inner and outer shelves, and also within intervening areas between the river and the embankment [11]. Dealing with problems associated with river embankments usually comes after events which caused material damages, or even worse, fatalities. Most often, embankment instabilities occur during river flooding events, when river water level rises. Even though rivers prone to floods are managed with attention, unexpected problems can occur. Mori [4] and Kang et al. [14] give an extensive overview of floods in world during past decade.

Importance of dealing with investigation of anomalies in embankments by using geophysical methods was recognized by German Ministry for Education and Research, which has supported a research project DEISTRUKT [5, 18]. A reason for support were flood events in central Europe which occurred in last decade, causing problems with river embankments where many of them are more than 100 years old. GPR survey was conducted along with two other geophysical methods, a mutual impedance of loop antennas measurements and D.C. resistivity method. One typical GPR profile is shown in Figure 7. Some features such as pipes, layer boundaries or erratic blocks can be recognized. It should be noted that penetration depth is limited, but acquired image has very high resolution.

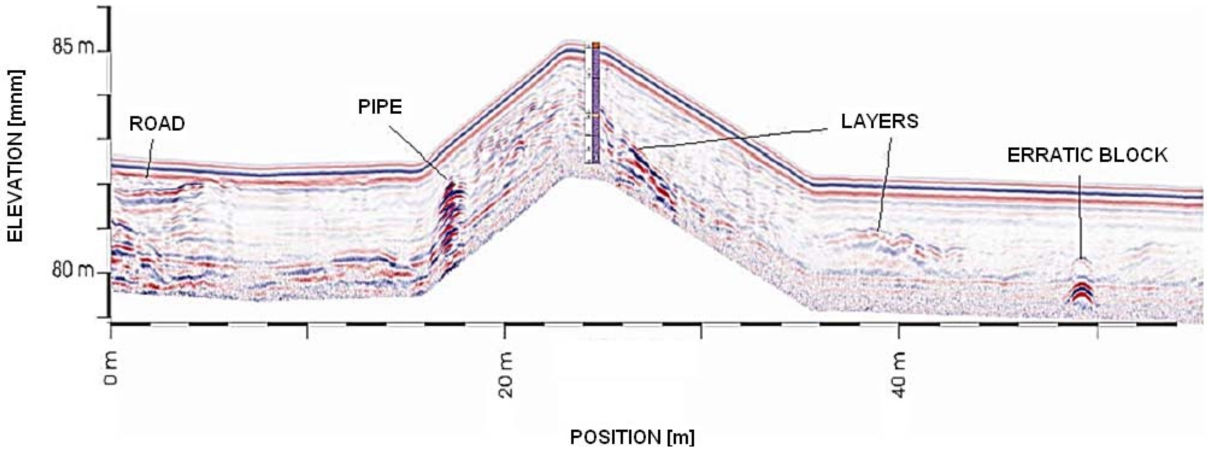
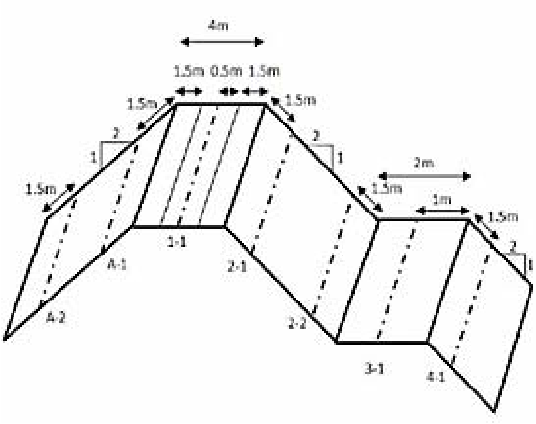


Fig. 7 GPR section across embankment [18]

Another project called 'IMPACT: Investigation of Extreme Flood Processes & Uncertainty' was dealing with river embankment assessment and reduction of flooding risk. It was supported by the European Commission under the Fifth Framework Programme, through Environment and sustainable development programme (1998-2002). Project objectives were to advance scientific knowledge and understanding, and develop predictive modelling tools in four areas, among which was the use of geophysical techniques for the rapid integrity assessment of flood defence embankments [19]. One of IMPACT documents 'A Review of geophysical monitoring methods' [20], a state-of-the-art overview, presents geophysical methods which can be useful for investigation of embankments. As a good example of GPR application practice, a research work of Johannson [21, 22] was pointed out. Johansson presents application of GPR at six embankments, in order to determine state of the watertight core. The accuracy was estimated to be about ±0.1 m, or about 5% of the measured depth to the core crest. One of investigated embankments was Grundsjön embankment on the Ljusnan River. Data collected with 500 MHz antenna showed good quality along the entire embankment, except at location of the sinkhole where the signal response was significantly weaker. Reconstruction works discovered presence of loose coarse sands and gravels, where the internal erosion (below the survey penetration depth) was manifested.

Southern Taiwan represents specific climate area where typhoons accompanied with heavy rainfalls (which exceed 2000 mm) can lead to flood causing serious damages. River embankments in this part of Taiwan are made of earth fills and covered by concrete plates on the surface. One typical embankment failure scenario includes water infiltrating between concrete plates and erode earth fillings. Lee et al. [23] conducted extensive GPR investigation works for detection of insignificant signs of damage on the concrete plates in the early stages of erosions. GPR results were validated by comparing to the actual conditions, where concrete plates were moved away. Four embankments were investigated, with total length of GPR profiles of about 1.7 km. For investigations, a 400 MHz antenna was used, with maximum two-way travel time set to 50 ns. Figure 8a shows survey lines on one of the embankments. Antenna was dragged along survey lines in the longitudinal direction, Figure 8b.

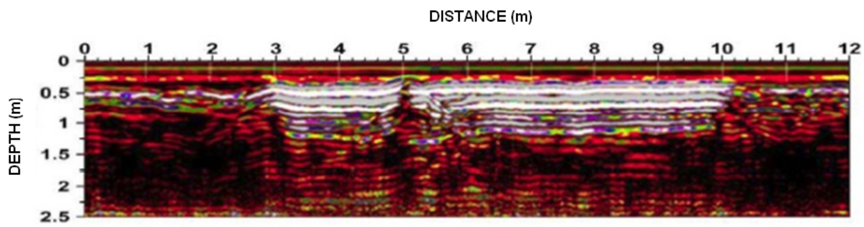


(a)

(b)

Fig. 8 GPR survey lines at the Shan-hua embankment (a) and antenna dragging on the embankment slope (b) [23]

Eroded cavities were detected and shown on GPR profiles, Figure 9a, as strong reflections. After removing concrete plates, Figure 9b, the accuracy of GPR investigation of cavities positions and extent was proven. The authors also propose an original classification system where embankments are divided into intervals, Figure 10. Each interval was assigned a number value between 1 and 5, representing interval with no erosion and interval with high erosion degree, respectively.



(a)

(b)

Fig. 9 The comparison of GPR profile (a) with actual cavity in embankment (b) [23]

No. of concrete plate		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22
Survey line	1–1	1	2	2	2	2	1	1	1	1	1	1	1	1	1	1	1	1	1	2	2	1	2
	1–2	3	3	2	1	1	2	3	3	2	2	1	1	1	2	2	1	1	1	1	1	2	2
	2–1	2	3	3	3	4	4	2	1	1	1	2	1	2	2	1	1	1	1	1	1	1	1
	2–2	3	3	3	3	3	3	3	1	2	1	1	1	1	2	1	1	3	3	3	3	1	2
	3–1	1	3	3	3	3	3	3	2	1	2	2	3	1	2	1	1	1	1	1	1	4	3
	4–1	3	3	3	3	4	3	3	3	3	3	3	3	3	3	3	4	4	3	4	3	5	2

Fig. 10 Erosion degree of each span at one part of Shan-shang river embankment [23]

Figure 11 shows result of GPR measurements of Odra river embankment using 200 MHz antenna [11]. Lower frequency in this case means greater penetration depth (in comparison to previous case studies), which enabled identification heterogeneities not just within the 4m high embankment but also within foundation soil. GPR profile shows no meaningful heterogeneities in embankment, but anomaly was identified in foundation soil. Borehole investigation works followed GPR survey and showed that anomaly is permeable layer of coarse-grained gravel, which could cause hydraulic puncture during high water events.

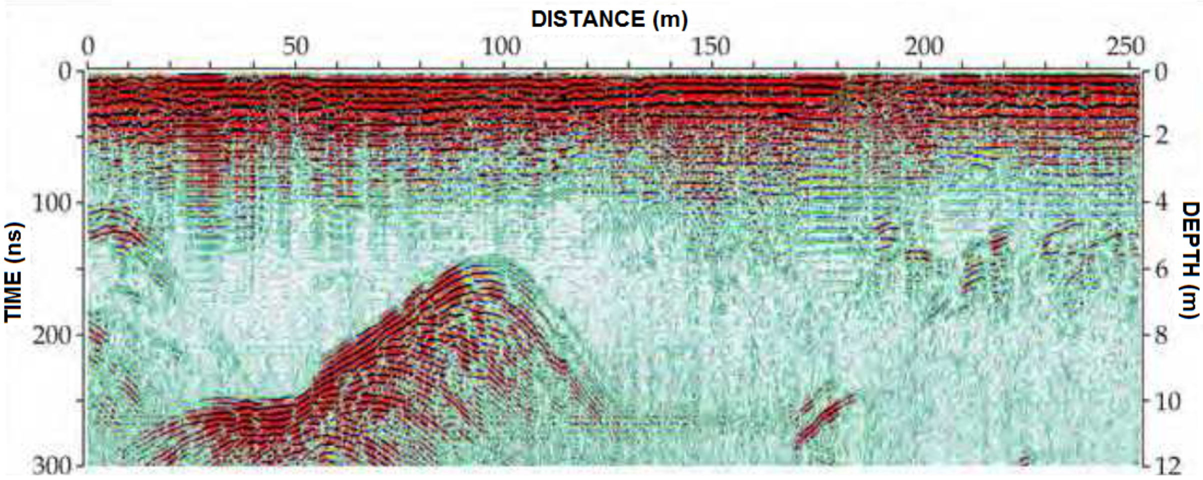


Fig. 11 GPR profile from investigation of Odra river embankment [11]

Two next examples show effectiveness of GPR measurements in detection of embankment anomalies linked with animal activities. Seriousness of animal activities consequences in embankments were discussed globally, for example US National Dam Safety Program Research Needs Workshop was organised under theme of 'Impact of Plants and Animals on Earthen Dams' [24]. Xu et al. [25] used GPR method for detecting a termite nests inside embankment body (a factor from D category proposed by Niederleithinger et al. [5]). Soil type which is used for embankment construction in China is suitable for termite nest. When river water level rises, the water enters in huge number of termite nests, leading to concentrated leakage of water. The results of investigation with 500 MHz antenna showed that features accompanied with termite activities such as principal nest, secondary nests and traffic routes can be easily detected with GPR technology, Figure 13. When this technology was applied to termite nests in Vietnam, detection accuracy level was up to 89%.

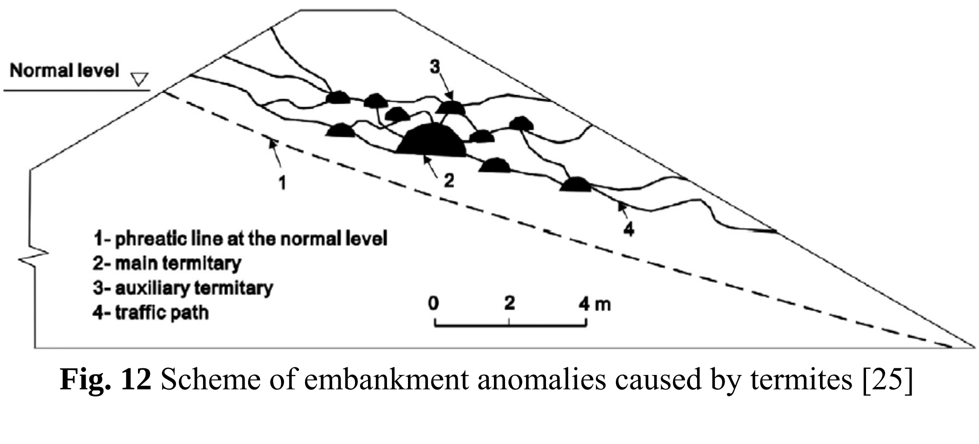
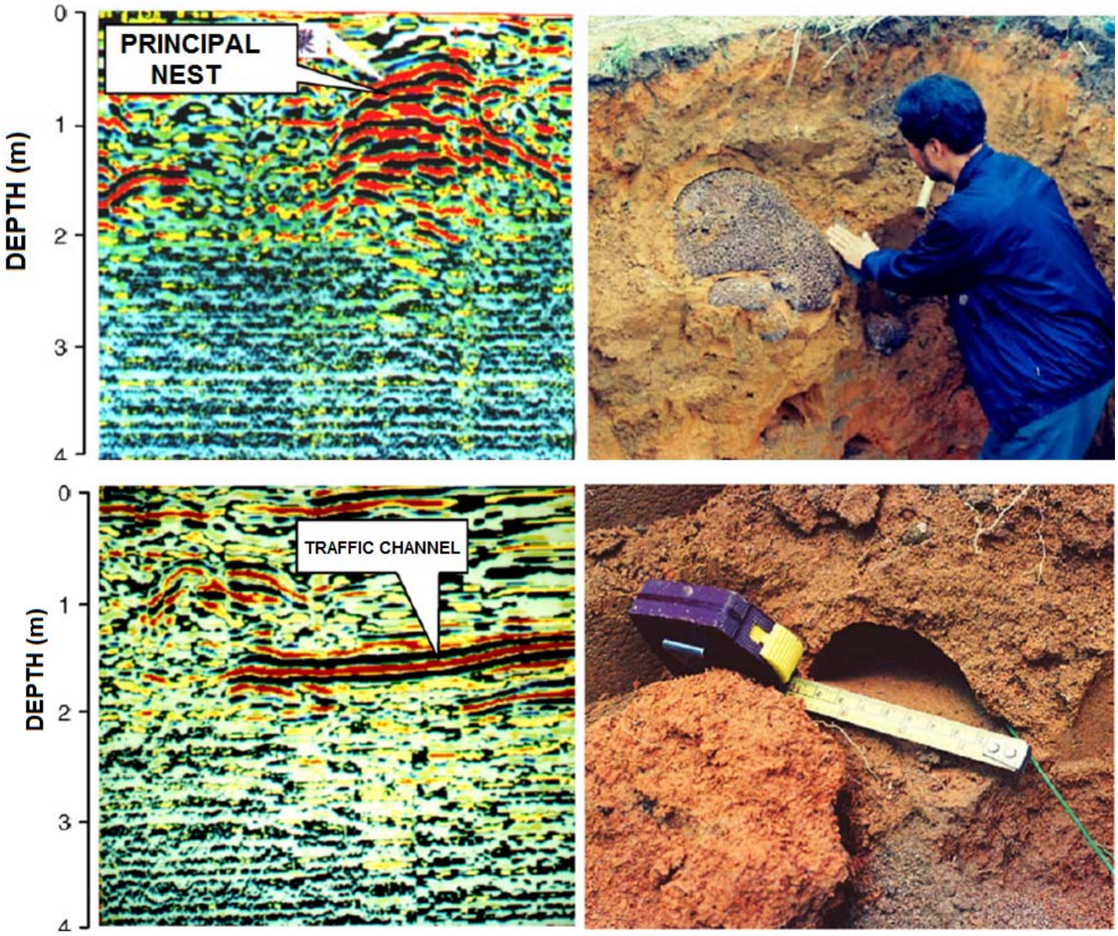


Fig. 12 Scheme of embankment anomalies caused by termites [25]



(a) (b)

Fig. 13 GPR images of termite nests and photos of excavation on location of principal termite nest (a) and traffic channel (b) [25]

Another application of GPR usage for detection of animal (nutrias and foxes) burrows is conducted by Di Prinzio et al. [3]. Here, embankments in area of Bologna district in Italy were investigated using a 250 MHz antenna. The main aim was to assess effectiveness of GPR in detection of voids in sandy-clayey soils. Burrows which were previously detected by visual inspection were confirmed, and those which were previously undetected (because they were masked by vegetation), were discovered by GPR. It was also observed that after few tens of centimeters in upper part, a burrows split in smaller branches. Correctness of shape and depth (up to 1 m) of detected burrows was confirmed after restoration and reconstruction of embankments. A 3D model of underground terrain was made, and Figure 14 shows a different time slices retrieved from 3D model.

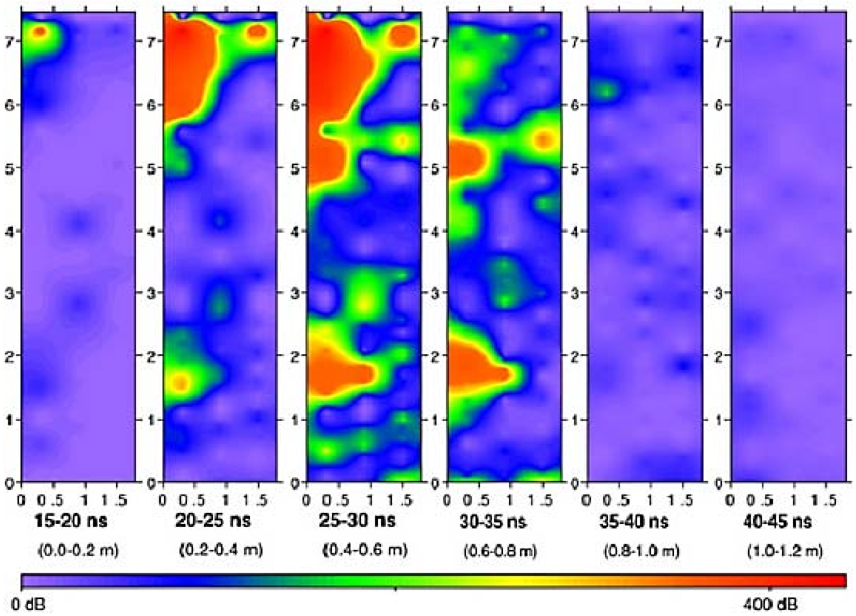


Fig. 14 Time slices obtained from 3D GPR model [3]

Besides construction of embankments as structures which prevent flooding events, embankments can be constructed for transportation purposes, as described earlier in paper. Some interesting examples of GPR assessment of transportation embankments will be shown in sequence.

A Victorian embankment from the Great Central Railway (GCR), East Leake, Nottinghamshire in England, was investigated using combined geophysical and geotechnical investigation works [26]. Purpose of investigation works was to identify changes in fill regimes and to provide evidence for linking these changes to poor track geometry. Significant characteristic of Victorian embankment is a very heterogeneous fill material, which is consequence of construction techniques. Originally designed ballast pavement is from 0.4 – 0.8, and indeed, it appears as strong continuous reflector on GPR profile (obtained with 250 MHz antenna) at approximately 0.5 m depth along line shown on Figure 15a. However, on Figure 15b it is manifested as highly disrupted reflector. Authors indicate that this disruption is related to the line, in this part, taking very high train loads.

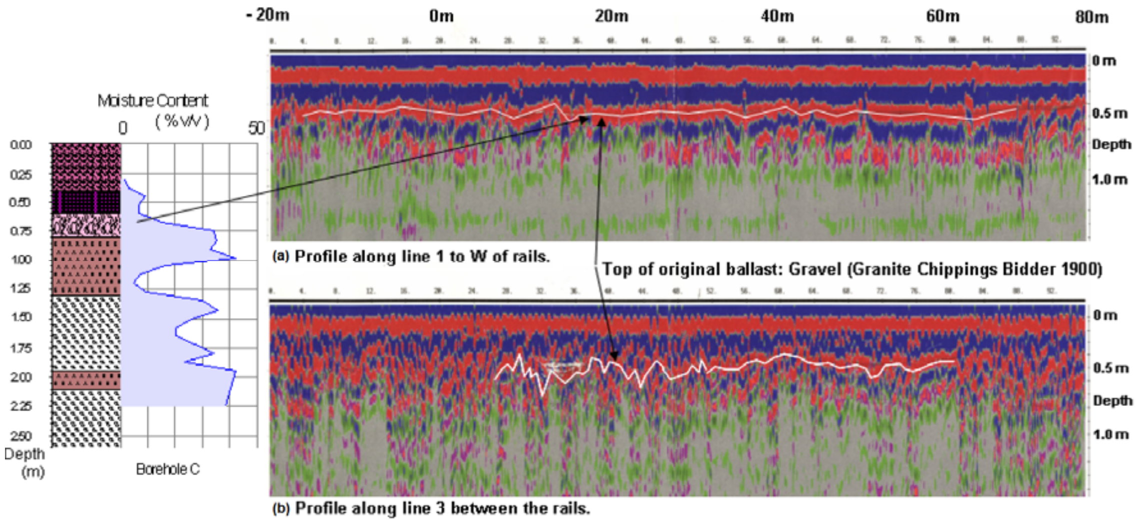


Fig. 15 GPR profiles indicating deterioration in the condition of the base of ballast between the rails [26]

4 CROATIAN EXAMPLE OF EMBANKMENT ASSESSMENT WITH GPR

On embankments at railway route Novska - Dubica in Croatia, a several landslide prone section were registered. Embankments, with height from cca 2 m up to cca 8 m, were constructed in two parts. First, lower, part represents a mixture of crushed rock and coherent deposits, and second, upper, part is constructed of crushed rock alone. Aggravating circumstance for stability of embankment was location of embankment which is situated next to river Sava, whose annual water level variations have negative influence on overall stability. Railway route was designed around three decades ago, but today no relevant geotechnical data exist for this location, and only available working data were old design drawings, Figure 16a. In order to evaluate a condition of embankment, a boundary between two parts of embankment had to be detected. A GPR survey was conducted, with 400 MHz antenna, and results for a part (300 m) of 14 km long route are shown on Figure 16b. A boundary between two embankment parts is shown as blue line, and depressions up to 1 m large are clearly visible. Lower part of embankment, a mixture of coherent deposits and crushed rock attenuated signal. After conducting GPR investigation, an optimization of investigation boreholes was possible.

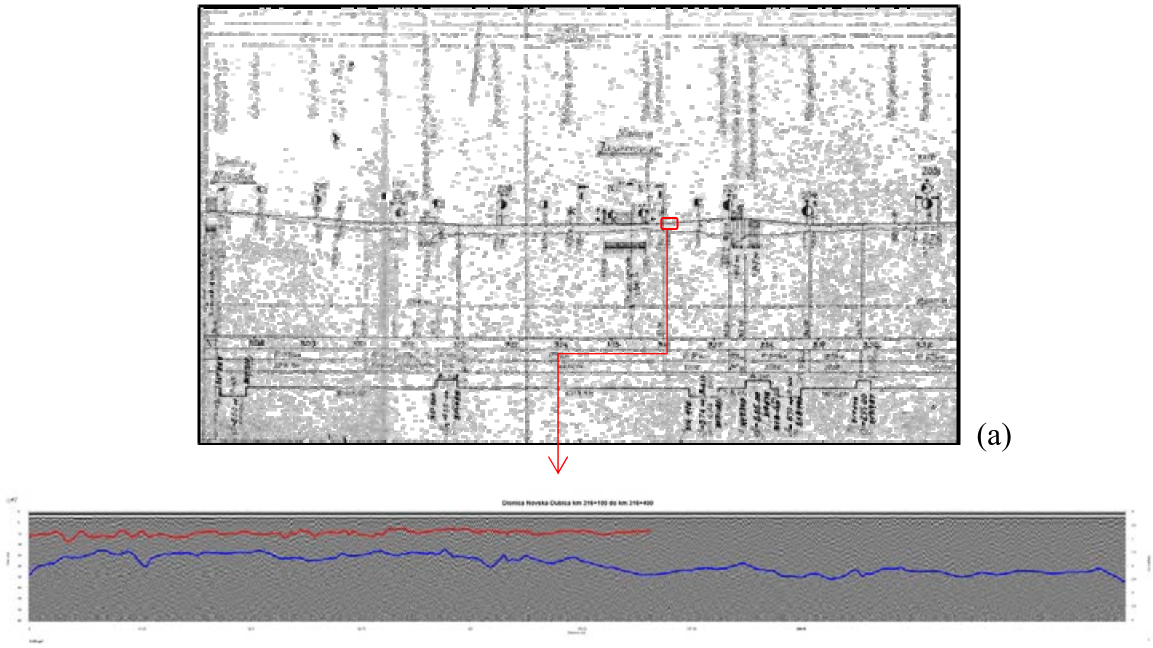


Fig. 16 Old drawing of Novska - Dubica railway route (a), a GPR image of 300 m long part of route (b)

5 CONCLUSION

Ground Penetrating Radar (GPR) represents a non-invasive geophysical method which is based on emission and receiving of electromagnetic waves, providing a high resolution images of subsurface. It is proven as reliable tool in series of applications in civil engineering, among which is investigation of embankments. Regardless of whether water management (constructed for flood mitigation) or transportation (constructed for road/railway to follow designed level) embankment is investigated, anomalies which cause its instability can be detected. It is recommended to conduct GPR investigations along the rim of an embankment, along inner and outer shelves, and also within intervening areas between the river and the embankment (if embankment for flood mitigation is investigated). Selection of suitable antenna is of great importance. If lower frequency antenna is chosen, one should be aware that a greater investigation depth will be achieved, but in same time image resolution would not be high as the resolution when using higher frequency antenna. In latter case, however, investigation depth is limited. GPR investigation can give answers to doubts whether there are certain anomalies in embankment, such as voids, surface cracks, shallow and deep cavities and inhomogeneities inside embankment fill, or whether there are anomalies in foundation soil (for which lower frequency antennas are recommended). Anomalies are consequence of number of influencing factors discussed in this paper. GPR investigations, however, cannot give qualitative information about embankment material parameters which are necessary for decision making process on necessary remediation measures. These parameters, which include soil strength, compressibility and permeability, can be determined by conducting invasive borehole drilling. GPR, in this case, can be very useful tool for borehole optimization process.

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DIRECT AND INDIRECT LABORATORY DETERMINATION OF THE HYDRAULIC CONDUCTIVITY OF FINE GRAINED SOILS

D. Marcic[1], A.Kordic[2] and M.S. Kovacevic[3]

Abstract
The hydraulic conductivity is one of the most important soil parameter for any water management project such us seepage through or under a dam and drainage from subgrades or backfills. It is defined by Darcy's law and represent a measure of the soil's ability to transmit water when submitted to a hydraulic gradient. The term coefficient of permeability is also sometimes used as a synonym for hydraulic conductivity. The paper presents results and comparison of direct and indirect laboratory determination of the hydraulic conductivity of fine grained soils. Directly, the value of hydraulic conductivity was determined in the hydraulic oedometer by applying modern flow-pump test. Indirectly, the value of hydraulic conductivity was determined based on the results from standard oedometer test using an incremental load procedure and consolidation curve.

Keywords
hydraulic conductivity, fine graind soil, laboratory methods, flow-pump test, hydraulic oedometer, soil consolidation

1 INTRODUCTION

The hydraulic conductivity of soil is a parameter that helps define the velocity of water passing through porous materials. It depends not only on the characteristics of the porous medium but also on the characteristics of the fluid itself such as density and viscosity [1].

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Determination of hydraulic conductivity is very difficult because of a number of problems resulting from the nature of permeability of fine grained soils which depends on mineralogical composition, microstructure, initial state of saturation, chemical composition of percolating water, initial hydraulic gradient or drainage conditions [2]. Field-testing the hydraulic conductivity covers a significantly larger volume of tested area with the testing duration relatively short. Obtaining the correct results is limited due to an inadequate familiarity of the geometry and hydraulic limits of the medium with a probability of the cost of works escalating. On the other hand, laboratory testing covers a small volume of the observed area so there is a problem in obtaining representative samples. Exceptionally important is the manner of extracting and preserving samples while the duration period of the experiment is often relatively long. The advantage is the low price of the experiment and acquired results may describe the medium's property quite well [3].
The hydraulic conductivity can be determined in a laboratory by direct and indirect methods. For fine-grained soil that has a small permeability, the hydraulic conductivity is directly determined by applying the *flow-pump test*. Indirectly, the coefficient of permeability for fine-grained soil is determined in a standard oedometer by applying the *consolidation test*.

2 INDIRECT DETERMINATION OF THE HYDRAULIC CONDUCTIVITY IN A STANDARD OEDOMETER

For indirect determination of the hydraulic conductivity in a standard oedometer, the consolidation test is applied [4]. The coefficient of consolidation (C_v) is determined, and is then used to calculate the appropriate hydraulic conductivity (k).
Following application of an individual load increment, a sudden increase in pore pressure occurs in the fully saturated soil sample. Given that the release of water is permitted in the standard oedometer on the upper and lower side of the sample, in time, and depending on the stiffness of the sample, the pore pressure decreases, and the effective stress increases and subsequently deformation of the sample. This process last until the changes in the pore pressure cease and is called a primary consolidation. Given that horizontal deformation in the oedometer experiment is prevented, and that water can flow only in the vertical direction, this involves one-dimensional consolidation of soil. One-dimensional consolidation of soil is described using the partial differential equation that links changes in pore pressure to time (t) and depth (z):

$$\frac{\partial u}{\partial t} = C_v \frac{\partial^2 u}{\partial z^2} \tag{2}$$

$$C_v = \frac{kE_{oed}}{\gamma_w} \tag{3}$$

where: u – pore water pressure,
C_v – coefficient of consolidation,
k – hydraulic conductivity,
E_{oed} – oedometric modulus of primary consolidation and
γ_w – bulk weight of water.

The solution of a partial differential equation (2) allows determination of the degree of consolidation (U) which represents the ratio of settlement in a particular time period and the total settlement. The average degree of consolidation is expressed as a function of the dimensionless time factor (T_v) in the form of Fourier series:

$$U = f\left(T_v\right) = 1 - \sum_{m=1}^{\infty} \frac{2}{M^2} e^{\left(-M^2 T_v\right)} \tag{4}$$

where: $M=(\pi/2)(2m+1)$.

For practical purposes, it becomes necessary for each average degree of consolidation (U), with the aid of an equation (4), to determine the time factor (T_v). For the soil sample that is tested in the standard oedometer, where draining is permitted on both sides, it is achieved using the following equations:

$$T_v = \frac{\pi}{4}\left(\frac{U}{100}\right)^2 \qquad \text{for} \quad U < 60\% \tag{5}$$

$$T_v = 1.781 - 0.933\log\left(100 - U\right) \qquad \text{for} \quad U \geq 60\% \tag{6}$$

Na primjer za 50% konsolidaciju odnosno vrijednost ukupnog slijeganja, vremenski faktor T_v iznosi 0.197, dok za 90% konsolidaciju T_v iznosi 0.848.
For example, a 50% consolidation or the value of total settlement, the time factor T_v amounts to 0.197, whereas for 90% consolidation T_v is equivalent to 0.848.
Bezidmenzionalni vremenski faktor T_v i stupanj konsolidacije C_v povezuje slijedeći izraz:
The dimensionless time factor T_v and the coefficient of consolidation C_v are connected using the following expression:

$$T_v = \frac{C_v t}{d} \tag{7}$$

Where: d – longest drainage distance.

Determining the coefficient of consolidation, and therefore indirectly also the hydraulic conductivity, is now reduced to seeking a parameter that allows the overlapping of theoretical and experimental curves for settlement of the sample in a standard eodometer for each particular load increment. In the eodometer test, the degree of consolidation U represents the ratio between sample settling in a particular time period $s(t)$ and total settlement s_f at the end of primary consolidation. Sample settling at the moment t in the standard eodometer is continually read using a displacement-measuring device. This represents the difference in the dial reading at the time t and the dial reading at the moment of applying the load increment:

$$s(t) = d(t) - d_0 = s_f U\left(T_v\right) = \left(d_{100} - d_0\right) U\left(\frac{C_v t}{d_m}\right) \tag{8}$$

where: d_0 – dial reading at the beginning of consolidation ($U = 0\%$),
d_{100} – dial reading after completion of consolidation ($U = 100\%$),

The average drainage path for water from the sample d_m for both-sided draining is equivalent to half the height of the sample and conforms to degree of consolidation $U = 50\%$:

$$d_m = \frac{1}{2}\left(h_0 + d_i - \frac{1}{2}(d_{100} + d_0)\right) \tag{9}$$

where: h_0 – initial sample height and
d_i – initial dial reading.

The overlapping of theoretical and experimental curves for the settling of the sample in the standard oedometer for each particular load increment is carried out only for the primary consolidation phase. There exist a number of methods for determining the coefficient of consolidation from the measured results in the consolidation test. The best known of all is the Casangrade log-time method, which determines the parameters d_0, d_{100} and C_v, (Figure 2).

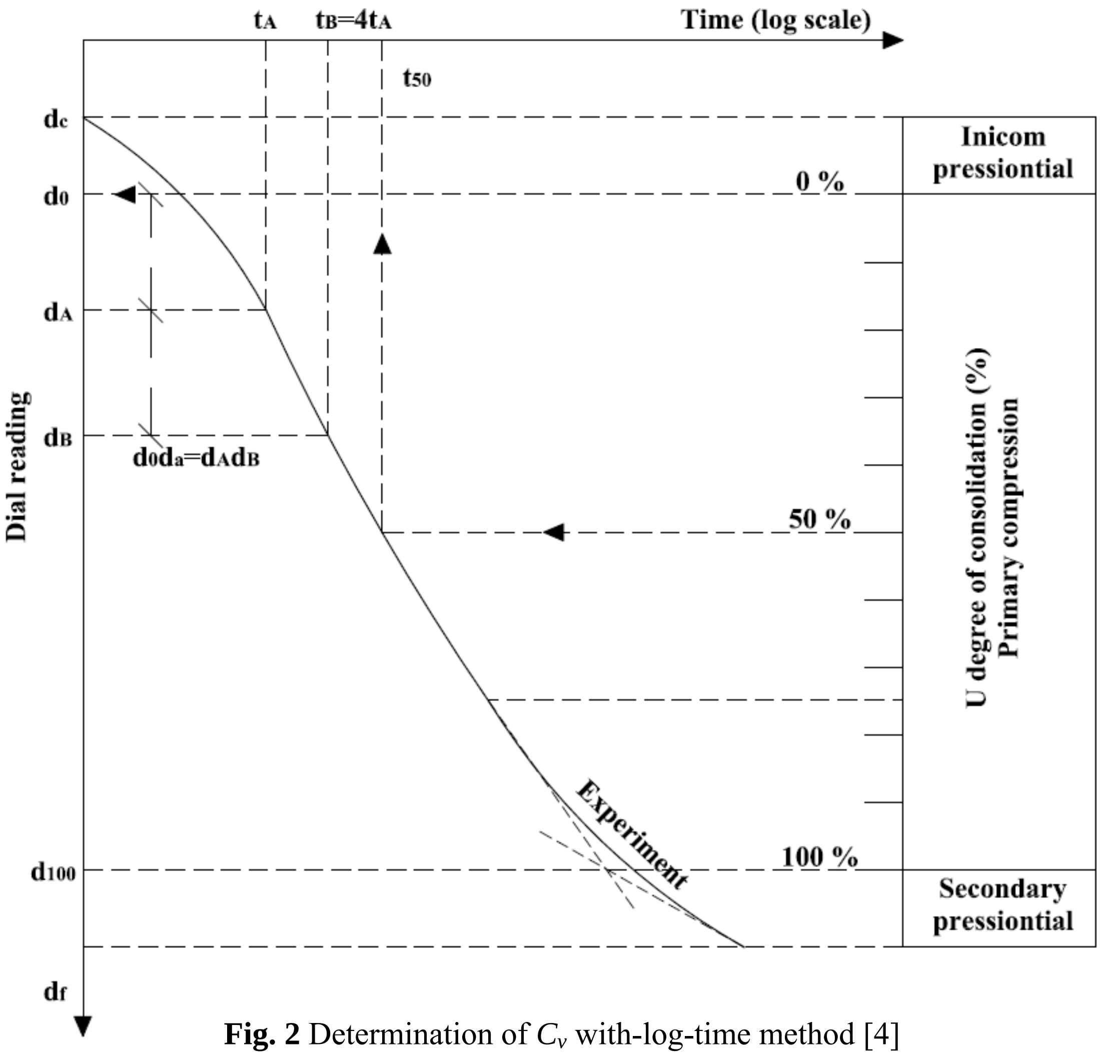

Fig. 2 Determination of C_v with-log-time method [4]

The start of primary consolidation (U=0%) and the parameter d_0 is determined by selecting two points A and B on the experimental consolidation curve in the region of U < 60%, which is to be also found on the theoretical consolidation curve.
Given that the first section of the curve for primary consolidation can be approximated by the parabola, the following is then true:

$$\frac{t_B}{t_A} = \frac{T_v^B}{T_v^A} = \frac{U_B^2}{U_A^2} = \frac{(d_B - d_0)^2}{(d_A - d_0)^2} \tag{10}$$

where: d_A – dial reading at point A,
d_B – dial reading at point B,
t_A – time at point A,
t_B – time at point B,
T_v^A – time factor at point A,
T_v^B – time factor at point B,
U_A – degree of consolidation at point A and
U_B – degree of consolidation at point B.

Now, with the aid of the equation (10), the start of primary consolidation can be determined:

$$d_0 = \frac{d_A\sqrt{t_B} - d_B\sqrt{t_A}}{\sqrt{t_B} - \sqrt{t_A}} \tag{11}$$

In practice, if often happens that $t_A \approx 1$ min and that $t_B \approx 4\ t_A$. Then, $d_0 = 2d_A - d_B$.
For determining the end of primary consolidation (U=100%) and the parameter $d_{100,}$ the fact is that the theoretical consolidation curve has an inflection point I at approximately 75% of consolidation ($U \approx 75\%$). At this point of the curve, the trend changes and has a larger inclination. The intersection of the tangent on the experimental curve at the inflexion point and extended line of the secondary consolidation is the point J, which defines the parameter d_{100}.
The reading that conforms to 50% of consolidation (U=50%) is obtained using the expression:

$$d_{50} = \frac{1}{2}(d_0 + d_{100}) \tag{11}$$

Reading d_{50} from the experimental curve provides a time of 50% consolidation t_{50}.
Given that U = 50%, Tv = 0.197, the coefficient of consolidation gives:

$$C_v = 0.197\frac{d_m^2}{t_{50}} \tag{12}$$

Finally, the hydraulic conductivity, for each loading increment, is calculated with the aid of the equation (3):

$$k = \frac{C_v\gamma_w}{E_{oed}} \tag{13}$$

3 DIRECT DETERMINATION OF THE HYDRAULIC CONDUCTIVITY IN A HYDRAULIC OEDOMETER

For direct determination of the hydraulic conductivity k in a hydraulic oedometer, the flow pump test was applied which represented an inversion version of the constant head test [5]. Instead of imposing a difference in total head at the ends of the sample and measuring flow, a process conducted by forcing a flow and measuring a constant difference in pressures at the sample ends. This is the most important advantage of the flow pump test with respect to the constant head test, since it was ascertained that the flow could be more precisely imposed than measured [5]. The procedure was carried out in such a way that the fluid aided by hydraulic pumps was forced into a sample at a constant speed, i.e. at a given flow rate, during which a change in the pressure difference is measured at the sample ends (Figure 3)

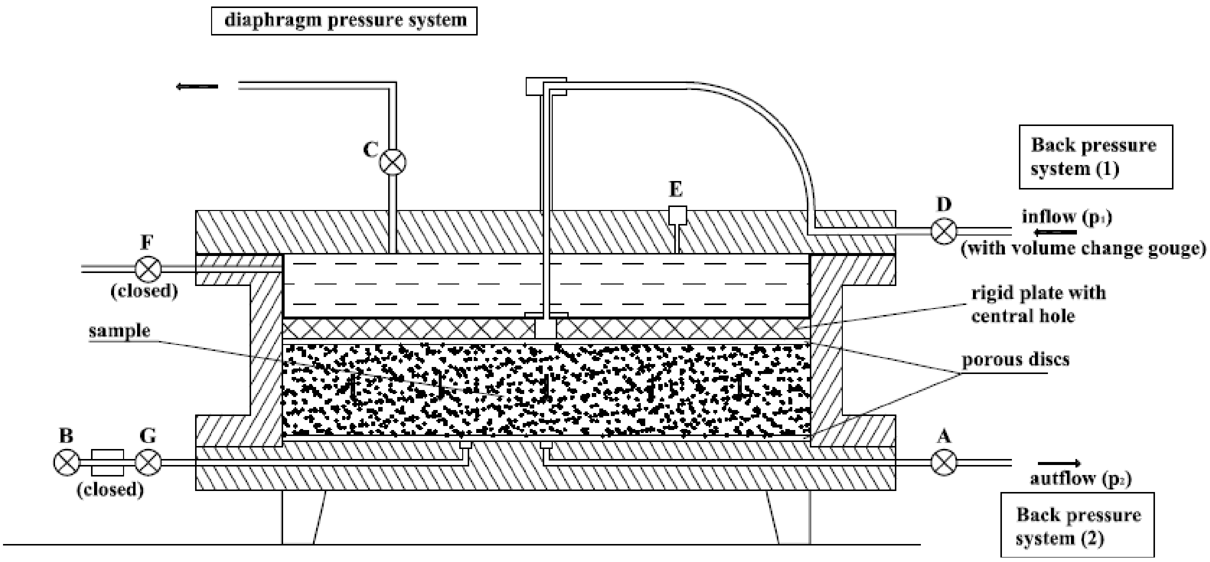

Fig. 3 Flow pump test in Rowe consolidation cell [5]

For each incremental loading and constant pressure difference in the upper and lower sides of the sample, the mean hydraulic gradient i can be determined by the expression:

$$i = \frac{\Delta p}{\gamma_w H} \tag{14}$$

where: i – mean hydraulic gradient,

$\Delta p = p_1 - p_2$

p_1 – inflow preassure

p_2 – outflaw preassure

H – height of the specimen and

γ_w – bulk weight of water.

For each incremental loading, the hydraulic conductivity (k) is calculated by the expression:

$$k = \frac{q}{Ai} \tag{15}$$

where: q = mean rate of flow, $q = \Delta Q/\Delta t$,
$A = D^2\pi/4$ – cross-sectional area of the specimen,
Q – cumulative flow of water and
t – elapsed time.

4 DETERMINATION OF HYDRAULIC CONDUCTIVITY FOR LANDSLIDE REMEDIATION ON THE ZAGREB-RIJEKA RAILWAY LINE

In order to avoid a larger number of intersections of the railway line and road and obtain better railway line traffic parameters on the respective section, between the Karlovac and Mrzlo Polje stations in 1963 the construction of a new section of railway line approx. 3 km in length began.

At the stated railway line route, practically since the time of construction, landslides have occurred regularly on the cutting slopes. In 1975, remediationss were conducted at 5 locations using RC piles and prefabricated support walls. At 5 locations, where landslides subsequently began to occur, temporary remediation measures were carried out by removing the foundation soil, and construction of a temporary support wall from the rails and wooden sleepers. On the upper plateau of the cutting, there are private structures, whose stability is greatly jeopardises by landslides of the foundation soil.

The conclusion was drawn that the main reason for landslides was inadequately regulated drainage of water from the cutting slopes. Given the configuration of the terrain, the general feeding of the upper clay layers relies on the role of direct infiltration from the terrain surface (especially intensive during precipitation or melting of snow) and underground tributaries behind the inclinations.

The underground tributaries may become activated due to porosity of the sewage network and/or water installations, as well as due to infiltrations caused by possibly inadequately resolved surface drainage and drainage of wastewater from the structure into the surrounding area. At places in the crown the cutting fissures were noticed (which probably occurred as a result of the moistening and drying of clay), which especially during rainy periods or when snow melts became filled with water.

Furthermore, as a consequence of the above stated, filtration of water occurs through the ground and the forming of filtration lines along with additional hydrostatic pressure, especially in the slope surfaces that are found under the filtration lines.

Additional geotechnical investigative works were carried out, which required carrying out investigate drilling, engineering and geological mapping, and laboratory testing of the physical and mechanical properties of the foundation soil. Particular attention was given to testing the hydraulic conductivity of fine-grained layers of soil.

Further in the paper, the results of direct and indirect testing of the hydraulic conductivity for chainage km 479+720.310 will be presented. At this chainage, a borehole B2 was carried out to a depth of 20 m. The sample of fine-grained soil on which testing was carried out was taken at depth of 4.3-4.7 m.

All the testing was conducted in the geotechnical laboratory of the Faculty of Civil Engineering, University of Zagreb (Figure 4 and 5).

Fig 4. Standard oedometer

Fig. 5 Hydraulic edometer

5 RESULTS OF DETERMINATION OF THE HYDRAULIC CONDUCTIVITY

The tested soil was high-plasticity clays, with the plastic limit of 77.73%, liquid limit of 19.95% plasticity index of 57.78%, the water content of 21.73%, dry density of 1.81 g/cm^3 and bulk density of 1.92 g/cm^3.

Direct and indirect determination of the hydraulic conductivity in a standard and hydraulic oedometer was conducted for incremental loads of 100, 200, 400 and 800 kPa.

The obtained values of the hydraulic conductivity *k* are given in Table 1 and Figure 6.

Tab. 1 Hydraulic conductivity *k* [m/s]

Effective vertical stress [kPa]	Indirect determination in a standard oedometer	Indirect determination in a standard oedometer
100	$2.32 \cdot 10^{-10}$	$4.42 \cdot 10^{-10}$
200	$1.37 \cdot 10^{-10}$	$2.13 \cdot 10^{-10}$
400	$1.18 \cdot 10^{-10}$	$1.36 \cdot 10^{-10}$
800	$8.69 \cdot 10^{-11}$	$2.34 \cdot 10^{-11}$

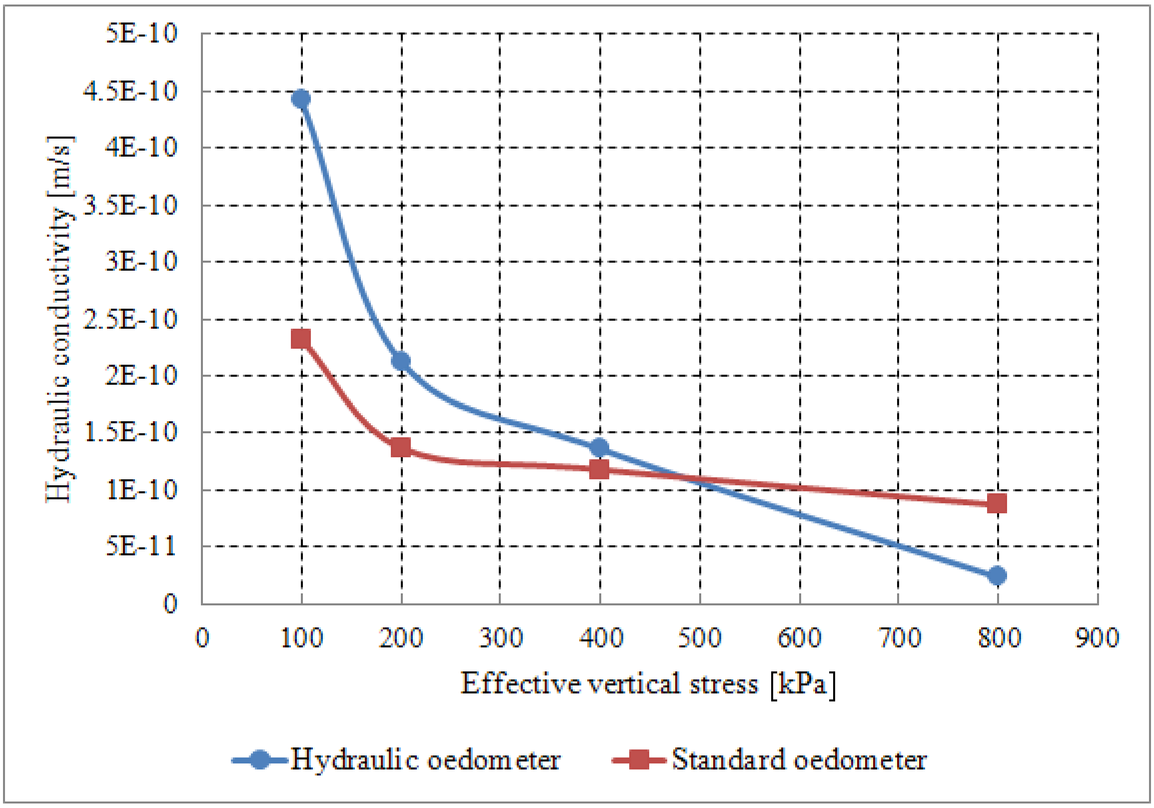

Fig. 6 Hydraulic conductivity

For practical purposes results show similar values in both tests. For vertical stress of 100, 200 and 400 kPa hydraulic conductivity in hydraulic oedometer is 1.9, 1.6 and 1.2 times greater than in standard oedometer respectively. But for vertical stress of 800 kPa hydraulic conductivity in hydraulic oedometer is 3.7 times less than in standard oedometer.

The value of hydraulic conductivity reduces when effective vertical stress increases in both test. It is expected, since the voids ratio reduces, permeability must reduce. This is especially true in the early stages of the test, when the greatest changes in voids ratio occurs. Simiral trend (Figure 7) is obtained in test on a tailings material from a mine in Western Australia [6].

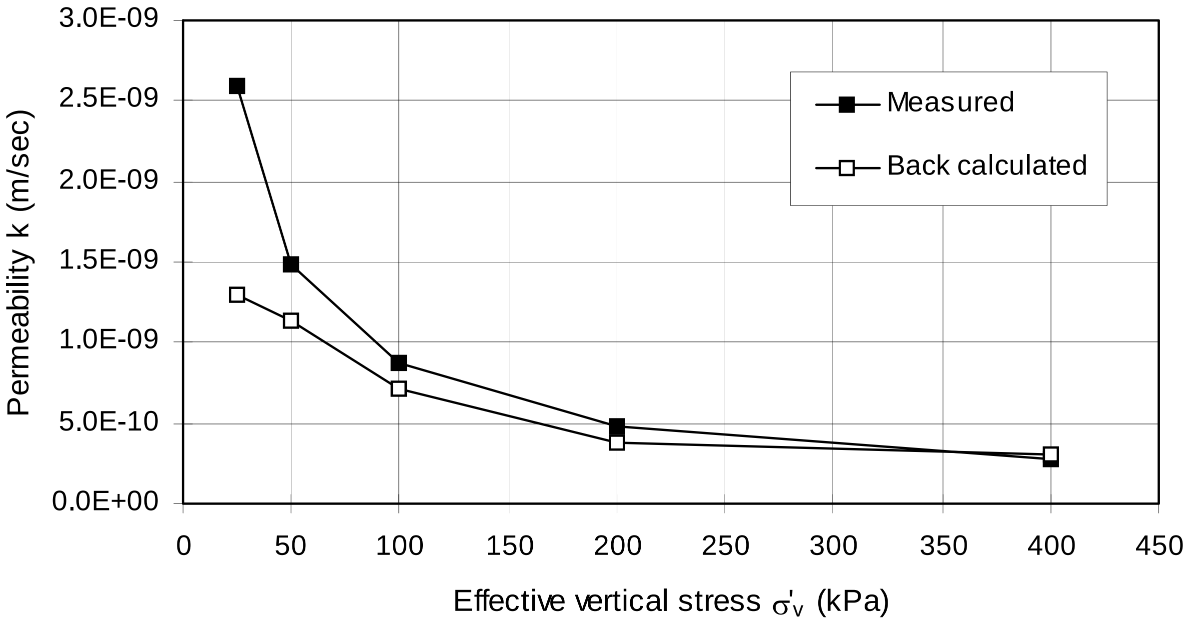

Fig. 7 Results of a direct (measured) and indirect (back calculated) determination of hydraulic conductivity on a gold tailings sample [6]

6 CONCLUSION

Determining the value of the hydraulic conductivity and within the limits in which these values lie is an important factor in implementing reliable hydraulic and geotechnical numerical analyses.

The hydraulic conductivity of fine-grained material can be successfully determined in a laboratory directly by applying the flow-pump test in hydraulic oedometer and indirectly by applying the consolidation test in standard oedometer. Both methods give useful results for practical purposes.

Flow-pump test is much quicker than consolidation test. Duration of flow-pump test for one load incerement is less than 10 min. Duration of consolidation test for one load incerement is not less than 24 h.

Standard oedometer is part of any geotechnical laboratory. Initial cost of equipment for flow-pump test is much higher than for consolidation test.

Flow-pump test is much more reliable and its results can be used as a representative value when comparing the existing laboratory methods for determination of the hydraulic conductivity of fine-grained materials.

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THE NUMERICAL ANALYSIS OF FREEZING OF FOUNDATION JOINT OF THE TERLICKO DAM SPILLWAY

J. Riha[1] and K. Adam[2]

Abstract

At the beginning of the year 2012 the reconstruction of the spillway and the chute proceeded at the Terlicko dam. During the January and February the temperatures decreased below -20 °C, moreover the temperatures below -10 °C continued for 17 days. During this time the upper degraded concrete layer was removed so the spillway chute slab was only 0.7 m thick. During the February the vertical upward displacements were observed at the chute which was probably attributed to freezing of the foundation joint. Additional geological survey indicated soft and water-bearing layer of a poor quality at the foundation joint between the slab and sub-base slates. In the paper the results of numerical modelling of possible freezing of the concrete slab during the event are discussed. The numerical model of the heat flow demonstrated possibility of freezing of the slab down to the foundation joint which may be the cause of the vertical deformation.

Keywords

freezing of concrete slab, spillway chute slab, heat flow, transient thermal analysis, ANSYS

1 INTRODUCTION

The Terlicko dam (Fig. 1) was built in the river Stonavka between the years 1959 and 1964. The dam is located between two cities named Havirov and Tesin in the Czech Republic. The reservoir catchment area is about 82 km^2, the reservoir volume is approximately 27,4 mil. m^3 and the flooded area is 267,6 ha. The embankment dam is 25 m high with inclined clayey core and shoulders made of granulated blast-furnace slag. The crest length is 617 m. The emergency spillway is a concrete side spillway with a chute. The foundation consists mainly of clayey slate, clay and sandstone. The reservoir serves dominantly for the supplying the

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Karvina coal field and the Iron Works in Trinec with service water, moreover, it serves for protecting the undermined territory below the dam from flooding.
One of the current remedial activities was primarily focused on the increase of the spillway capacity and the general repair of the chute downstream of the emergency spillway. The originally planned extensive spillway reconstruction was abandoned as experimental modeling shown that the side spillway (with the crest length 48 m) had sufficient capacity for a safe transfer of a discharge $Q_{10\,000}$ and did not require any considerable interventions of the side weir which was provided by stone block facing. Then the rehabilitation was focused mainly to the spillway chute, its side walls and concrete surface. The modifications were focused mainly on the chute defects which had firstly manifested already in the 1970s. After the verification of spillway and chute dimensions on a physical model, the shape of the cross section was modified and the bottom concrete slab was widened from 8 to 10 m. At the same time the longitudinal section of the chute which was originally designed with a vertical bend was aligned. The significant improvements of hydraulic conditions were made in the bottom part of the chute close to the bridge. The dimensions of the stilling basin below the chute were modified as well.

Fig. 2 View of the Terlicko dam (source: www.vzsterlicko.cz)

The reconstruction of the emergency spillway started in fall 2010 and the deadline of the reconstruction was scheduled for spring 2013. At the beginning of the year 2012 the reconstruction of the spillway and the chute proceeded. During the January and February temperatures decreased down to -20 °C, moreover the temperatures below -10 °C continued for 17 days. During this time the upper degraded concrete layer was already removed, so the spillway chute slab was only 0.7 m thick in the most exposed surface. During the February the upward displacements were observed at the chute. These displacements were by a preliminary expert's estimation attributed to freezing of the foundation joint. Additional geological survey (Fig. 2) proved soft and water-bearing layer of a poor quality at the foundation joint between the slab and sub-base slates. It is probable that the poor quality material is the remain of measures "improving" the damp foundation joint for the travel of mechanization and trucks during the chute construction.

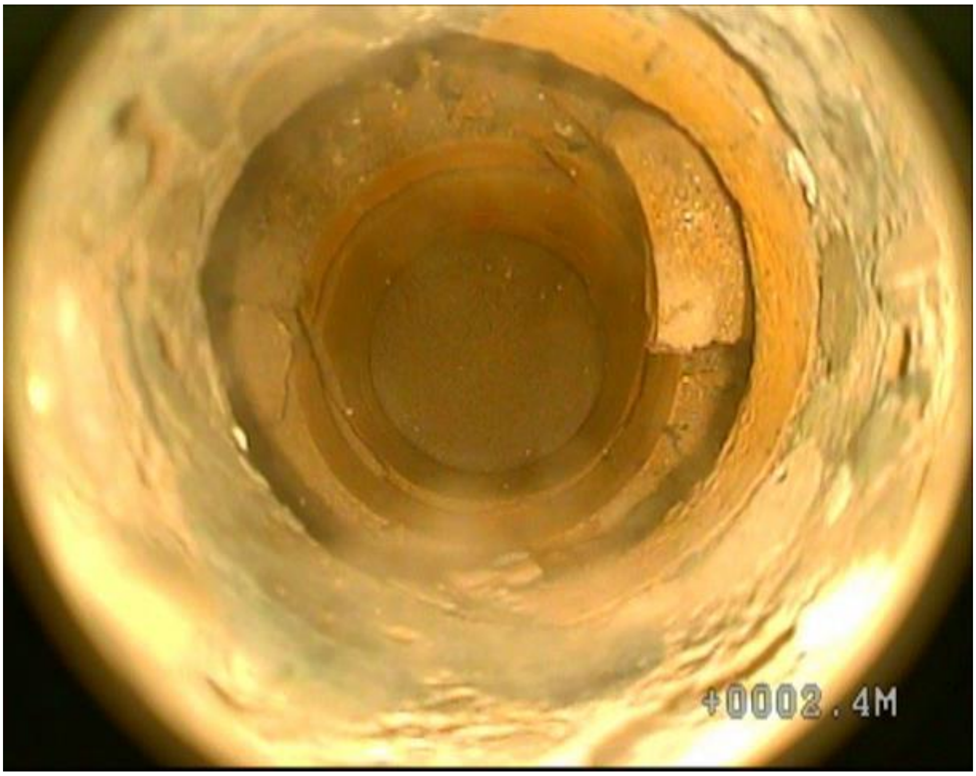

Fig. 2 Geological Survey - Borehole Footage Screen (photographed by V. Bradac)

2 AIMS OF THIS STUDY

The aim of this study is to asses the possibility of the downward freezing of the slab down to the foundation during the very cold period and find out, if the cause of vertical deformation of the slab could be the freezing of the poor quality material at the foundation joint. A numerical heat flow model was applied for this purpose. Firstly the one-dimensional (1D) model was used, after that the two-dimensional (2D) model was set up for more accurate modelling of the heat flow conditions during the frosty period. Finally the results from both models were compared.

3 METHODS OF SOLUTION

Two different methods and software tools were used for the computation. The MS Excel was used for simplified 1D analysis. For 2D modelling commercial ANSYS software was employed. The following paragraphs describe the preliminary assumptions, governing equations, boundary and initial conditions adopted.

3.1 1D-numerical model

Assumptions for the modelling

The heat flow model in 1D considers the heat flows only in one direction – vertical *y* axis. Heat flow in other directions is neglected. The continuum is assumed homogeneous by parts. This corresponds approximately to the heat flow in the central axis of the chute (Fig. 3). The boundary condition on the surface of the concrete slab is taken into account as a heat convection. The constant temperature boundary condition is considered in the sub-base of the chute in the depth of 6 m below the slab surface. The initial condition - temperature distribution along the depth was taken from the steady-state heat flow analysis.

Governing equations and boundary conditions

The energy conservation equation for heat transfer in y direction holds:

$$\frac{\partial}{\partial y}\left(\lambda \frac{\partial T}{\partial y}\right) = c\,\rho\,\frac{\partial T}{\partial t}, \qquad (1)$$

where T is temperature, λ is thermal conductivity (diffusivity), c is specific heat, ρ is density and t is time.

The boundary conditions are as follows:

1st type – Dirichlet condition

$$T/\Gamma_1 = T(L,t) = T_L = \text{konst.}, \qquad (2)$$

3rd type – Newton condition (heat convection)

$$-\lambda\frac{\partial T}{\partial y} = \alpha\,(T(0,t) - T_A(t)), \qquad (3)$$

where $T(y)$ is temperature, α is heat transfer (film) coefficient and T_A is air temperature.

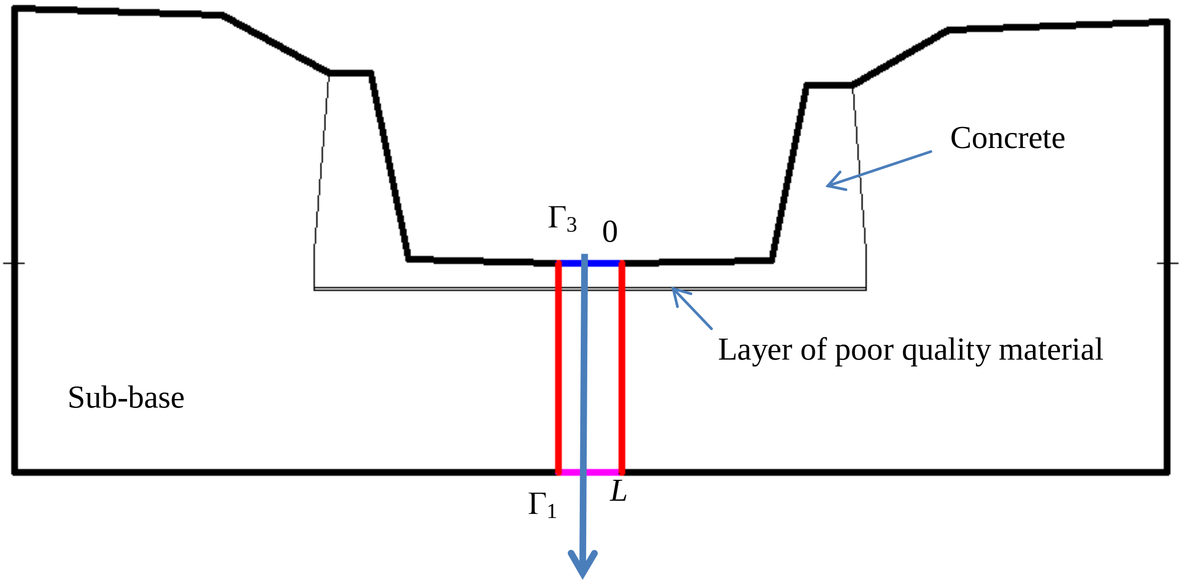

Fig. 3 Scheme of the 1D model

Numerical solution

For the numerical solution the explicite finite difference method was used. The numerical scheme is as follows:

$$c\cdot\rho\cdot\frac{T_i^{k+1} - T_i^k}{\Delta t} = \lambda\cdot\frac{T_{i-1}^k - 2T_i^k + T_{i+1}^k}{\Delta y^2}, \qquad (4)$$

$$T_i^{k+1} = \frac{\lambda.\Delta t}{c\cdot\rho}\cdot\frac{T_{i-1}^k - 2T_i^k + T_{i+1}^k}{\Delta y^2} + T_i^k, \qquad (5)$$

where i expresses spatial discretization, k time discretization, Δy is the spatial step and Δt is time step. For the solution of heat convection the following equation was used:

$$T_{\Gamma 3}(0,t) = T_A(t) - \frac{\lambda}{\alpha}\frac{T_2^{k+1} - T_1^k}{\Delta y}, \qquad (6)$$

The equation (6) was solved iteratively in every time step.

3.2 2D-numerical model

Preliminary assumptions

At the 2D heat flow model it was assumed, that the continuum is homogeneous by parts and also isotropic. The heat convection on the surface of the concrete slab is taken account. The constant temperature in foundation is considered in the depth 6 m, other sources of heat are not considered. The initial condition was the known temperature distribution over the domain; it was computed by steady-state analysis for the time period before the low temperature event.

Governing equations and boundary conditions

The flow domain Ω with a boundary Γ is assumed (Fig. 4). The boundary Γ consists of the parts Γ_1, Γ_2 and Γ_3, $\Gamma = (\Gamma_1 \cup \Gamma_2 \cup \Gamma_3)$ and $(\Gamma_1 \cap \Gamma_2 \cap \Gamma_3) = 0$. For 2D heat transfer holds:

$$\frac{\partial}{\partial x}\left(\lambda_x \frac{\partial T}{\partial x}\right) + \frac{\partial}{\partial y}\left(\lambda_y \frac{\partial T}{\partial y}\right) = c\,\rho\,\frac{\partial T}{\partial t}, \tag{7}$$

where T is temperature, λ_x and λ_y are thermal conductivities, c is specific heat, ρ is density and t is time.

The boundary conditions are as follows:

1st type for the Γ_1 – Dirichlet condition

$$T/\Gamma_1 = T(x, y, t) = T_0 = \text{konst.}, \tag{8}$$

2nd type for the Γ_2 – Neumann condition

$$\lambda_x \frac{\partial T}{\partial x} n_x + \lambda_y \frac{\partial T}{\partial y} n_y = 0, \tag{9}$$

3rd type for the Γ_3 – Newton condition

$$\lambda_x \frac{\partial T}{\partial x} n_x + \lambda_y \frac{\partial T}{\partial y} n_y = \alpha\,(T_{\Gamma 3}(t) - T_A(t)), \tag{10}$$

where n_x and n_y are the directions of the normal vector to Γ_2 boundary, $T_{\Gamma 3}$ is a temperature on the top of the concrete on Γ_3 boundary and T_A is an air temperature.

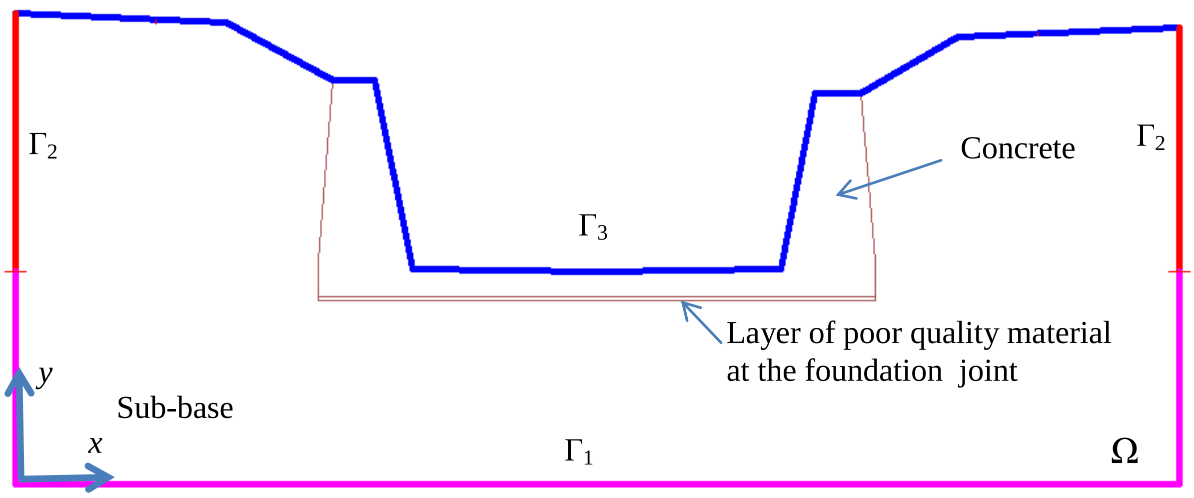

Fig. 4 Scheme of the 2D model

Numerical solution

The implicit numerical integration method in ANSYS software uses iteration for the computation of unknown function T. The flow domain discretization is performed by finite element method (FEM). For the solution the 4-node element with bilinear approximation was used. The equation (7) was transformed by generally known FEM operations [6] to the system of ordinary differential equations:

$$[C]\{\dot{T}\}+[L]\{T\}=0, \tag{11}$$

where $[C]$ is the specific heat matrix, $[L]$ is the conductivity matrix, $\{T\}$ is the vector of nodal temperatures, $\{\dot{T}\}$ is the vector of time derivatives of nodal temperatures. The procedure employed for the solution of the equation (11) is based on the Crank-Nicolson's scheme:

$$\{T_{k+1}\}=\{T_k\}+(1-\theta)\Delta t\{\dot{T}_k\}+\theta\Delta t\{\dot{T}_{k+1}\}. \tag{12}$$

θ = 0,5 is the transient integration parameter, k and k+1 subscripts indicate the time discretization, $\Delta t = t_{k+1} - t_k$ is time step, $\{T_k\}$ resp. $\{T_{k+1}\}$ are vectors of nodal temperatures at time t_k resp. t_{k+1}, $\{\dot{T}_k\}$ resp. $\{\dot{T}_{k+1}\}$ are time rates (derivatives) of the nodal temperature values at time t_k resp. t_{k+1}. Equation (11) can be written for the time t_{k+1} as follows:

$$[C]\{\dot{T}_{k+1}\}+[L]\{T_{k+1}\}=0, \tag{13}$$

Substituting $\{\dot{T}_{k+1}\}$ from equation (12) into equation (13) yields:

$$\left(\frac{1}{\theta\Delta t}[C]+[L]\right)\{T_{k+1}\}=[C]\left(\frac{1}{\theta\Delta t}\{T_k\}+\frac{1-\theta}{\theta}\{\dot{T}_k\}\right), \tag{14}$$

Once $\{T_{k+1}\}$ is obtained, $\{\dot{T}_{k+1}\}$ is updated using equation (12) by means of iteration. In a nonlinear analysis, the Newton-Raphson method is employed along with the equation (12).

4 SOLUTION

4.1 Input data

For the simulations the input data from the site were not available. Therefore the typical values of individual concrete / sub-base properties were taken from literature and also web sources. Following material characteristics were applied in the modelling:

Concrete [5]:	λ = 1,4 W/m/K
	c = 900 J/kg/K
	α = 14 W/m^2/K
	ρ = 2300 kg/m^3
Water bearing layer [3]:	λ = 0,6 W/m/K
	c = 4186 J/kg/K
Foundation [3] - slates:	λ = 1,9 W/m/K
	c = 770 J/kg/K
	ρ = 3500 kg/m^3

The boundary condition on the surface of the concrete slab is represented by air temperature taken from the monitoring between January 2012 and March 2012 (Fig. 5). The boundary condition for the sub-base temperature (1st type) goes from long term measurements of Czech hydrometrological institute, the temperature is taken into account as 10,5 °C in the depth of 6 m below the slab surface.

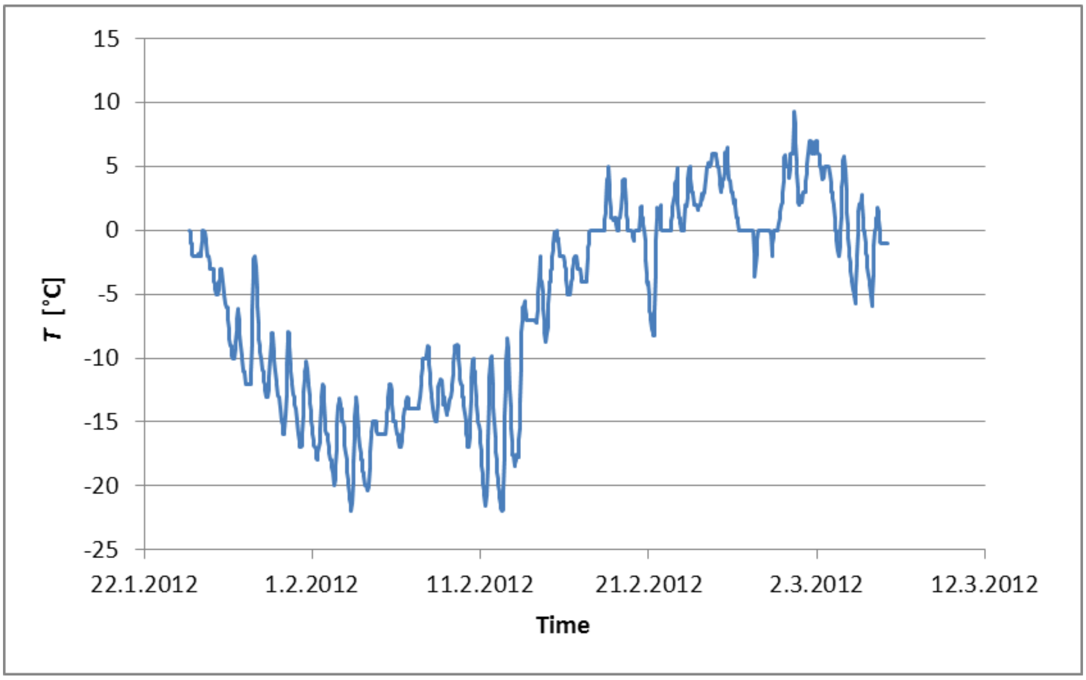

Fig. 5 Air temperature January 2012 – March 2012

4.2 Numerical solution of 1D-model

The solution of the equations (5) and (6) was performed using the MS Excel software. The spatial step Δy was 0.1 m, the length L of the model domain was 6 m and the time step Δt was 1 hour = 3 600 s. The initial condition (Fig. 6) was determined from steady-state analysis with boundary conditions T_A 0 °C and $T/\Gamma 1$ 10,5 °C.

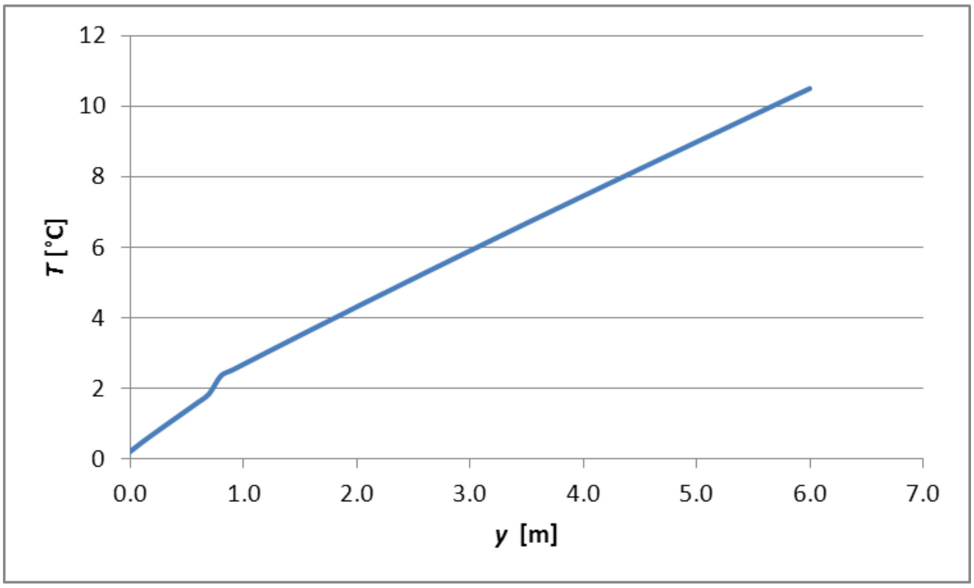

Fig. 6 Temperature distribution over the section

4.3 Numerical solution of 2D-model

For the 2D numerical analysis the PLANE 55 element was chosen in the ANSYS commercial FEM software package. PLANE55 can be used as a plane element with a 2-D thermal conduction capability. The element has four nodes with a single degree of freedom, temperature, at each node. The flow domain is shown in Fig. 4. The initial condition expresses temperature distribution at the beginning of the calculation and was taken from the steady-state analysis with boundary conditions T_A = 0 °C and $T_{/\Gamma 1}$ = 10,5 °C.

5 RESULTS

The results from the 1D model are obtained in the nodes placed on the chute vertical axis in different distances from the top surface of the slab. The comparison of results obtained from both 1D and 2D model are shown in Fig. 7. The diagram shows, that the 1D explicit model (dashed lines) when compared with the 2D implicit solution provides higher temperatures during the air temperature drop. In opposite, when the air temperature is rising, the 1D model gives lower temperatures. These differences are given partially by the chosen mathematical model (1D, 2D) and dominantly by the numerical scheme (explicit, implicit). 2D thermal analysis better describes the conditions in the chute cross section and also the numerical scheme is much more stable than 1D explicit one. Figure 8 shows us the temperature distribution over the 2D flow domain on 14. 2. 2012 when the lowest temperature -6 °C was reached at the depth y = 0.7 m (here and also in the figure 7 the y coordinate expresses the depth below the slab surface and corresponds to the 1D model).

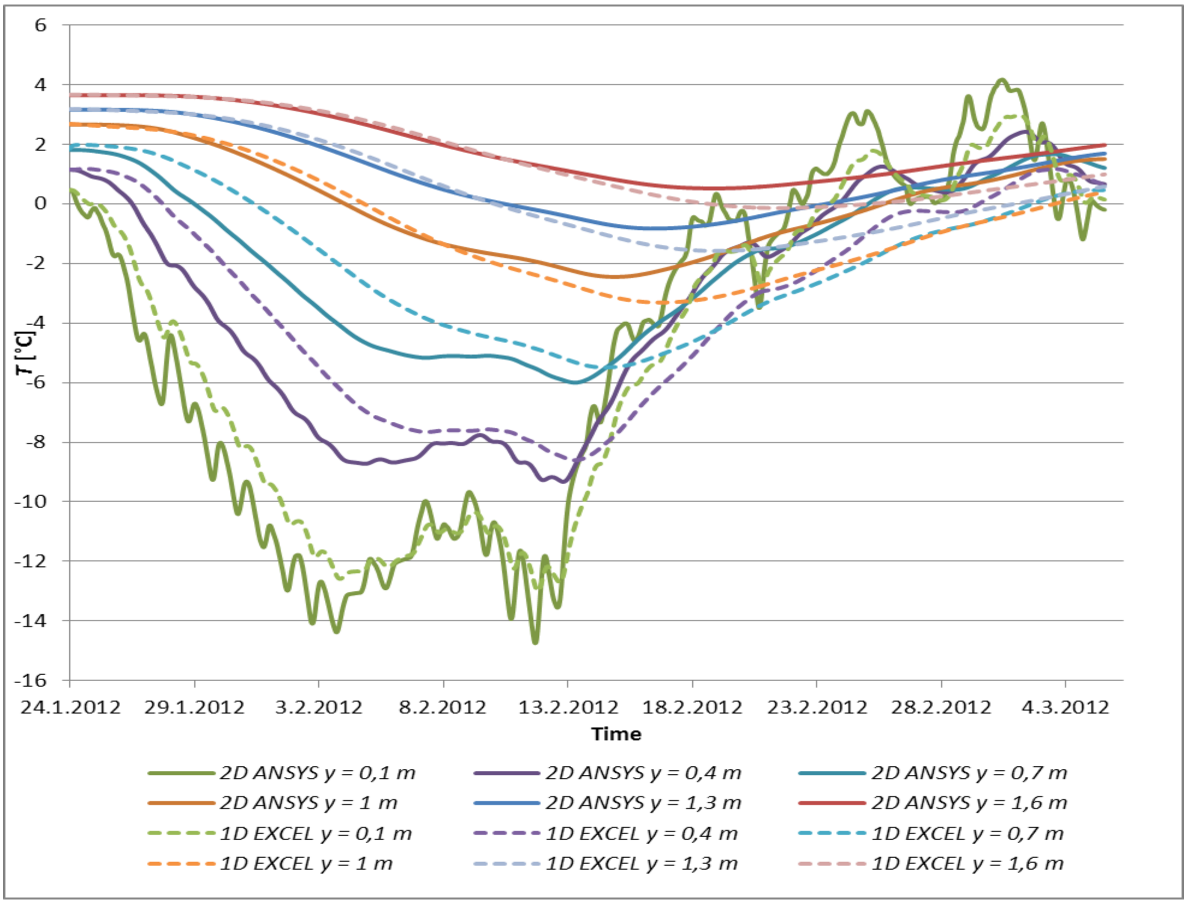

Fig. 7 Resulting temperatures over time (y coordinate corresponds to the 1D model)

From the figure 8 it can be seen that 1D model can be quite good approximation of the conditions in the chute axis.

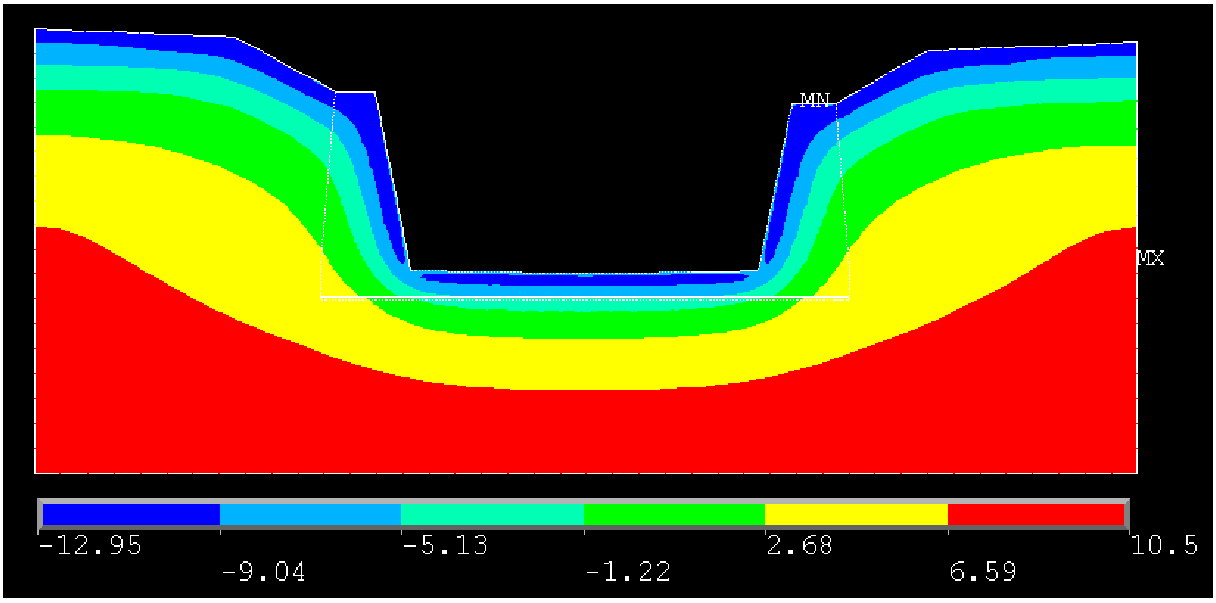

Fig. 8 Temperature distribution over the 2D flow domain (date: 14. 2. 2012)

6 CONCLUSIONS

In this study, the analysis of freezing of concrete slab is demonstrated. Two types of models were applied, the comparison of results was carried out. The results of the comparison are discussed in the chapter 5. The results show that the slab was frozen through due to its relatively small thickness and quite long duration of the period with temperatures below 0 °C. The water bearing layer below the slab was exposed to zero temperatures for approximately 28 days. With respect to the expansion of water when frozen, the freezing of foundation joint may be the probable cause of vertical displacements of the slab.

Acknowledgements

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EXTRAPOLATION OF ROCK MASS PROPERTIES FOR HYDROTECHNICAL TUNNELS USING EMPIRICAL –STATICAL -DYNAMICAL (ESD) METHODOLOGY

Z. Zafirovski[1]

Abstract

It is well known, that process of investigation in rock masses as a media in interaction with engineering structures is extremely difficult. This is especcicaly important in a process of design of hydrotechynical tunnels. More precisely, the main problem is how to extrapolate the parameter from the zone of testing to the whole volume that is of interes for interaction analyses of the system rock mass-structure. So, in a frame of this article, so called Empirical-Statical-Dynamical (ESD) methodology of extrapolation is presented. The basis of the methodology lies in combination of the results from geotechnical and geophysical testings and rock mass classification, connected with definition of adequate regressive models. The practical importance of this problem is explained trought several case histories for imoprtant hydrotechnical tunnels in R.Macedonia.

Keywords

Classification, deformability, extrapolation, geotechnical model, rock complexes

1 INTRODUCTION

During the design process for tunnels in hydrotechics, one of the main problems is how to extrapolate the deformability and shear strentgh rock mass parameters from the zone of testing to the whole area (volume) of interes for interaction analyses between structure abd natural environments.

The extrapolation procedures are innitally developed for design problems at large dams by Kujundžić [2], [5], but later is constantly expanded oby Lokin and Čolić 1980, 1990 and 1996; Lokin, Lapčević, Petričević, 1989; Čolić, Manojlović, 1983; Jovanovski, Gapkovski,

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1998; Jovanovski et al., 2000; Jovanovski, Gapkovski, Ilijovski, 2002, 2003, 2004; Ilijovski, Jovanovski, Velevski, 2004; Ilijovski, 2005 etc.
Contribution to defining deformability and shear strength of rock massif through empirical failure criteria is given by Hoek and Brown, 1980, 1983, 1988.
Classification systems developed in the field of rock mechanics that need to be highlighted are Geomechanical Classification – Rock Mass Rating system (Bieniawski, 1970, 1973, 1974, 1975, 1976, 1979, 1989); RSR - Rock Structure Rating (Wickham, Tiedemam and Skinner, 1972, 1974); Q system - Rock Mass Quality (Barton, Lien and Lunde, 1974);
Computers development in recent decades has contributed to the development of numerical calculation method in rock mechanics which enabled new and wider possibilities of stress and deformation calculation. This had significantly stimulated the development of rock mechanics and tunneling as scientific and technical discipline as well as the wider application of research results into practice.
So, this article describes a methodology that shows how it is possible to integrate all these approaches in a problem for extrapolation of the parameters for hydrotechical tunnels.

2 METHODS OF ANALYSES

Limitations in a process of investigation in rock masses comes from the fact that the whole tunnel length can not be completely covered with detailed geological and geotechnical investigations. So, it is necessary to find a way to extrapolate the necessary parameters from smaller volume of testing zone to the whole volume of the rock mass along the tunnel length.
The given approach in a frame of this article can be defined as Empirical–Static-Dynamic (ESD) methodology of extrapolation. The prerequisite for using this methodology is following:

1. To have enough data for reliable rock mass classification.
2. To have enough testing data for deformability with static tests.
3. Whole structural zone (in this case tunnel) to be covered with geophysical seismic tests.

Such testing must be performed in a manner that will insure reliable data for geotechnical modeling of the whole area along the tunnel length.
One of the key problems here is to divide tunnel length in so called quasi-homogenous zones with relative uniformity of the deformability and shear strength rock mass properties as basic and constitutive elements of geological model. Inside such zone some conditions or properties are the similar in every point, and very different outside it. Each and every zone is determined by space limits and consists, in some way, properties which are important for the study.
It shall be noted, that the process of extrapolation is strictly connected and interrelated to the process of geotechnical modelling of the terrain. The complex geotechnical model is consisted of three basic models (Pavlović, 1995, 1996):

- model of natural geological environment;
- model of engineering activity - geotechnical model in narrow sense (GM);
- model of interaction - model of stress-strain behavior.

A flowing-chart which shows the connections between each model is presented in Figure 1.

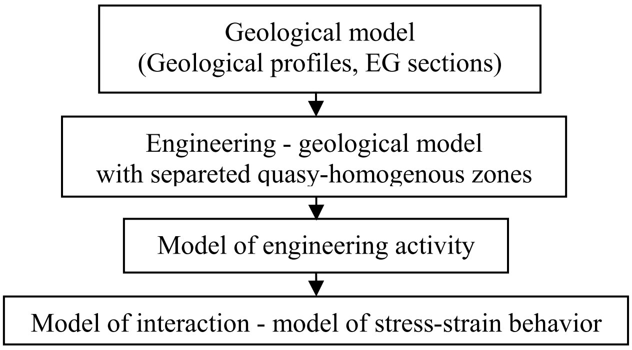
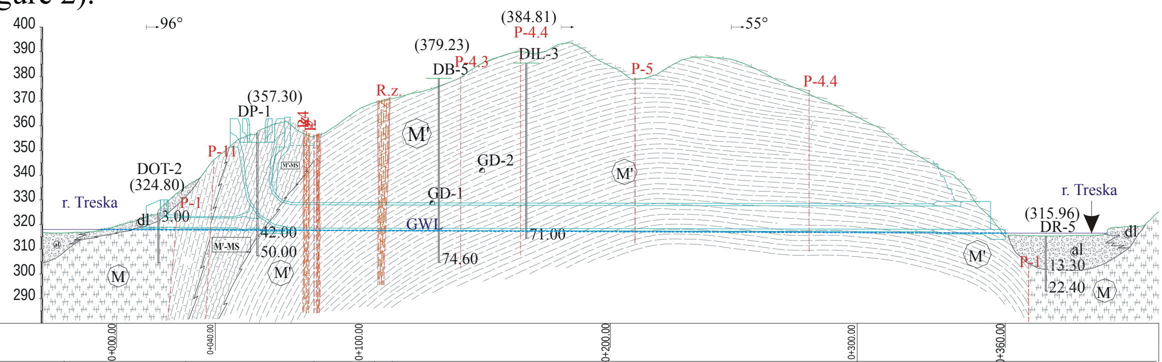


Fig. 1 Steps for connection between different models

It can be underlined that the model of engineering activity and the model of interaction are final phases of geotechnical modelling.

3 RESULTS

To illustrate the methodology, one example is shown for the hydro technical tunnel constructed for the area of arch dam “Sveta Petka” on a river Treska in Republic Macedonia (Figure 2).



Zone	A	B	C	D	E	F
D [MPa] E [MPa] V_p [m/s]	D=3900 - 4500 E=8900 - 12000 Vp=4000-4600	D = 4900-5800 E = 11000-14000 Vp=4600-4700	D=4900 E=11000 Vp=4700	D = 700-800 E = 1600-1700 Vp = 2600	D = 6500-7500 E = 15000-17000 Vp = 4800-5000	D = 3200-3500 E = 7000-8000 V_p = 3600
(Q-value)	1.46	1.58	1.80	0.40	1.66 - 2.0	1.33
(RMR-value)	49	52 - 56	54	21-25	57	48

Fig. 2 Engineering Geological Section for hydro technical tunnel for arch dam "Sveta Petka" in R.Macedonia: M-marbles; M’-foliated marbles; al-alluvial deposits; GWL-groundwater level; DP-1-boreholes; GD-investigation galleries; Rz-fault zones; P-faults; D-deformation modulus; E-elasticity modulus; Vp-value of longitudinal seismic waves defined with geophysical tests; Q-value of rock mass quality after Barton et all; RMR-Rock Mass Rating after Bieniawski.

To define this Engineering-Geological Section, the following methodology of investigations is used:

1. Collection of data for rock massif test results, particularly laboratory and field test results of strength, deformation, discontinuities and other parameters.
2. Specific laboratory and field testing for a specific purposes.
3. Statistical analysis of the tested parameters.

After that, all of the results from geological, geotechnical and geophysical investigations were used for establishing physical model through the RMR, Q and GSI classification. Correlations between the quality of rock massif (RMR, GSI and Q indexes), dynamic (Vp) and static properties (D and E) of rock masses are expressed using results of the detailed classification of the rock massif around the measuring point with dilatometer testing's. Typical deformability diagrams from dilatometer tests are given in Figure 3.

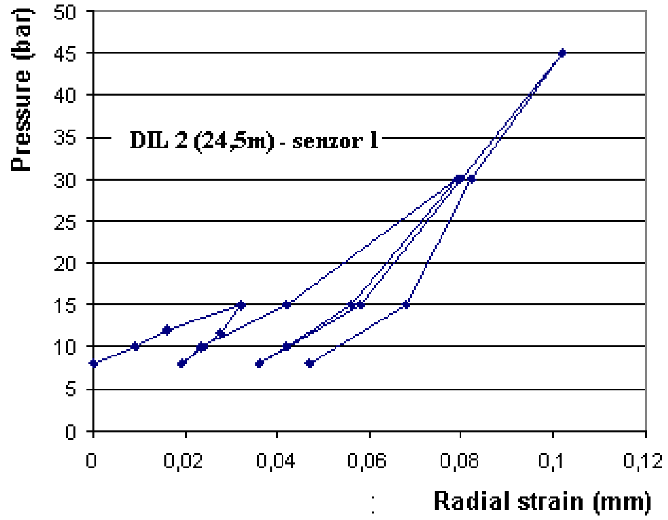


Fig. 3. Typical diagram from dilatometer testing for a Rock Mass with low rating (RMR=20-25)

Diagrams shown on Figure 3 are basic for estimating of deformation modulus and elasticity modulus and imply not only to its' value than also to dependence of the modulus on pressure itself, so the point is rock mass „strenghthening“ or „softening“ regarding to pressure.

Based on detail analyses, a numerous regression models are obtained in order to fulfill the necessary criteria for extrapolation. Here, several type of regression models can be used. For example, some regressive dependances between quality of rock mass *RMR* and velocities of longitudinal elastic waves *v* and deformation modulus (D) for several locations (between them for dam „Sveta Petka” is presented in Figure 4 and Figure 5.

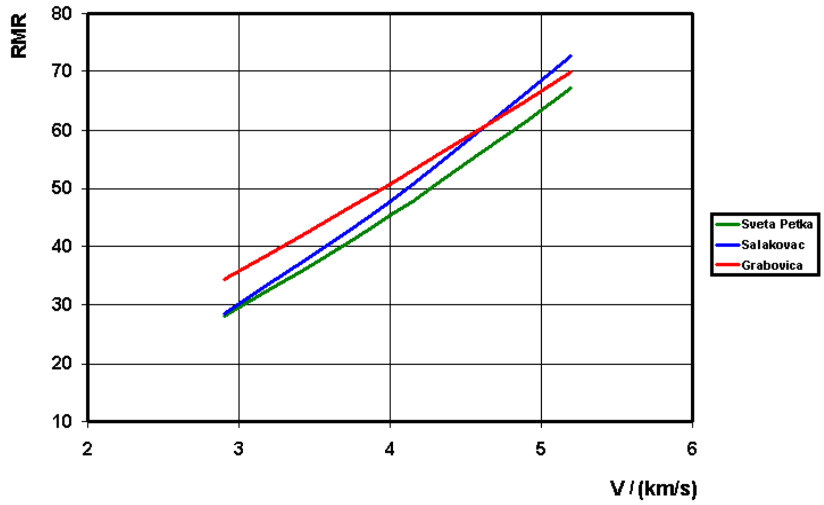


Fig.4. Regression curves between quality of rock mass RMR and velocities of longitudinal elastic waves v_l from the location on "Salakovac" dam $RMR=9{,}8519 x v_l^{1{,}1721}$ and "Grabovica" dam $RMR=9{,}4537 x v_l^{1{,}2179}$ with correlative dependances fort the „Sveta Petka" dam $RMR=5{,}6848 x v_l^{1{,}4979}$

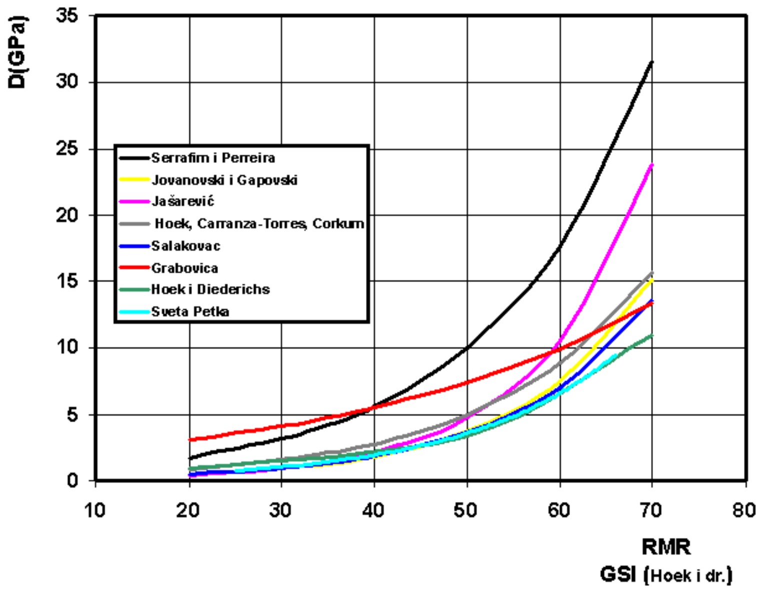


Fig.5. Correlative dependences between quality of rock mass RMR (GSI) and deformation modulus D from the location on "Salakovac" dam $D = 0{,}1369\,x\,e^{0{,}0657\,RMR}$ (GPa), „Grabovica" dam $D = 1{,}6963\,x\,e^{0{,}0295\,RMR}$ (GPa) and "Sveta Petka" $\mathrm{D = 0{,}1104\,x\,e^{0{,}0703RMR}}$ dam with known correlative dependences from literature [2], [5].

4 DISCUSSION

Advantage of analytical models showed in Figures 4 and 5, is that can make predictions and extrapolations relatively quickly and accurately and in that form is very appropriate for practical using. Whereat, when prediction of the parameters is made in this way, it needs to mention that different in situ test are made with different levels of vertical stress, for different strenghts, anisotropy and etc. It is clear that if we combine empiric an field's methods, we can succesfully cover a lot of cases which are important for project analysis, but it is also clear that examples on the figures always have to be carefully used, reexamined and attentively engaged in geotechnical models.

5 CONCLUSION

The presented empirical–static–dynamic method for data extrapolation can be very useful tool in preparation of geotechnical models for numerical analyses in tunneling. It can be mentioned, that aanalytical models for prognosis of possible intervals of deformation modulus *D* are useful as input data in numerical analysis for relatively shallow tunnels.

It is important to underline, that the process of modelling must correspondence with design phases. Results from initial models in first design phases for a level of Preliminary Design, can indicate the need for new data in next phases, and this, in the other hand, influences the improvement of models or leads to new ideas for new model types.

We can conclude that there are many possibilities for further researches in this area. The purpose is to improve and confirm the methodologies suggested in this article, yet not only when it comes to tunnelling but also for other types of structures.

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