RESULT COMPARISON OF SIMULATIONS OF DESIGN DISCHARGES USING ENERGY LOSS EQUATION AND DYNAMIC WAVE APROXIMATION

POROVNÁNÍ VÝSLEDKŮ SIMULACÍ NÁVRHOVÝCH PRŮTOKŮ UŽITÍM METOD ROVNICE ZTRÁTY ENERGIE A DYNAMICKÉ VLNOVÉ APROXIMACE

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Abstract

The flood risk is still a very serious problem, especially in urban areas. Man's knowledge in the fields of hydrology, water management and also information and geoinformation technologies develops very quickly, so there are many techniques of the flood risk assessment. Ones of the most effective and actual tools of the hydrological practice are hydrological models. Nowadays, there is a large number of these models and the most advanced ones offer a number of several computation hydrological techniques. The aim of this contribution is to compare the results of the design flood waves moving through a river computed using two different hydrological techniques, the energy loss equation and dynamic wave approximation.

Abstrakt

Vsoučasné době je hydrologům k dispozici celá řada technik a nástrojů umožňujících širokou paletu hydrologických výpočtů. Mezi velice efektivní patří hydrologické numerické modely, které se v souvislosti s rychlým technologickým rozvojem společnosti stávají čím dál tím vice aktuálními. Aplikační možnosti v rámci hydrologie jsou pak v případě těch nejkomplexnějších a nejpokročilejších modelovacích produktů velice pestré a tyto modely často integrují pro analýzu určitého hydrologického fenoménu několik různých metod současně. Cílem tohoto příspěvku tedy bylo porovnání výsledků modelování pohybu návrhových povodňových vln dvěma různými výpočetními metodami, a to rovnicí ztráty energie a metodou dynamické vlnové aproximace.

Key words: energy loss equation, dynamic wave, design discharge, hydrodynamic model

1 INTRODUCTION

Water is one of the most important components of the environment and its function in the landscape is not substitutable. Overall said, water is due to its exceptional physiochemical properties the main medium of the landscape metabolism. There are two aspects, how to look at water – water as an irreplaceable source or water as an element. But all the human activities in the landscape are strongly limited by the existence of water there.

The role of water as a source is crucial and due to population growth and very fast technological progress man needs still more and more water to satisfy his requirement. Thus the optimization of supplies and rational exploitation of water belong to the most actual social, scientific and technological problems of our time [1]. By the aspect of source the quality and quantity of water are both determinative. By the aspect of element, more important than the quality is the quantity and dynamics of water in the environment, particularly during extreme events like floods or drought.

This contribution concerns only the first said extreme events, which are floods. As is said, in the Czech legislative the flood is a temporary increase of the water stage in watercourses or other surface water recipients, during which water inundates the areas of the floodplain and may cause some damages [2]. The important part of the previous citation is that which is saying that during the flood water flows outside the riverbed. Ever before human activities in the landscape were closely related to the water environment. On our conditions of very intensive land use and quite high density of population that causes the rapid (sub)urbanization growth, the problems of flood protection are still actual especially in urban areas, which are very often situated just in the natural inundations and floodplains. As Cílek broadly says [3], the flood memory is short.

The advantage of our time is a very quick technological progress in all spheres of our life as well as in the science and applications related to the hydrology, water management or hydrological prognosis. Hand in hand with the progress in the field of geoinformation technologies the practical application of a numeric hydrological model is more and more common. Generally, we can divide the hydrological models into two groups according to the simulated hydrological phenomenon transformation. The first group is the group of rainfall-runoff models that solve the hydrological transformation of precipitation in a catchment. The second group includes the hydrodynamic models, which simulate the hydraulic water mass transformation in a riverbed [4].

The aim of this contribution is some of the options of the hydrodynamic modelling. The concrete goal of the contribution was to compare the results of the simulations of the design discharges in the riverbed of the Stonávka River using two different computational methods, specifically the energy loss equation and the model of dynamic wave put in differently dynamic wave approximation. All the works were done using two modelling software applications, the hydrodynamic models HEC-RAS and MIKE 11. The HEC-RAS model uses the energy loss equation, the MIKE 11 model is capable to work with more different computational methods including the dynamic wave approximation. Theoretically, all the computations could be done using only one modelling software application, in particular the MIKE 11 model, but due to the temporal license unavailability of its full version the model HEC-RAS had to be used. Because of that the basic methodological precondition used the exactly identical schematizations of the study area.

Another precondition was the same input hydrological data, thus the several design discharges were used as an initial condition at the upper cross section and in both cases only a steady flow was concerned.

2 STUDY AREA

As the study riverbed the 6 km long bottom reach of the Stonávka River was chosen. The reason for this selection was the data accessibility. Since only a limited number of geodetically surveyed cross sections was available, the schematized river section can be in some measure considered as only fictional one. But to make the picture complete there is a brief description of basic conditions of the studied area below.

The Stonávka River is the sinistral tributary of the Olše River, thus it is the third order river. The Stonávka River springs on the northern slopes of the Moravskoslezské Beskydy Mts. The elevation of its spring is about 750 m above sea level, its confluence with the Olše River is situated broadly 220 m above sea level in the cadastre of the town of Karviná. The river length is approximately 33 km and the basin area about 131 km^2.

The dynamics of the flow is dictated by the gradient of the northern nappe slopes of the Moravskoslezské Beskydy Mts. and in the upper part the river has the character of white water. The gradient conditions of the lower parts of catchment are considerably milder and furthermore the hydrological regime of this part is strongly influenced by the existence of the Těrlicko water reservoir. Between the dam and the confluence with the Olše River the riverbed is of a meander morphology and flows through a flat and not very wide valley with steep side slopes. The floodplain is covered with discontinuous built-up areas of the Stonava village. The natural runoff conditions of this area are also influenced by the coal mining activities [5].

3 THEORETICAL BACKGROUND OF HYDRODYNAMIC MODELLING

As mentioned earlier, the main purpose of hydrodynamic modelling is a water mass transformation in riverbeds, both natural and artificial. In other words, hydrodynamic models can be called also as hydraulic models. For better understanding the following simplified explanation can be useful. The inputs of the hydrodynamic models can be discharges (water stage or another hydrologic parameter) considered at the cross section *A* in the time *t* and then the outputs of these sort of models can be discharges (water stages or another hydrologic parameters) computed at the cross sections *B*, *C* and others down the stream in the time *t+n*.

The water flow in rivers has mostly a turbulent flow character, so it happens in three directions, but the most dominant is the water movement in the longitudinal stream profile direction. According to the capability of the models to simulate the water movement in riverbeds in different directions the models can be separated into three groups, which are 1D models (capable to simulate the water movement only in a longitudinal stream profile direction), 2D models (capable to simulate the water movement in two horizontal directions) and 3D models (capable to simulate the water movement in 3 directions). Both used models, HEC-RAS and MIKE 11, are one- dimensional models. MIKE 11 also offers a quasi 2D flow modelling. [4]. By the time aspect of the flow velocity stability the water flow can be divided into two types, which are the unsteady and steady flow. In this paper only 1D steady flow was considered.

3.1 Used mathematical apparatus

3.1.1 Energy loss equation

Basic computational mechanism of this equation is based on the Bernoulli's and Manning's equations. The computation of water stages or flow velocities at the cross sections is given by the equations of the following forms [4], [6], [7]:

$$Y_2 + Z_2 + \frac{\alpha_2 v_2^2}{2g} = Y_1 + Z_1 + \frac{\alpha_1 v_1^2}{2g} + h_e \tag{1}$$

where:

Y_1, Y_2 - depth of water at cross sections [m] ,

Z_1, Z_2 - elevation of main channel invert at cross sections [m],

α_1, α_2 - velocity weighting coefficients [-] ,

v_1, v_2- average velocities at cross sections (total discharge/total flow area) $\left[\frac{\mathrm{m}}{\mathrm{s}}\right]$,

g - gravitational acceleration $\left[\frac{\mathrm{m}}{\mathrm{s}^2}\right]$,

h_e - energy head loss [m] .

For h_e the following is considered [4], [6]:

$$h_e = L\bar{S}_f + C\left|\frac{\alpha_2 v_2^2}{2g} - \frac{\alpha_1 v_1^2}{2g}\right| \tag{2}$$

where:

L - discharge weighted reach length [m],

$\bar{S}_f$ - representative friction slope between two sections $\left[\frac{\mathrm{m}}{\mathrm{m}}\right]$,

C - expansion or concentration loss coefficient [-] .

The form of the Manning's equation for the flow velocity is as follows [4], [6]:

$$v = \frac{c_m}{n} R^{\frac{3}{2}} S_f^{\frac{1}{2}} \tag{3}$$

where:

c_m - coefficient liable to unit system (SI c_m =1, US c_m =1.49) [-] ,

n - Manning´s roughness coefficient [-],

R - hydraulic radius [m],

S_f - friction slope $\left[\frac{\mathrm{m}}{\mathrm{m}}\right]$.

To simulate the flow using that method the model HEC-RAS was used.

3.1.2 Dynamic wave approximation

The model of dynamic wave is based on the complete solution of the one - dimensional Saint-Venant equations and uses the complete momentum equation [8], [4]. The dynamic wave approximation equation assumes the following form:

$$U\frac{\partial y}{\partial x}+y\frac{\partial U}{\partial x}+\frac{\partial y}{\partial t}=0 \qquad (4)$$

$$\frac{\partial U}{\partial t}+U\frac{\partial U}{\partial x}+g\frac{\partial y}{\partial x}-g(S_0-S_f)=0 \qquad (5)$$

where:

x - distance along the channel [m],

y - depth of water at cross sections [m],

U - flow velocity, [ms^{-1}]

t - time [s],

g - gravitational constant $\left[\frac{\mathrm{m}}{\mathrm{s}^2}\right]$,

S_0 - channel slope $\left[\frac{\mathrm{m}}{\mathrm{m}}\right]$,

S_f - friction slope $\left[\frac{\mathrm{m}}{\mathrm{m}}\right]$.

For friction slope S_f for uniform steady flow it can be written [9], [4]:

$$S_f=\frac{n^2|Q|Q}{\mu^2A^2R^{4/3}}=\frac{|Q|Q}{K_c^2} \qquad (6)$$

where:

n - Manning´s roughness coefficient [-],

Q - discharge $\left[\frac{\mathrm{m}^3}{\mathrm{s}}\right]$,

μ - units conversion factor (1.49 for U.S. units and 1.00 for SI) [-] ,

A - cross section area [-],

R - hydraulic radius [m],

K_c - channel conveyance factor [-].

To simulate the flow using that method the model MIKE 11 was used.

4 INPUT DATA

For building hydrodynamic models and following simulations some input data are required. The quantity of input data depends on the quantity and demanding of the performed analysis. It can be generally said that for hydrodynamic modelling the quality of input data is more important than its quantity.

All input data for hydrodynamic models (but also for other hydrologic models including rainfall- runoff models) can be separated into two groups. The first group consists of static data (relative concept according to the time dimension of the modelling) and the second group is represented by dynamic data, consists of hydrologic time-series data. Both groups can be effectively analysed and processed in the GIS environment into a format required by different modelling software applications. These analyses and processes are generally called data preprocessing.

The used input data was mainly collected from these sources: online database VÚV TGM DIBAVOD, digital geographical model ČÚZK ZABAGED, ČHMÚ (Czech Hydrometeorological Institute) and Povodi Odry, s.p. (The River Odra Catchment, State Enterprise). The following data was used as the input data for building the hydrodynamic models:

- Digital elevation model of the Stonávka river basin,
- Vector representation of river network ground plan,
- Geodetically surveyed cross sections,

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- Design discharge data.

The design discharge data consists of design discharges complying with the exceedance probability of 1 year (Q_1), 5 years (Q_5), 10 years (Q_{10}), 50 years (Q_{50}) and 100 years (Q_{100}). They were linked with the nearest hydrological station up the stream, which is called Těrlicko. The discharge values used as an input data are in the table 1.

Tab. 1 Input hydrological data (design discharges of relevant return frequency).

Return Frequency [year]	1	5	10	50	100
Discharge [m^3/s]	27.8	61.2	78.8	127	150

5 MODELS BUILDING AND SIMULATION RUN

In general, the process of building the hydrodynamic models can be divided into several steps. The first step is a preprocessing, in other words a schematization of riverbed, alluvial plain and technical facilities in the river or alluvial plain. The schematization of riverbed, except the technical facilities like bridges, weirs and others, consists in the generation of longitudinal profiles of hydraulic lines that are streamlines and flow routes in riverbed and alluvial plains and the generation of cascade of cross-sections through the riverbed and alluvial plain perpendicularly to the longitudinal profiles. Then the hydraulic parameters like the Manning's roughness coefficient and others are assigned to the schematized lines and other objects.

After the correct completing of the preprocessing, user can start setting up the simulation. In this step the hydraulic character of simulated flow is chosen, the input hydrological data is defined, initial and border conditions are specified. In our case, we were quite limited by the available input hydrological data, so we could simulate only the steady flow.

After the successful completing of the simulation some output data like water stage, flow velocity or discharge is available for particular cross-sections and for longitudinal profile. The data is available in the numerical or graphical formats. The final step of the modelling process is so called post-processing, i.e. processing the output data into maps of flooded area etc.

6 RESULT COMPARISON

The main purpose of this contribution was to compare the results of simulations of steady flow computed with two different computational methods – energy loss equation based on the Bernoulli's equation and the dynamic wave approximation based on the Saint-Venant's equations. The comparison was necessarily executed in two different models (HEC – RAS –Bernoulli, MIKE 11 – dynamic wave), due to a temporal license unavailability, but on the identical schematizations of study area, which was the main condition for possibility to comprise the outputs of the two different models. The study area was covered by 14 geodetically surveyed profiles. Stationing of these profiles is summarized in table 2.

Tab. 2 Cross Section Stationing

Profile ID	1	2	3	4	5	6	7
Profile Stationing [m]	8.66	524.7	975.4	1424.74	2022.92	3076.94	3298.79
Profile ID	8	9	10	11	12	13	14
Profile Stationing [m]	3506.96	3650.1	4004.38	4601.14	4973.45	5628.37	5988.63

As simulated hydrological phenomena the design discharges complying with the Q_1, Q_5, Q_{10}, Q_{50} and Q_{100} were chosen. Due to the different structures of the used computational methods it was expected that the results of the simulations won't be exactly the same, which was testified. For the result comparison two outputs were chosen – water stage and flow velocity in particular cross- sections. As a supporting output the cross-section area was visualised together with the flow velocity. The result comparison follows the same pattern for different designed discharges, thus only the result comparisons for Q_1 and Q_{100} are illustrated.

Looking at the figures 1, 2, 6 and 7 it is evident that the values of the water stages at the cross-sections computed by the dynamic wave model are in the most cases lower than these computed by the energy loss equation. The water stage mean differences at the cross-sections were for Q_1 21cm, Q_5 30 cm, Q_{10} 32 cm, Q_{50} 35 cm and for Q_{100} 38 cm. It is obvious, that with the rising values of initial design discharge the mean differences of water stages computed by the considered methods are rising too. It can be explained by different mathematical structures of the used methods. The energy loss equation does not include any change of momentum between the cross-sections. This equation does not include the parameter operating with the water level change in the segment between two cross sections. On the other hand, the dynamic wave model includes beside the energy equation also the momentum equation. These problems are well analysed in [8].

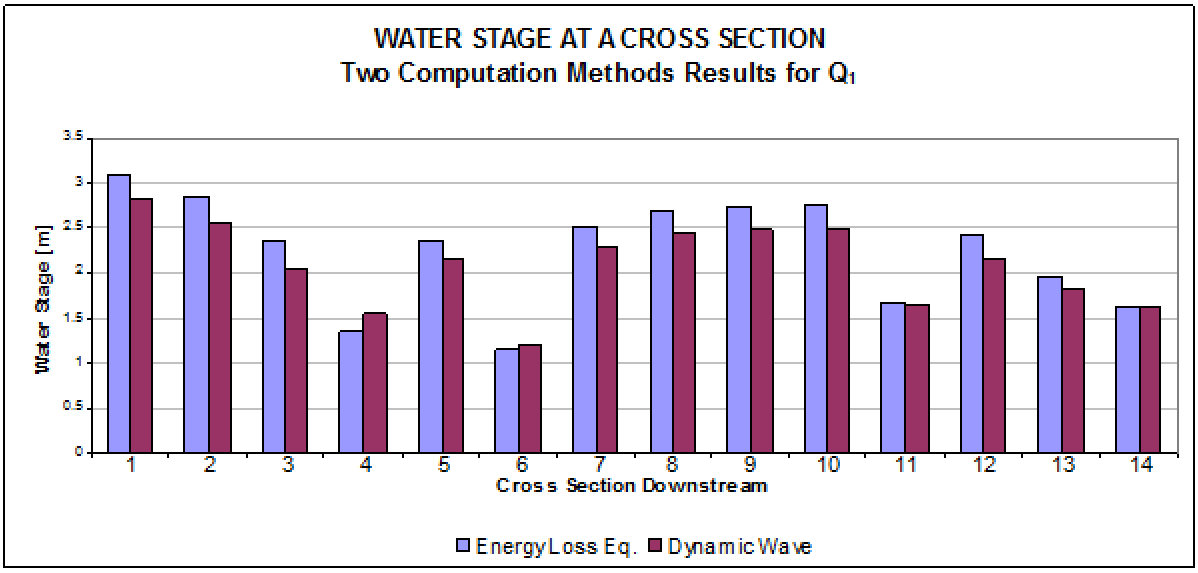

Fig. 1 Comparison of water stage at the cross sections computed by considered methods for Q_1.

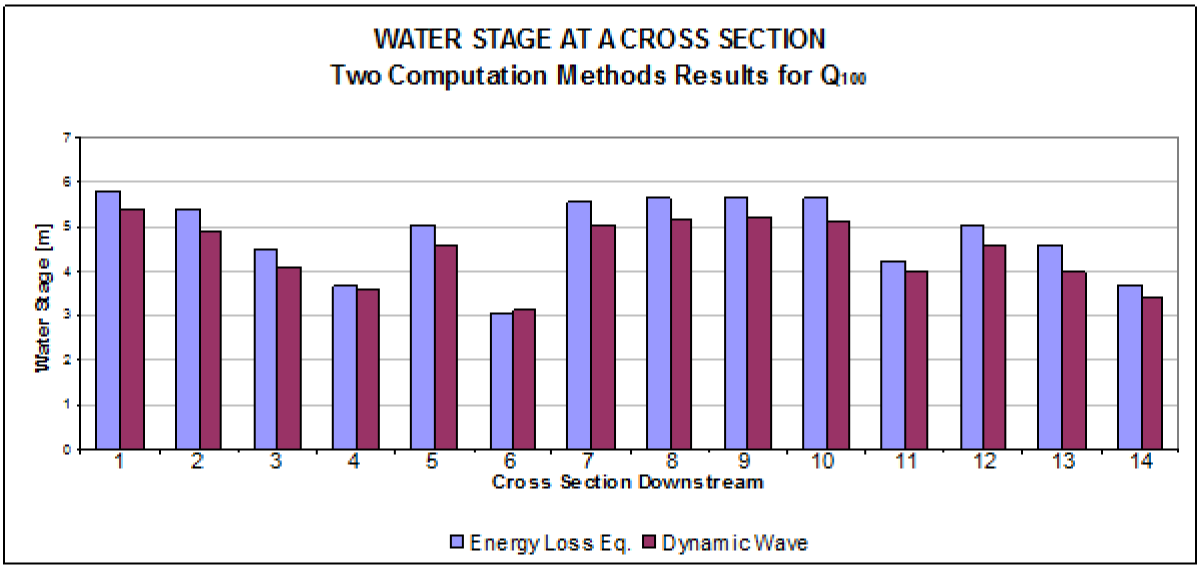

Fig. 2 Comparison of water stage at the cross sections computed by considered methods for Q

The situation of the flow velocity at the cross sections (see fig. 4, 5, 6 and 7) is opposite to the situation with the water stages, so the results of simulation using the dynamic wave are higher than these using the energy loss equation almost at all cross sections. The flow velocity mean differences at the cross sections were for Q_1 0.19 m/s, Q_5 0.20 m/s, Q_{10} 0.22 m/s, Q_{50} 0.28 m/s and for Q_{100} 0.31 m/s. The diagram output of flow velocity at the cross sections includes also the illustration of the cross section areas as mentioned before. The cross section area has a significant influence on the change of flow velocity at cross sections. Due to the flow continuity the flow velocity is higher at the smaller cross sections than at the cross sections with a larger area where the flow velocity is slower.

The characteristics of cross sections are not the only factors affecting the flow velocity. An important factor is also the bottom slope between the cross sections, which can be understood as a change of flow energy. The reaction to the rising bottom slope is the rising flow velocity downstream. Considering the limited number of cross sections available for the river schematisation this could have a significant impact on the flow velocity.

The differences of the considered methods results of water stage and flow velocity at the cross sections are illustrated in figures 6 and 7.

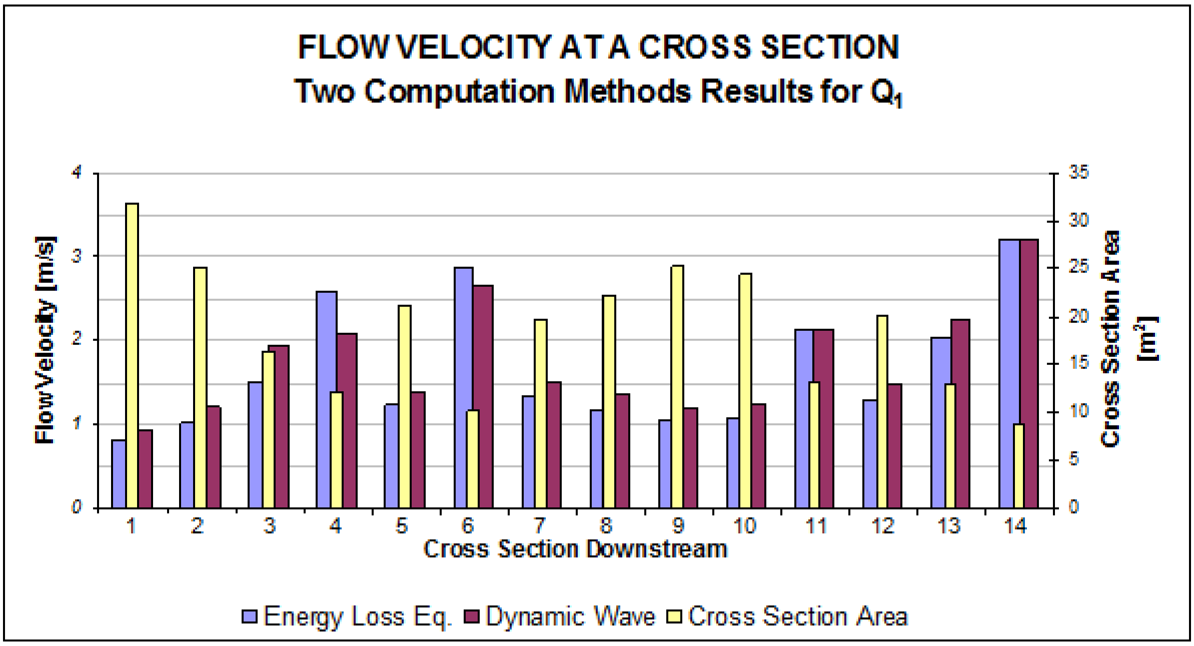

Fig. 4 Comparison of flow velocity at the cross sections computed by considered methods for Q_1.

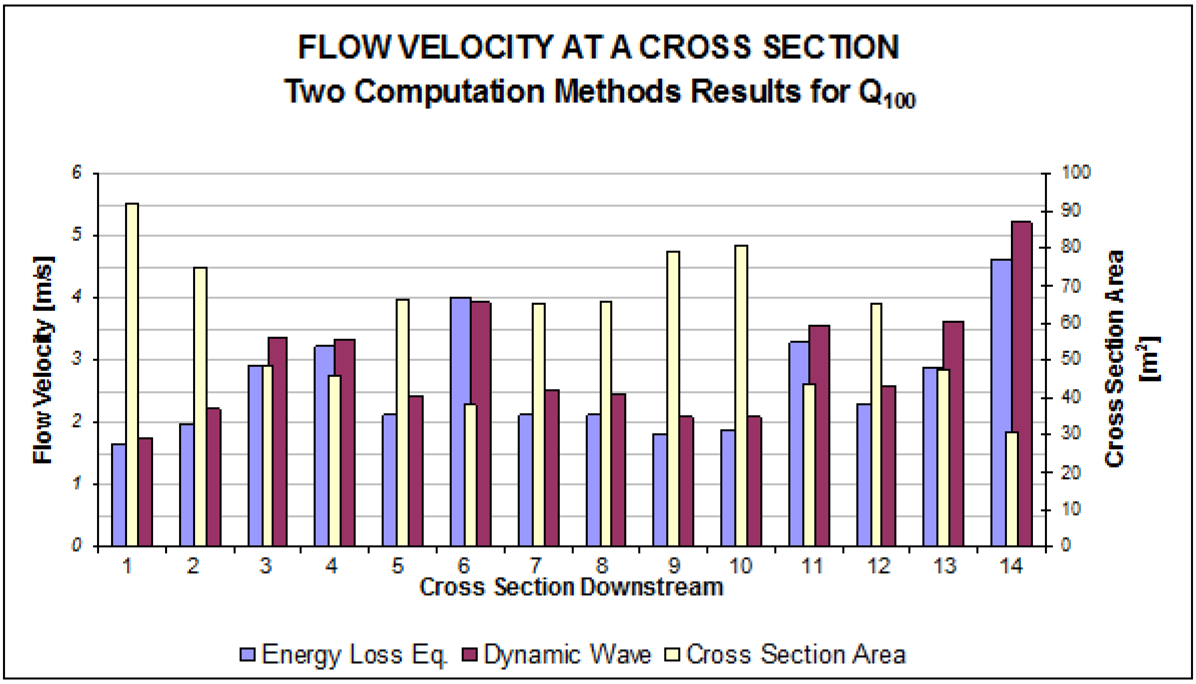

Fig. 5 Comparison of flow velocity at the cross sections computed by considered methods for Q

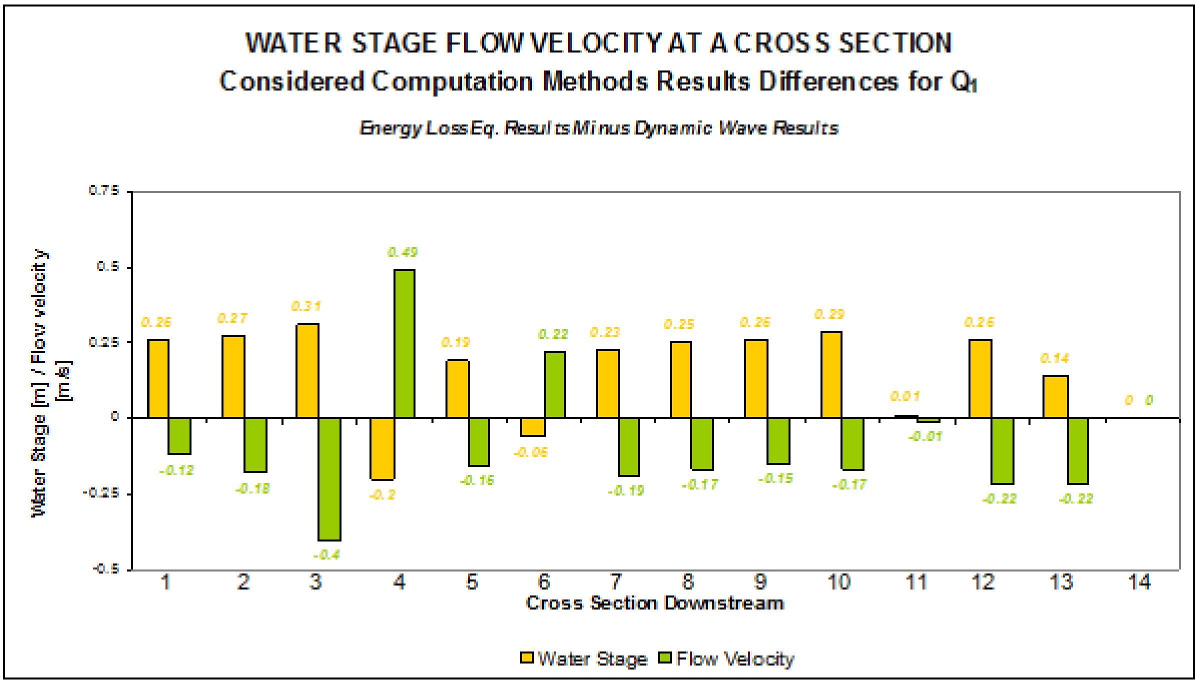

Fig. 6 Comparison of the results differences of water stage and flow velocity at the cross sections computed by considered methods for Q_1.

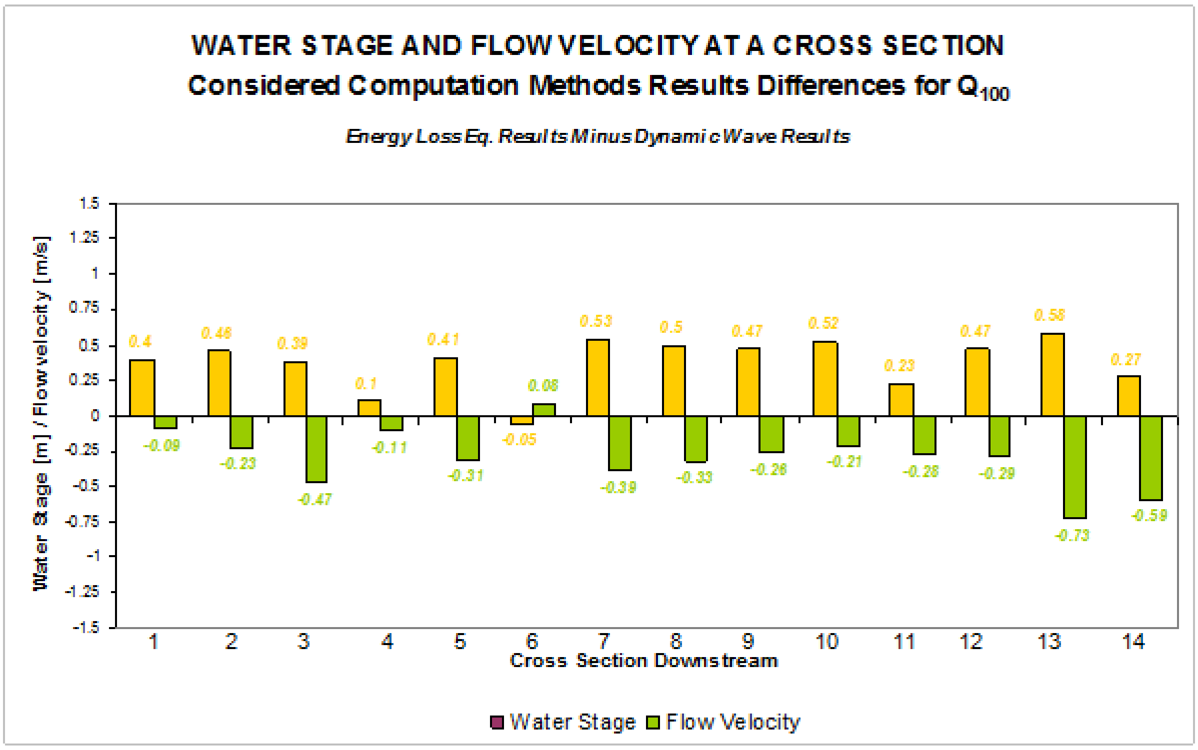

Fig. 7 Comparison of the results differences of water stage and flow velocity at the cross sections computed by considered methods for Q_{100}.

7 CONCLUSIONS

The main purpose of this paper was to compare the results of the simulations of steady flow in the open channel computed using two different hydrologic computational methods. The used methods differ in their mathematical-physical fundamentals. These methods were the energy loss equation based on the Bernoulli's equation and the dynamic wave based on the Saint-Venant's equations. The latter one includes besides the energy equation also the momentum equation. The results given by both methods differ in the water stage values at the cross sections as well as in the flow velocity values. In the most cases, the energy loss equation gives higher water stage values than the dynamic wave, the dynamic wave gives mostly higher values of flow velocity than the energy loss equation.

The confirmation of the initial pre-requisite that the different computing methods will give different results proves that the choice of an appropriate computational tool is very important in the hydrologic practice and the sensitivity of the concrete apparatus to the different conditions is the important factor affecting the output results. Generally, the higher generalization of mathematical equations and neglecting some elements, the lower variety of conditions, to which the considered method is applicable. Suitability of some hydrological methods is described in [8].

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RESUMÉ

V současné době je hydrologům k dispozici celá řada technik a nástrojů umožňujících širokou paletu hydrologických výpočtů. Mezi velice efektivní patří hydrologické numerické modely, které se v souvislosti s rychlým technologickým rozvojem společnosti stávají čím dál tím více aktuálními. Aplikační možnosti v rámci hydrologie jsou pak v případě těch nejkomplexnějších a nejpokročilejších modelovacích produktů velice pestré a tyto modely často integrují pro analýzu určitého hydrologického fenoménu několik různých metod současně. Cílem tohoto příspěvku tedy bylo porovnání výsledků modelování pohybu návrhových povodňových vln dvěma různými výpočetními metodami, a to rovnicí ztráty energie a metodou dynamické vlnové aproximace.

Obě metody byly porovnávány na totožné schematizaci dolního toku řeky Stonávky. K interpretaci výsledků byly využity dvěma metodami simulované výšky hladin a rychlosti proudění v příčných profilech. Co se týče výšek hladin, tak ve většině sledovaných příčných profilech byla hladina vody nižší v případe výpočtu dynamickou vlnou, což platí pro všechny hodnoty N-letostí. Průměrné rozdíly výšky hladiny v příčných profilech byly v případe Q_1 21cm, Q_5 30 cm, Q_{10} 32 cm, Q_{50} 35 cm a Q_{100} 38 cm. Co se týče rychlosti proudění v příčných profilech, tak zde je situace oproti výšce hladin zcela opačná, tedy vyšší hodnoty poskytuje téměř ve všech profilech metoda dynamické vlnové aproximace. Průměrné rozdíly rychlostí proudění v příčných profilech byly pro Q_1 0.19 m/s, Q_5 0.20 m/s, Q_{10} 0.22 m/s, Q_{50} 0.28 m/s a Q_{100} 0.31 m/s.

Potvrzení výchozího předpokladu, že odlišné výpočetní metody budou při výpočtu totožného jevu na identické schematizaci poskytovat rozdílné výsledky dokazuje, že volba vhodného výpočetního nástroje hraje v hydrologické praxi významnou roli a citlivost konkrétního nástroje k daným podmínkám je důležitým faktorem ovlivňujícím výsledky výpočtů. Obecně s rostoucí generalizací matematických vztahů a zanedbáním některých činitelů klesá i rozpětí podmínek, pro které je nástroj aplikovatelný.