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APPLICATION OF NUMERICAL MODEL SIMULATIONS FOR ESTIMATION OF MORPHDYNAMICS AND VEGETATION IMPACT ON TRANSPORT OF DISSOLVED SUBSTANCES IN THE WARTA RIVER REACH

ZASTOSOWANIE SYMULACJI NUMERYCZNYCH DLA OSZACOWANIA WPŁYWU ZMIAN MORFODYNAMICZNYCH I WEGETACYJNYCH NA PRZENOSZENIE DOMIESZEK NA ODCINKU RZEKI WARTY

Joanna Wicher-Dysarz, Tomasz Dysarz

Poznan University of Life Sciences

Abstract. The main problem analysed in this paper is the impact of sediment accumulation and vegetation growth on transport of dissolved substances in a river. The system studied is the reach of the Warta River located upstream of the Jeziorsko Reservoir inlet. The three processes, namely sediment deposition, vegetation growth, and pollutant transport, are crucial for the functionality of reservoir. Classical HEC-RAS package is used for the reconstruction of steady flow conditions in the river reach. The transport of admixture is simulated by means of convection – dispersion model with additional elements describing storage of solutes in the floodplains. The results that the degree of maximum concentration decreases as the river bed geometry and vegetation cover are changed.

Streszczenie. W artykule poruszono problem wpływu akumulacji rumowiska i wzrostu roślinności na przenoszenie domieszek w rzece. Analizy przeprowadzono dla odcinka rzeki Warty zlokalizowanego powyżej dopływu do zbiornika Jeziorsko. Dla funkcjonowania zbiornika istotne są trzy procesy: akumulacja rumowiska, wzrost roślinności i przenoszenie zanieczyszczeń. Klasyczny pakiet HEC-RAS został wykorzystany dla zrekonstruowania ustalonych warunków przepływu na odcinku rzecznym. Przenoszenie domieszek jest symulowane za pomocą modelu adwekcji – dyspersji z dodatkowymi elementami

Corresponding authors – Adres do korespondencji: dr inż. Joanna Wicher-Dysarz, dr inż. Tomasz Dysarz, Department of Hydraulic and Sanitary Engineering, Poznań University of Life Sciences, ul. Wojska Polskiego 28, 60-637 Poznań, e-mail: jwicher@up.poznan.pl, tdysarz@gmail.com.

opisującymi magazynowanie substancji rozpuszczonych na terenach zalewowych. Uzyskane wyniki pokazują, zmiany geometrii i pokrycia roślinnością mają wpływ na zmniejszenie stopnia dyssypacji substancji rozpuszczonej.

Key words: river processes, admixture transport, sediment accumulation, riparian vegetation

Słowa kluczowe: procesy rzeczne, przenoszenie substancji rozpuszczonych, efekty akumulacji rumowiska, efekty wzrostu roślinności nadrzecznej

INTRODUCTION

Most of the pollutants injected into the environment in the rural catchments come from farming and sanitary sewage without admixture of heavy metals. Many of them are chemical substances which are well mixed with water and do not interact with flowing stream [e.g. Sawicki 2003]. Such substances are called passive pollutants or passive dissolved substances, suspended load and biogenic compounds. The term 'passive' is crucial for the mathematical description of the transport process in the river channels and enables splitting of the problem into two separate subproblems namely that of water flow and that of admixture transport [Szymkiewicz 2000].

This paper the transport of the passive admixtures in the river is analysed from the specific point of view. We would like to of sediment accumulation in a river and vegetation growth on the main features of the admixture transport decrease in its maximum concentration and changes in the time of its travel. The analysis was performed with the use of numerical models. The simulations made in this study covered a wide range of different morphodynamic and hydraulic conditions, representative of the selected system studied. The river system used for the presentation of our concepts and testing our hypothesis is the Warta river reach in the inlet part of the Jeziorsko reservoir. The study has been undertaken to analyse changes in the transport parameters in relation to the bed geometry transformations and vegetation growth. The processes listed above as sediment deposition, vegetation growth and pollutant transport, are crucial for the functionality of the reservoir.

The paper is divided into six parts with this Introduction as the 1st part. In the second part the system studied is briefly presented. The models used and the basic methods implemented are discussed in the third part. The next part presents two elements. These are (1) a relation between the models discussed in the previous part and (2) the representative morphodynamic and hydraulic conditions. The fifth part includes selected results and their discussion. In the last part conclusions are presented.

The main idea of the study presented is to run a number of simulations representing transport of hypothetical admixture in different conditions

THE REACH OF THE WARTA RIVER ANALYSED

General information

The Warta is the greatest river in the Oder River catchment. It is also the third biggest river in Poland, after the Vistula and the Oder. Its length is 808.2 km and total watershed area is 54528.7 km^2. One of the most interesting hydraulic in the Warta river is the

Jeziorsko reservoir located in the centre of the river course. The Jeziorsko dam is located km 484,300 and the inlet part of the reservoir is between Km 503,560 and km 502,000. River stations located nearby are Sieradz (km 521,000), Warta town (km 503,750) and Uniejow (km 466,000). The Sieradz station is located beyond the influence of the Jeziorsko reservoir. The measurements and observations in the Sieradz station are the most relevant ones to assess the natural conditions in the area under investigation. Hence, the Sieradz station data are used to characterize the general hydrological conditions related to the discharge variability.

Are annual and total values, extremes as well as averages, for the period of 1963–1983 and 1993–2001. The average discharge is 51.94 $m^3 \cdot s^{-1}$, but the discharge variability ranges from 10 $m^3 \cdot s^{-1}$ to 440 $m^3 \cdot s^{-1}$ (fig. 1). One of the characteristic flows taken into account in the analyses is also the average of annual maximum flows (average maximum) equal to 181.56 $m^3 \cdot s^{-1}$.

PROCESSES IN THE INLET PART OF THE JEZIORSKO RESERVOIR

The Jeziorsko reservoir is a relatively new. It was built in 1986 in the central part of Poland. The reservoir is located in the Warta River, between the Sieradz (upstream) and Uniejów (downstream) stations. For the first time it was filled up to the admissible maximum water level (121.5 m a.s.l.) in 1991. The hydropower plant was put into operation in 1995. The total admissible minimum and maximum water levels in the reservoir are 116,0 m a.s.l. and 121.5 m a.s.l., respectively. The Jeziorsko reservoir and the Warta River form very interesting water system. The three main processes which make the system non-stationary are: reservoir performance, sediment deposition, and vegetation growth. The processes are briefly discussed in the section. Their impacts on the hydraulic conditions in the backwater part of the Warta River is emphasized.

The effect of the reservoir existence and performance is very simple. Increase in water levels due to the reservoir operation causes an increase in the water levels in the upstream backwater part of the river. The increase in water levels makes flow velocities decrease. This phenomenon is compatible with basic mass balance giving the relationship between discharge Q, cross-section area A and average velocity U in the form $U = Q/A$. If area A is larger due to the increase in water levels, the velocity U must be smaller. Changes in the velocity patterns foster the process of sediment deposition observed to occur from the very beginning of the reservoir performance. The most intensive sediment accumulation occurred in the river reach below the bridge in Warta town, between Km 503,000 and Km 500,000 [Wicher-Dysarz and Przedwojski 2005, Dysarz et al. 2006, Dysarz & Wicher-Dysarz 2011]. It is clearly visible that accumulation of the sediment during the period of reservoir existence totally transformed the river channel causing additional increase in the water level.

Additionally, the increase in the river bed levels causes also an increase in the water level, leading directly to deeper and longer inundation of floodplains. The mechanism described seems to the area protected by dikes in the upper part of the reservoir. In 1998 the results of field measurements made along the river reach from Km 494,330 to Km 520,850 confirmed an important decrease in the water surface slope [Wicher-Dysarz and

Przedwojski 2005, Dysarz et al. 2006, Dysarz and Wicher-Dysarz 2011]. The decrease in the water level slope was caused by a significant increase in the bottom level in general. Such changes affect the flood risk related to dike break or overtopping. It is expected that this risk is increasing.

Nature reserve and its impact on water management

The sediment transport and accumulation processes described foster vegetation growth in this study area. Rising flooding frequency enabled establishment of a nature reserve for birds in the upper part of the reservoir in 1998 [Wicher-Dysarz and Przedwojski 2005, Dysarz et al. 2006, Dysarz and Wicher-Dysarz 2011]. Since this time an important of the reservoirs been purpose conservation of water conditions sufficient sustaining for biological life in the reserve area. Because of the existence of the nature reserve and of the Environment Protection Act of the Ministry from 1998, the river engineering activities in the reservoir The protection of birds' habitats and biological life in the upper part of the reservoir.

However, the regulation causes also some problems. The water levels in the upstream backwater part are gradually increasing. Long-term results of these processes may cause changes of river bottom in the inlet part of the reservoir as well as a decrease in the reservoir capacity. Another risk is related to the destruction of flood protect and backwater dikes by overtopping. The loss that would occur is difficult to estimate but the probability of this threat seems to increase.

However, the Ministry has enabled the performance of the study presented. In other conditions, the authorities responsible for the effective and secure use of the reservoir would have taken measures to prevent the processes. Now, it is possible to observe the flooding frequency changes in the natural form. It means that these changes are not affected by any hydro-engineering actions, which could prevent from undesirable effects, e.g. frequent and longer inundation of floodplains. The only influences are the reservoir operation, sediment accumulation and vegetation growth.

Water quality problems

The water quality in the Jeziorsko reservoir is by the Voivodeship Inspectorate for Environmental Protection (WIOŚ – pol. Wojewódzki Inspektorat Ochrony Środowiska) in Lodz. It is a part of regional monitoring so measurements are made quite frequently. In 2004 there were six measurements the period July–November. The results of the monitoring show dramatic problems with the water quality in the Jeziorsko reservoir. The physical indicators such as pH, colour and suspended solids were typical of low classes. The oxygen indicators BOD and COD, however, testify to high level of organic carbon. The nitrogen content and salinity were also higher than the admissible values. It was also observed that the concentrations of heavy metals such as mercury, iron and manganese were much higher then their safe levels. The phenol and petroleum derivatives were also detected.

These results illustrate the poor state of the water quality in the Jeziorsko reservoir. Tourist activity such as sailing and swimming should also be carefully controlled.

MATHEMATICAL MODELS APPLIED

HEC-RAS package for river flow simulation

For the simulation of river flow in the study site the HEC-RAS package including the hydrodynamic flow model is used. The basis of this computer model is a well known set of St. Venant equations for unsteady flow simulations and the Bernoullie equation for calculation of steady flow profile. Each case is approximated numerically. The basic principles and procedures of its solution have already been described by many researchers. Examples may be found in Ligget and Cunge [1975], Abbott [1979] and Cunge et al. [1980]. The particular implementation was developed by Barkau [1982] in order to simulate the flow in meandering river with broad floodplains. Finally, the complete description of the approaches and methods applied in the HEC-RAS package was presented by Brunner [2002].

The HEC-RAS model was prepared in such a way that the system's geometry corresponds to the four stages of the river bottom evolution. The first analyzed state of the system is the primary bottom described by Matan [1975]. This geometry was considered as pre-dam conditions reflecting the data for the year 1985. The second geometry was prepared on the basis of measurements made in 1997. This stage of the system is observed a few years after commencement of the reservoir operation. The roughness in the particular cross-sections is the same as in 1985. It does not describe the slight changes in the vegetation cover in this period. Hence, the impact of pure sediment deposition may be assessed. The third and fourth geometries of the system were prepared on the basis of measurements made in 2004 and correspond to the current conditions. However, the third scheme does not reflect the roughness changes due to the vegetation growth. This element is included in the fourth system geometry. Hence, the current state of the system may be assessed in two ways: with and without the vegetation growth.

The riparian vegetation impact on the flow conditions may be modelled by changes in the Manning roughness coefficients. However, this coefficient depends on the vegetation characteristics as well as the water depth. Klopstra et al. [1997] presented the formula describing the velocity changes in the channel with non-submerged vegetation in the steady flow conditions.

$$u = \sqrt{\frac{2gi_0}{C_w md}} \tag{1}$$

where:

- g – acceleration of gravity, $m \cdot s^{-2}$,
- i_0 – the bottom slope, –;

the parameters C_w, m and d describe the features of riparian vegetation:

- C_w – the coefficient reflecting seasonal changes in the leaf cover, –; its values were taken as $C_w = 1.05$ for the winter period and $C_w = 1.40$ for the summer [Armanini et al. 2005],
- m – the average number of plants in one square meter, m^{-2},
- d – the average diameter of bush stalks, m.

Following this concept and applying the Manning equation for wide channel, the formula for the roughness coefficient may be derived

$$n = h^{2/3} \sqrt{C_w \frac{md}{2g}} \tag{2}$$

where:
- n – Manning roughness coefficient, $sm^{-1/3}$
- h – the depth, m.

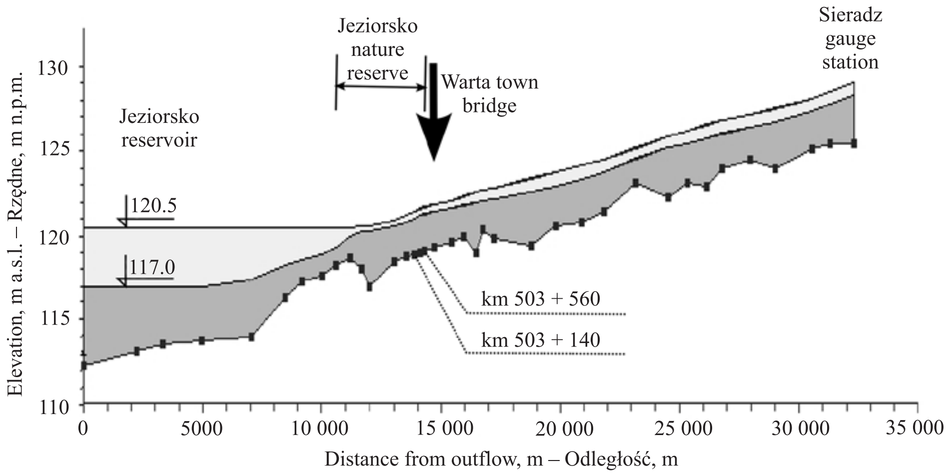

Fig. 1. Water profile in the investigated Warta river reach taken from HEC-RAS postprocessor
Ryc. 1. Układ zwierciadła wody na długości badanego odcinka rzeki Warty otrzymany jako wynik obliczeń za pomocą programu HEC-RAS

The approach presented was used to describe changes in the roughness of the flood-plain. Parameters *m* and *d* were determined on the basis of measurements performed in 2004 and 2005 [Walczak and Przedwojski 2005]. Fig. 1 presents the water profile copied from the HEC-RAS postprocessor. The maximum and minimum wate elevation levels obtained from the simulations are presented. The area under consideration, namely the Jeziorsko is also marked. The study area is located below the bridge in Warta town which is also marked in fig. 1. Hydro-morphological stages of the system's evolution are presented as five potential river states:
1) initial bottom reflecting pre-dam conditions (1985) with free outflow in the outlet (before dam);
2) initial bottom reflecting pre-dam conditions (1985) with outflow corresponding to the average annual scheme of reservoir performance;
3) bottom measured in 1997, but with the initial roughness and outflow as above;

4) bottom measured in 2004, but with the initial roughness and outflow as above;
5) bottom measured in 2004 with roughness reflecting current state of vegetation and its seasonal variation, outflow corresponding to the average annual scheme of reservoir performance (final stage).

The steady state flow module of HEC-RAS package is used in presented research. The main basis for such calculations is Bernoulli equations. In the package this equation is implemented in a way enabling reconstruction of flow in main channel and floodplains. The details of the methodology are presented in Brunner (2002). The basic data provided for model include discharge in modelled river reach and boundary condition. The boundary condition for a single reach is the condition determined for one cross-section. In the case of super-critical flow this particular cross-section is reach inlet. In the case of sub-critical flow the condition is imposed in the outlet. When the mixed flow regime is expected, the conditions have to be determined in both, inlet and outlet cross-sections.

The problem analysed is a steady state and sub-critical flow. Hence, the boundary conditions are imposed only in the outlet cross-section. For simulation of flow in pre-dam the normal depth is used. Analysis of stages with dam performance requires determination of proper water level in the outlet cross-section. The values used are defined in following chapters.

Convection – dispersion model with storage zones

The one-dimensional model of solute transport is used for the simulation of passive dissolved substances distribution and the changes in their content along the river reach. The model consists of three main terms: convection, longitudinal dispersion and storage zones. The first element describes the transport of admixtures in the stream at the average velocity. The next element represents additional mixing related to the nonuniformity of the velocity distribution. The third term describes the exchange of admixtures between the main stream and water stored in the floodplains. The main assumption is that the water in the floodplain is well mixed but stagnant. The detailed description of the storage zone model may be, for example, found in Czernuszenko and Rowiński [1997] and Czernuszenko et al. [1998].

Transport of the solutes in the main stream may be described by:

$$\frac{\partial C}{\partial t} + u\frac{\partial C}{\partial x} - \frac{1}{A}\frac{\partial}{\partial x}\left(K_L A \frac{\partial C}{\partial x}\right) = \frac{\varepsilon}{T}(C_D - C) \tag{3}$$

where:

x – longitudinal direction, m,
t – time, s,
$C(x,t)$ – admixture concentration, $kg \cdot s^{-3}$,
$u(x)$ – cross-sectionally averaged velocity of water, $m \cdot s^{-1}$,
$K_L(x)$ – dispersion coefficient, $m^2 \cdot s^{-1}$,
$A(x)$ – cross-sectional area of the channel, m^2.

It has been assumed that the hydraulic conditions in the channel are, i.e. parameters u, A, K_L do not change in time. $C_D(x,t)$ denotes the concentration of admixtures in the storage zone, parameters $\varepsilon(x)$ and $T(x)$ denote the ratio of the volume of the storage zones to volume of the main stream for the unit length and the penetration time of admixtures into the storage zones, respectively.

The balance of mass in the storage zones is represented by:

$$\frac{\partial C}{\partial t} = \frac{1}{T}(C - C_D) \tag{4}$$

Equations (3) and (4) describe changes of the admixture concentration in time and along the main channel and the storage zones. The model equations are complemented by the following conditions: (1) initial conditions as $C(x,t = 0) = C_p(x)$, $C_D(x,t = 0) = C_{Dp}(x)$ for $x \in [0, L]$ and (2) boundary conditions $C(x = 0,t) = C_0(t)$, $C(x = L,t) = 0$ for $t \geq 0$. C_p and C_{Dp} are the initial distributions of admixtures concentration along the channel reach in both the main stream and the storage zones and C_0 describes the inflow of admixtures at the initial cross-section. L is the length of the channel reach

Equations (3) and (4) with additional conditions were solved numerically. The splitting technique was used [e.g Szymkiewicz 2000, Dysarz et al. 2003, Rowinski et al. 2004]. The problem was divided into three basic subproblems related to the particular transport processes: convection, dispersion and storage zone effects. The details of this methodology are described in Dysarz et al. [2003].

Empirical formula for estimation of dispersion coefficient

The values of coefficients in equations (3) and (4) estimated using different methods. Some of the parameters can be determined on the basis of hydraulic conditions simulated by HEC-RAS. These are: velocities u, cross-section areas A and the ratio of the volume of the storage zones to the volume of the main stream for unit length denoted as ε. The other ones have be found by different methods. One of such parameters is the penetration time T whose values are selected on the basis of earlier experience.

Another approach is used to estimate the longitudinal dispersion coefficient K_L. Its value is determined on the basis of the empirical formula linking the dispersion intensity with the hydraulic conditions in the channel.

By definition the longitudinal dispersion coefficient (KL) determines the proportionality of changes in the dispersive flux and area-average concentration gradient in longitudinal direction. Such definition is inconvenient for practical applications. Instead the empirical formula is used. For the purpose of this study we used the equations presented in the book by Sawicki (2003). They are listed in table 1.

Table 1. Formula for determination of longitudinal dispersion coefficient K_L [on the basis of Sawicki 2003]
Tabela 1. Wzory pozwalające wyznaczyć współczynnik dyspersji podłużnej K_L [na podstawie Sawicki 2003]

No.	Researcher's name Nazwiska badaczy	Equation Równania
1	Elder	$K_L = 5.93hv$
2	Krenkel	$K_L = 9.81hv$
3	Parker	$K_L = 20.2R_h v\sqrt{gC_{CH}^{-1}}$
4	Thackston	$K_L = 7.25hv(v/v)^{1/4}$
5	Patterson & Gloyny	$K_L = 0.8\exp(0.34v\sqrt{A})$

where: h – the water depth, m; A – the cross-section area, m^2; R_h – the hydraulic radius, m; C_{CH} in formula no. 3 is the discharge coefficient for the Chezy formula; v – the velocity, $m \cdot s^{-1}$, averaged over the cross-section; v^* – the dynamic velocity, $m \cdot s^{-1}$. In all formulae in table 1 the unit for longitudinal dispersion coefficient K_L is square meter per second, $m^2 \cdot s^{-}$

RESEARCH CONCEPT

The conditions are related to states of the bottom and different hydraulic effects such as discharge in the reach and the water level in the outlet induced by the reservoir performance. The bottom states taken into consideration are presented ealier in this paper, where the configuration of HEC-RAS is discussed. The representative discharges were chosen on the basis of the river hydrology analysis. Hence, we decided to test the transport process model with the total averaged discharge of 42 $m^3 \cdot s^{-1}$ and two discharges from the low and high value discharges. These are 20 $m^3 \cdot s^{-1}$ and 300 $m^3 \cdot s^{-1}$, respectively. In the first stage of the bottom evolution free outflow from the reach was assumed. For other stages the hydraulic conditions in the outlet of the reach were chosen on the basis of the reservoir performance rules. Hence, the tests were for the minimum water level 117.0 m a.s.l in the reservoir, for average – 119.0 m a.s.l. and for the maximum water level 120.5 m a.s.l. analyzed.

The problem is more complex because the values of two important parameters are uncertain and can only be estimated. These parameters are the dispersion coefficient and penetration time. The first was estimated on the basis of the empirical formula presented in table 1. The penetration time values were chosen on the basis of the earlier research for other rivers. The values applied are as follows: 65 s, 120 s, 180 s and 210 s. According to the authors knowledge and experience the chosen values cover a typical range of penetration time variability. One more assumption was made for the changes in the concentration in the inlet. We assumed a "triangular" distribution for changes of the admixture mass in the inlet cross-section as it is shown in fig. 2. The parameter characterizing such a temporal distribution is the maximum concentration. Simulation tests were performed for tree typical values namely 0.12 kg · m^{-3}, 0.20 kg · m^{-3} and 0.5 kg · m^{-3}.

ANALYSIS OF RESULTS

The test results from the performed showed a good agreement with expectations. Shown in fig. 2 and 3. The first presents typical changes in the admixture concentration along the river reach for the bottom state (1) scenario. The "triangular" graph in the left represents potential changes in the admixture in the inlet, at 521,81 km the Sieradz gauge station. Are the inflow and outflow of the admixture in the reach between km 503,560 and km 499,890, where the most intensive processes of sediment accumulation and vegetation growth occurred. We analyzed changes in the maximum concentration in this reach in relation to states of bottom evolution. An result is shown in fig. 3. Different lines in the figure are different formulae used for estimation of the dispersion coefficient (K_L). The results presented in fig. 3 were obtained for the discharge of 20 m^3 · s^{-1}, water level in the reservoir 117,0 m a.s.l., penetration time 65 s and the maximum concentration in the inlet 0.12 kg · m^{-3}. Similar results were obtained for other cases.

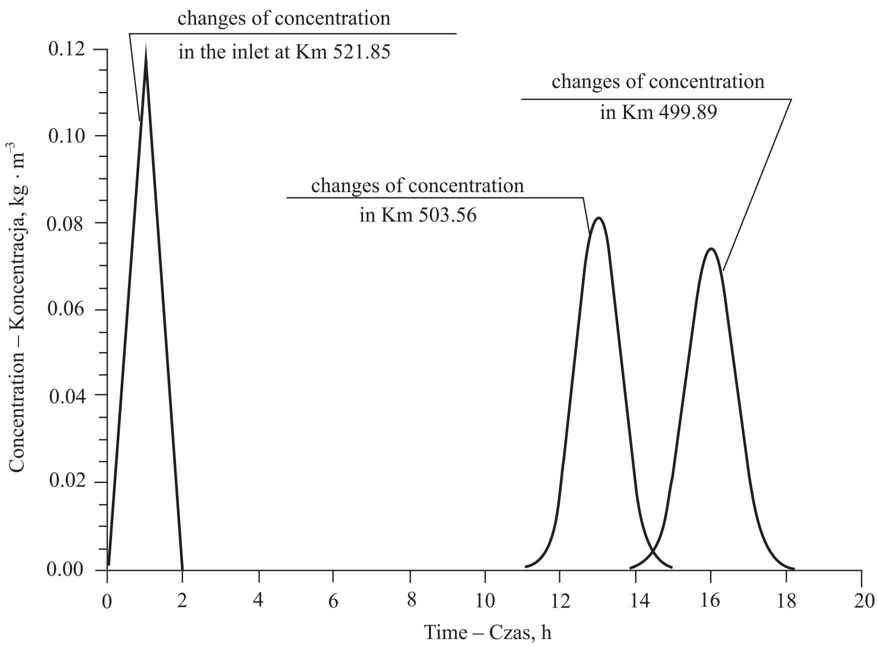

Fig. 2. Typical results of admixture transport simulation; parameters: bottom state (1), discharge 20 m^3 · s^{-1}, water level 117.0 m a.s.l., maximum concentration 0.12 kg · m$^{-3,}$ penetration time 65 s, dispersion estimated from Elder formulae

Ryc. 2. Typowe wyniki symulacji przenoszenia domieszki; parametry: stan dna (1), przepływ 20 m^3 · s^{-1}, stan wody 117,0 m n.p.m., maksymalna koncentracja 0,12 kg · m^{-3}, czas penetracji 65 s, współczynnik dyspersji wyznaczany na podstawie wzoru Eldera

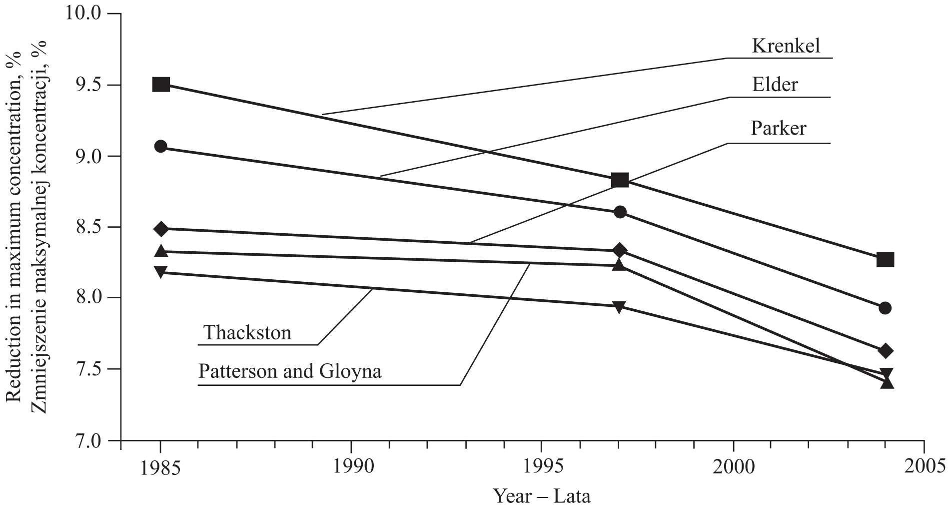

Fig. 3. Changes in the maximum concentration of potential dissolved substance in the reach under investigation in percentages for the discharge 20 $m^3 \cdot s^{-1}$, water level 117.0 m a.s.l., penetration time 65 s and maximum concentration in the inlet 0.12 $kg \cdot m^{-3}$

Ryc. 3. Zmiany w maksymalnej koncentracji potencjalnej substancji rozpuszczonej na badanym odcinku przedstawione w procentach dla przepływu 20 $m^3 \cdot s^{-1}$, stanu wody 117,0 m n.p.m., czasu penetracji 65 s oraz maksymalnej koncentracji 0,12 $kg \cdot m^{-3}$

The degree of reduction in the maximum concentration of the dissolved substances decreases due to the morphodynamic changes and vegetation growth (fig. 3). At the beginning in 1985 and at the end by 2004 of the lines the impact of the basic reservoir performance and pure vegetation growth, respectively, on the maximum concentration visible. However, these two processes have relatively small influence on the transport process as the results for two states of 1985 (fig. 3) and two states of 2004 are the same.

The most important factor is sediment accumulation, which is visible as a decrease in each line over years (fig. 3). The sediment accumulation decreases the cross-section area A. If the area is smaller, the flow velocity increases. Then the transport of dissolved substances is faster and the decrease of its concentration is smaller, because the depressive mixing lasts shorter.

CONCLUSIONS

The results analyzed have shown that there is an important dependence between changes occurring in the selected river reach and the admixture transport. The sediment deposition in the selected river reach causes a decrease in the cross-section area in the main stream and more frequent inundation of floodplains. Another result is an increase in the flow velocity of the main stream. This gives the effect of faster transport, worst mixing and lower degree of reduction in the maximum concentration. It means that potential pollutants flow faster to the reservoir and the river self-treatment is limited.

The reservoir performance and vegetation growth are considered to be important factors impacting the changes in the selected river reach. The first of them should influence much the inundation frequency. As shown in fig. 3, the related effects in the admixture transport characteristics are not observed. The influence of the reservoir performance can be neglected.

It is expected that the vegetation increases the flow resistance and friction loss. It should decrease the flow velocity and counteract the changes observed in fig. 3. This effect was not observed. The vegetation seems to be a factor of low importance in our analyses.

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