

## FORECASTING MODEL OF TOTAL RUNOFF AND ITS COMPONENTS FROM A PARTIALLY URBANIZED SMALL LOWLAND CATCHMENT

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### ABSTRACT

The aim of the work is to present a new hydrological model, developed at the Institute of Technology and Life Sciences (Instytut Technologiczno-Przyrodniczy), which can be used to simulate and study the formation of total runoff and its components from a small lowland agricultural catchment, in various meteorological conditions, taking into account the different initial retention status of the catchment and the impact of urbanized areas on the said runoff.

The study involved the necessity of choosing the type of the model, and the manner of describing hydrological processes. Based on studying subject literature, as well as on the results of previous research and observations conducted in the upper Mławka river catchment (left tributary of the Wkra river), it was assumed that the most appropriate tool for prognostic and simulation purposes would be a conceptual model, that – in its structure and description of hydrological processes – would take into account the theory of active areas (variable source areas). It was assumed that a simple model would be developed as far as possible, that would create the possibility of simulating hydrographs of total runoff, surface runoff, subsurface runoff, and ground water runoff from a natural catchment, and from catchments which have been urbanized to varying degrees. The work also includes the presentation of proposed principles for the classification of urbanized areas, taking into account their location within the catchment area and the way of draining rainwater, as well as a description of adequate modules of the model that would make it possible to simulate the runoff from the urbanized sub-catchments.

The model is described by state-space representations (differential equations) and output equations. It works with a one-hour time interval. The input is total precipitation and reference evapotranspiration. On the output end, we receive the total runoff hydrograph, and the hydrographs of the components of this runoff. The model includes 10 optimized parameters, occurring in procedures simulating processes in the natural, non-transformed part of the catchment, and 10 measurable parameters used to describe partially urbanized catchments.

The article presents the results of identifying the parameters for the model, and of its verification. For this purpose, 5 hydrographs of total runoff were used, between 192 and 432 hours long, including storm surges. They were recorded in the catchment of the upper-course Mławka river. The obtained values of the criterion functions lead us to conclude that the quality of the model is very good. The model has been used for simulation studies of the impact of urbanization of the agricultural catchment on the parameters of river runoff (Krężałek 2018).

**Keywords:** river runoff, river catchment, urbanization, active areas (variable source areas), conceptual model

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## INTRODUCTION

The impact of the degree of urbanization of the catchment area on river runoff has already been the subject of many studies. The results of most studies concerned mainly the impact of increasing the share of sealed areas within the catchment on the parameters of flood surges, taking into account only surface (overland) runoff, and assuming that the latter is formed on the entire surface area of the catchment, in accordance with the Horton theory. Most frequently, very simple conceptual models with concentrated parameters of the Nash cascade of linear reservoirs are used, along with the effective SCS-CN rainfall-runoff model (for instance, Banasik 2009), enabling the simulation of the runoff hydrograph even with a very small time interval; or complex models with distributed parameters are used, such as for example, the SWAT, i.e. Soil and Water Assessment Tool, however, for this type of models, a daily time interval is generally adopted, which significantly reduces the reliability of results obtained for small river catchments (Neitsch et al. 2011). In the case of the first type of models, it is only possible to account for the total share of sealed areas in the catchment, without taking into account their location, whereas the description of the river runoff formation is very simplified and – as is clear from numerous literature sources as well as from previous studies carried out in the experimental catchments of the Institute of Technology and Life Sciences – it is not always consistent with the actual observations. This mainly applies to the only partially urbanized catchment area, with a significant permeability of unpaved terrain, for which it is recommended to use models that take into account the concept of variable source (active) areas.

Models intended for simulation of runoff from typical urbanized and channelled catchments, for instance, the EPA-SWMM (Rossman and Huber 2016) also tend to apply very simplified models of the unpaved catchment component.

Therefore, there arose a need to develop a tool for forecasting the impact of urbanization on river runoff, in the form of a model that would reflect a catchment that had been only partially urbanized, whose component of the natural catchment could be assessed and verified on the basis of data from observations.

The purpose of this work was to develop a hydrological river runoff model for a lowland agricultural catchment, together with the urbanized catchment area modules, as well as to determine the classification of different types of these areas, so that the type-specific way of influencing the formation of flood runoff processes can also be accounted for in the structure of the model, resulting from the location of the given area within the catchment.

The development of a new hydrological model involved the necessity of choosing the model type, and the method of describing hydrological processes. Based on literature studies as well as the results of previous research and observations conducted in the selected catchment (for instance, Szymczak and Szelenbaum 2003), it was assumed that the most appropriate tool for accomplishing the adopted main research objective would be a conceptual model with lumped parameters, taking into account the theory of variable source areas in its structure and description of hydrological processes. This set another intermediate goal, consisting in the development of a possibly simple model that would enable the simulation of total, direct (surface and subsurface) and groundwater runoff hydrographs from the natural catchment, and from the sub-catchments that had been urbanized to varying degrees, as well as the verification of the model's correctness based on the data from measurements (Krężałek 2017).

## MATERIAL AND METHODS

The present research involved development of a river runoff model with optimized parameters, for a small natural lowland catchment, and the verification of that model, based on the results of long-term hydrological observations. The aforementioned model was then supplemented with modules for modelling the runoff from urbanized areas with parameters determined on the basis of measurable physical features of these areas. Identification of the parameters of the natural catchment component was carried out on the basis of data from the research catchment of the Institute of Technology and Life Sciences, with an area of 66.17 km<sup>2</sup>, located in the upper course of the Mławka river, which is the left tributary of Wkra river. Measurements and observations of hydrological processes have been carried out

in this catchment from 1976 and they continue to the present day. Based on the results of these observations (for instance, Szymczak and Szelenbaum 2003) as well as literature studies (Horton 1933, Betson 1964, Cappus 1960, Dunne and Black 1970a, 1970b, Hewlett and Hibbert 1967, Weyman 1970, 1973), it was decided that the model should be developed in accordance with the theory of variable source (active) areas.

According to this theory, only a part of the catchment area with time-varying dimensions takes part in the formation of direct runoff, whereas surface runoff occurs mainly on those surfaces that are completely saturated with water, called the variable source areas (VSA). This phenomenon occurs when the soil is so saturated that the water is not able to penetrate into it, and it is called the saturation excess surface runoff. The total saturation of the ground, which determines its formation, is caused by raising the groundwater table, or by the occurrence of sub-surface runoff. Variable source areas usually occur around watercourses or areas where the water level and the thickness of the soil layer produce a rapid saturation of the soil with water, caused by the lateral flow of water in the soil, by sub-surface runoff, or by groundwater outcrop. However, active VSA areas may also occur outside the immediate vicinity of watercourses (Ward 1984).

It was decided that a conceptual model with lumped would be developed, described with state-space equations, and the output equations. The final structure of the model was determined as a result of numerous attempts and subsequent simplifications of more complex models. During the identification of their parameters, it often turned out that the omission or lumping of some processes did not significantly affect the values of the model quality indicators or the compliance of the simulated hydrographs with the measured ones.

The model's software, and the study of the model's intermediate versions were developed using the SYSMOR River Runoff Modelling System (Szymczak 1996, 2003). Next, the model's parameters were identified using the optimization method based on observational data from the upper-course Mławka river catchment. The next step in the research was the verification of the model, using both the dependent and independent material. After obtaining satisfactory results of the verification of the natural catchment mod-

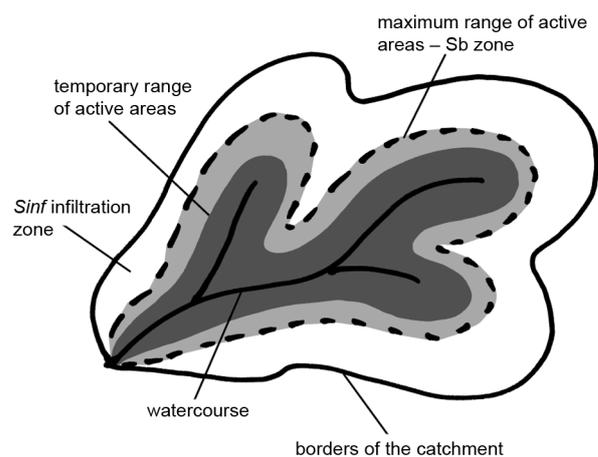
el, its structure was supplemented with modules used for modelling the runoff from urbanized areas. For this purpose, a well-known and widely used urban drainage catchment module has been used, included in the EPA-SWMM model. The aforementioned model has been well verified and described in numerous international literature studies (Rossman and Huber 2016) as well as in Polish literature on the subject (for instance, Szeląg et al. 2016). As a result, a model of a partially urbanized agricultural catchment was constructed, so that the size of the urbanized area could be determined at the same time, as well as its position relative to the active VSA areas and the manner of draining rainwater.

## RESEARCH RESULTS

### Description of the model developed for a natural catchment

The most important assumption when constructing the model was to consider the theory of active areas (variable source areas). Therefore, it was necessary to distinguish these areas and to take into account three components of runoff: surface runoff  $H_p$  and subsurface runoff  $H_{pp}$  (together forming direct runoff  $H_b$ ), as well as groundwater runoff  $H_{gr}$ .

When developing the model, it was assumed that the catchment area is divided into two parts: the *Sb* direct flow forming zone and the *Sinf* infiltration zone (see: Fig. 1).



**Fig. 1.** Maximum and temporary range of variable source (active) areas in river catchment

Source: own study

The first of the aforementioned zones, adjacent to the river network and therefore more waterlogged, takes part in the formation of surface and subsurface runoff. On the other hand, the precipitation falling on the surface of the second zone of the catchment, which is removed from the watercourses – when we have taken into account losses on interception, surface retention and evaporation – can only infiltrate, supply soil retention and groundwater, and as a result, it will reach the river bed in the form of a much delayed groundwater runoff. The total runoff is therefore calculated as the weighted sum of the runoffs from both these parts of the catchment, and the weights are the shares of the surface of these parts in the total catchment area. It has been assumed that the area on which *Sb* direct runoff is formed has constant dimensions, determined by the maximum range of variable source (active) areas with an  $A_{OC_{max}}$  area size. In the *Sb* area, precipitation reaching the area covered by the temporary range of variable source areas is fed by saturation excess surface runoff. Otherwise, it is infiltrating and, after exceeding a certain threshold of soil and ground retention, it supplies the subsurface runoff. The ratio of the maximum area of the catchment's part involved in the formation of the direct runoff  $A_{OC_{max}}$  to the total surface of the catchment  $A_c$  is the maximum direct-supply ratio:

$$\alpha_{max} = \frac{A_{OC_{max}}}{A_c} \quad (1)$$

The above is included in the model as an optimized parameter, which acts as a weighting factor in the above-described weighted sum (see: node W10, figure 2).

The following elementary procedures (nodes) of the SYSMOR system were used to build the model: SUM – the sum of two signals, MUL – the multiplication of the signal by a constant, ROZN – the difference between two signals, DZPE – the divisor of input signal to 3 output signals controlled by the value of the designated coordinate, ZL – the linear reservoir, PZL – the threshold linear reservoir, KZN – the cascade of non-linear reservoirs, SWP – the weighted sum.

The model has two inputs: total precipitation  $P$  and evaporation  $E$ , and one main output: total runoff  $Q_c$  as well as 13 intermediate outputs. All the outputs are registered; therefore it is possible to follow the pro-

gress in time of the modelled constituent processes, such as surface and subsurface runoff.

The model includes, in a simplified way, the processes of interception, surface retention, and soil retention, through their aggregation and balancing of precipitation with evaporation at the model's input.

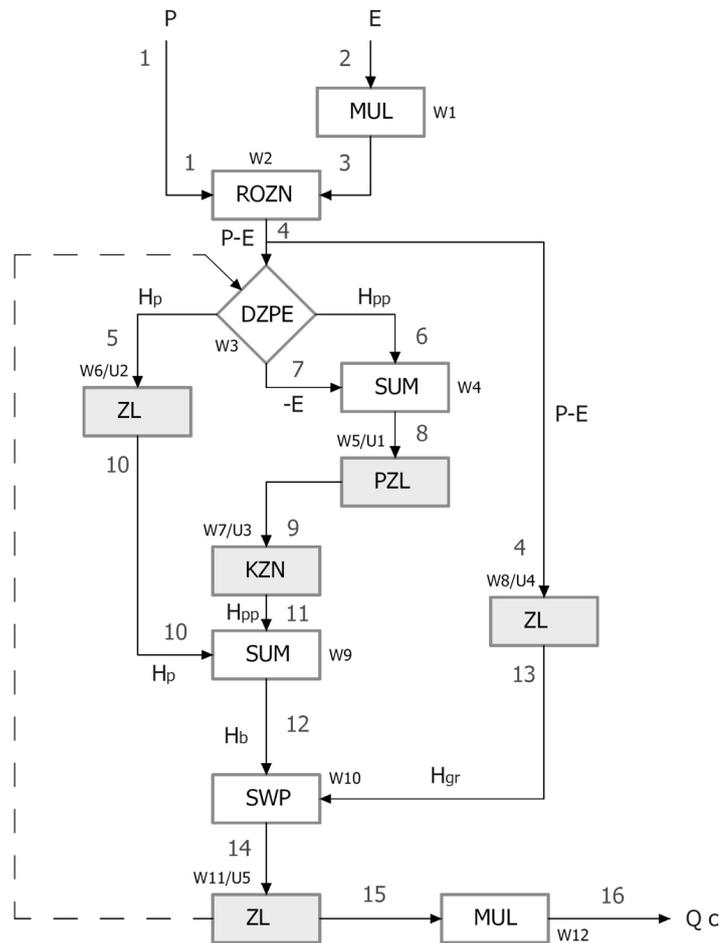
The surface runoff process is represented by a single linear reservoir, whereas the sub-surface runoff is represented by a cascade of 5 non-linear reservoirs. The concept of a linear reservoir was also used for modelling groundwater retention and groundwater runoff as well as for the description of the flow transformation process in the riverbed.

The input signals to the model are the sequences of total precipitation sums  $P$  and the Penman reference evapotranspiration index  $E$ , calculated for the assumed time interval of  $\Delta t = 1$  h, and expressed in millimetres per hour.

At the beginning, the evapotranspiration  $E$  (signal 1) is multiplied by the coefficient  $e$  (node W1) to adjust the value of the Penman reference evapotranspiration index to the value of the actual evapotranspiration in the field:

$$E = e \cdot E_p \quad (2)$$

In the W2 node, the difference between total precipitation and evapotranspiration in the field is calculated, which depending on the sign determines the catchment supply volume ( $P - E > 0$ ) or the outflow of retention reserves ( $P - E < 0$ ). The signal 4 therefore assumes the value of  $P - E$ . It is then divided in a specific way into three signals: 5 – part of the precipitation supplying the surface runoff  $H_p^*$ , 6 – part of infiltrating precipitation  $Inf$ , 7 – part of the evapotranspiration (with negative value) equal to the result of the balance of precipitation and evapotranspiration when the volume of the latter exceeds the amount of the former. This division is carried out with the use of the DZPE procedure (node W3) in accordance with the equations (3), (4) and (5). The proportions of the division into signals 5 and 6 ( $H_p^*$  and  $Inf$ ) are variable, and they depend on the coordinate of state – filling of the linear reservoir, which, within the model, is responsible for the process of flow transformation in the riverbed (node W11 / System 5). The parameters of divisor  $B$  and exponent  $b$  are determined by way of optimization.



**Fig. 2.** Schematic model of a natural catchment: 1, 2, ..., 16 – numbers of signals; W8/U4 – number of the node and number of the system within the model’s structure;  $C_3, m, n$  etc. – parameters of the given node; remaining explanations included in the text

Source: own study

$$H_p^* = \begin{cases} 0 & \text{for } P - E < 0 \\ \left(\frac{Z_5}{B}\right)^b \cdot (P - E) & \text{for } P - E \geq 0 \text{ and } Z_5 \leq B \\ P - E & \text{for } P - E \geq 0 \text{ and } Z_5 > B \end{cases} \quad (3)$$

$$Inf = \begin{cases} P - E - H_p^* & \text{for } P - E \geq 0 \\ 0 & \text{for } P - E < 0 \end{cases} \quad (4)$$

$$E^* = \begin{cases} 0 & \text{for } P - E \geq 0 \\ P - E & \text{for } P - E < 0 \end{cases} \quad (5)$$

The concept of operation of the DZPE divisor is such that increasing the filling of the riverbed is accompanied by a corresponding increase in the extent of the saturation zone and thus an increase in the supply of surface runoff, at the expense of supplying the infiltration. Expression  $\left(\frac{Z_5}{B}\right)^b$  in the formula (7.5) (for  $Z_5 \leq B$ ) adopts the value of the relative share of area currently occupied by the  $A_{OC}$  variable source areas in the maximum area they can occupy in the  $A_{OCmax}$  catchment, thus  $\left(\frac{Z_5}{B}\right)^b = \frac{A_{OC}}{A_{OCmax}}$ .

The process of precipitation transformation  $H_p^*$  (signal 5) in surface runoff of  $H_p$  (signal 10) is realised via the linear reservoir model (W6 node / Layout 2) (see: Fig. 3), whose operating principle is described by formulas (6) and (7).

$$Y(t) = c \cdot Z(t) \quad (6)$$

$$\frac{dZ(t)}{dt} = X(t) - Y(t) \quad (7)$$

where:

- $X(t)$  – inflow to the reservoir at time  $t$ ,  $\text{mm} \cdot \text{h}^{-1}$ ;
- $Y(t)$  – runoff from the reservoir at time  $t$ ,  $\text{mm} \cdot \text{h}^{-1}$ ;
- $c$  – is the runoff factor from the reservoir,  $\text{h}^{-1}$ ;
- $Z(t)$  – filling the reservoir at time  $t$ .

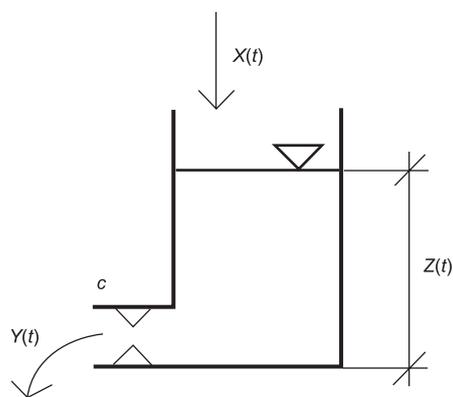
Further in the paper, a simplified equation will be used, omitting the time variable of  $t$ .

In the case of the discussed W6 node, constituting the U2 system, the following apply:

$$H_p = c_2 \cdot Z_2 \quad (8)$$

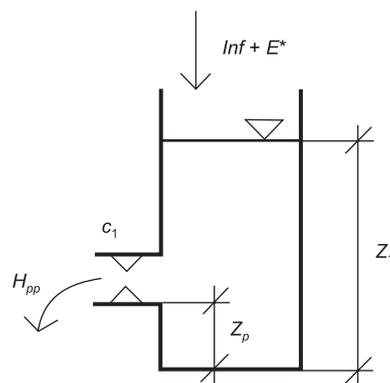
$$X = H_p^*, Y = H_p. \quad (9)$$

The infiltrating water supplies the soil water reservoir, described as the threshold linear reservoir, whereas increasing its soil retention  $Z_1$  (see: Fig. 4). After exceeding the  $Z_p$  limit value, the supply of subsurface runoff  $H_{pp}^*$  begins. The soil water reservoir is also emptied as a result of  $E^*$  evapotranspiration losses. The input to the reservoir is therefore signal 8, taking into account the balancing of  $Inf$  supply and the  $E^*$  losses, which takes place at the W4 node.



**Fig. 3.** Model of a linear reservoir

Source: own study



**Fig. 4.** Model of a linear reservoir with the threshold outflow; symbol explanations included in the text

Source: own study

$$H_{pp}^* = \begin{cases} 0 & \text{for } Z_1 \leq Z_p \\ c_1 \cdot (Z_1 - Z_p) & \text{for } Z_1 > Z_p \end{cases} \quad (10)$$

The signal 9 constituting the supply of subsurface flow  $H_{pp}^*$  is then transformed in the cascade of non-linear reservoirs (see: Fig. 5).

The cascade of  $N$  non-linear reservoirs can be described by the system of equations:

$$Y_i(t) = c \cdot Z_i^m(t) \quad (11)$$

$$\frac{dZ_1(t)}{td} = X(t) - Y_i(t) \quad (12)$$

$$\frac{dZ_i(t)}{td} = Y_{i-1}(t) - Y_i(t), i = 2, \dots, n \quad (13)$$

where:

- $X(t)$  – inflow to the first reservoir at time  $t$ ,
- $Y_i(t)$  – runoff from the  $i$ -th reservoir at time  $t$ ,
- $c$  – discharge coefficient, which is constant for all reservoirs,
- $Z_i(t)$  – filling of the  $i$ -th reservoir at time  $t, i = 1, 2, \dots, N$ ,
- $m$  – the exponent resulting from the non-linearity of the process.

Sub-surface flow (signal 11) is calculated as the output from the last reservoir in the cascade, and it is expressed by the following formula:

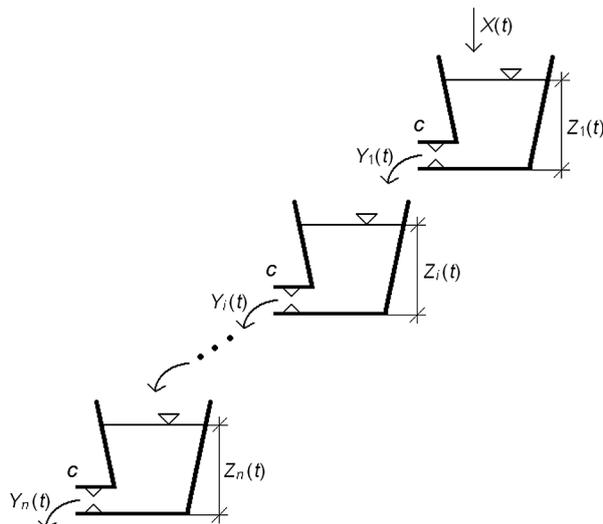
$$H_{pp} = c_3 \cdot Z_{3,n}^m \quad (14)$$

where:

- $c_3$  – value of the discharge coefficient for reservoirs in the KZN cascade, constituting System 3;
- $Z_{3,n}$  – filling the last reservoir in the cascade;  $m$  – exponent.

Direct flow  $H_b$  (signal 12) is calculated as the sum of surface runoff  $H_p$  and subsurface runoff  $H_{pp}$  at node 9:

$$H_b = H_p + H_{pp}. \quad (15)$$



**Fig. 5.** Schematic presentation of non-linear reservoirs cascade; symbol explanations included in the text

Source: own study

The groundwater runoff (signal 13), in accordance with the adopted assumptions, is supplied in areas where direct runoff does not occur. The total precipitation, reduced by the amount of evaporation (signal 4) infiltrates, while supplying the groundwater reservoir. Similarly to surface runoff, also groundwater runoff is modelled using a linear reservoir (W8 node / System 4), described with equations (6) and (7), and shown in Figure 3. The output equation takes the following format:

$$H_{gr} = c_4 \cdot Z_4 \quad (16)$$

where:

- $Z_4$  – filling of the groundwater reservoir;
- $c_4$  – discharge coefficient from the groundwater reservoir.

The total runoff supplying the river network  $H_c$  (signal 14) is calculated as the weighted sum of the direct runoff  $H$  and the groundwater runoff  $H_{gr}$  in node 10 (SWP):

$$H_c = w \cdot H_b + (1 - w) \cdot H_{gr} \quad (17)$$

The weight coefficient  $w$  is equal to the relative proportion of the area corresponding to the maximum extent of the areas on which the direct runoff is formed in the total catchment area. This parameter is optimized.

The process of flow transformation in the riverbed influences the ultimate form of the flood wave, observed in the analysed profile. As a result of the determined retention capacity of the riverbed and the hydraulic resistances, the flood wave flattens along the course of the river. The effect is the greater the longer the wave has to travel. Flow transformation in the riverbed in the discussed model is realized by means of a linear reservoir (node W11 / System 5), the operation of which is described by the system of equations (6) and (7), whereas equation (8) takes the following form:

$$H_{tr} = c_5 \cdot Z_5, \text{ mm} \cdot \text{h}^{-1} \quad (18)$$

where:

- $Z_5$  – filling of the flow transformation reservoir;
- $c_5$  – discharge coefficient from the flow transformation reservoir.

Last stage of calculations is the conversion of units in which hourly values of total runoff are given, expressed as the runoff layer over time equal to the model time interval of  $\text{mm} \cdot \text{s}^{-1}$  (signal 15), to the flow rate expressed in cubic meters per second  $Q_c$ , as follows:

$$Q_c = j \cdot A \cdot H_{pr}, \text{m}^3 \cdot \text{s}^{-1} \quad (19)$$

where:

$j$  – conversion factor of units;

$A$  – size of catchment area. The  $j \cdot A$  product for the analysed catchment is 18.381.

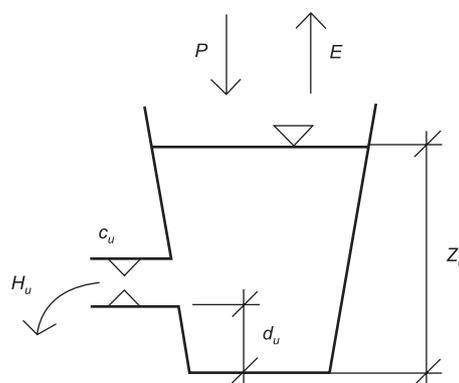
The conversion of units is carried out in the last node (node W12), from which the main output from the model (signal 16) is derived in the form of total runoff  $Q_c$  expressed in  $\text{m}^3 \cdot \text{s}^{-1}$ . The calculation of current coordinates of states (water levels) of linear reservoirs and nonlinear reservoirs is performed by numerical integration of mass continuity equations of type (7) after substitution of output equations of type (6) or (11) respectively.

## THE MODEL OF AN URBANIZED SUB-CATCHMENT

The hydrological model of an urbanized sub-catchment was developed in accordance with the concept described in the Storm Water Management Model (SWMM) developed at the National Laboratory for Risk Management (NRMRL) in the USA (Rossman and Huber 2016). In the SWMM model, the sub-catchment is represented by a rectangle with a given surface area and width. For the calculation of runoff, the catchment is divided into two types of surfaces – permeable (on which infiltration is considered) and impermeable. The runoff is calculated separately for each surface and then added up. Due to the very high infiltration capacity of the soil in the upper-course Mławka river catchment, the surface runoff component arising in the permeable areas was omitted.

The stream of rainfall sewage flowing away from the catchment is determined using a single reservoir with a threshold nonlinear output.

Within the SYSMOR system, the procedure describing such a reservoir constitutes a node, marked with the PZN symbol (see: Fig. 6).



**Fig. 6.** Schematic presentation of with a non-linear reservoir with a threshold output; symbol explanations included in the text

Source: own study

The filling of the reservoir corresponds to the volume of water that was in the catchment, reduced by losses on evapotranspiration and land filling.

The equation of output from the reservoir is as follows:

$$H_u = c_u \cdot (Z_u - d_u)^{m_u} \quad (20)$$

where:

$H_u$  – runoff from urbanized areas,  $\text{mm} \cdot \text{h}^{-1}$ ;

$c_u$  – discharge coefficient,  $\text{mm}^{2/3} \text{h}^{-1}$ ;

$Z_u$  – reservoir filling, mm;

$m_u$  – exponent (in the SWMM model, it takes a constant value of  $m_u = 5/3$ );

$d_u$  – volume of surface retention of impermeable areas, mm.

The value of the discharge coefficient  $c_u$  from the reservoir can be determined after appropriate transformation of equations provided in the documentation of the SWMM model (Rossman and Huber 2016).

$$c_u = \frac{36 \cdot i^2}{\eta \cdot L}, \text{mm}^{-2/3} \text{h}^{-1} \quad (21)$$

where:

$\eta$  – coefficient of roughness of sealed areas,  $\text{s} \cdot \text{m}^{1/3}$ ;

$i$  – average decrease in the size of impermeable areas, -;

$L$  – length of the flow route, m

The Viessman and Lewis formula (2003) was used to determine the value of the second parameter – the height of the nonlinear output threshold (2003):

$$d_u = 25,4 \cdot (0.136 - 0.032 \cdot i_u) \quad (22)$$

where:

$d_u$  – surface retention of impermeable areas, mm;

25.4 – unit conversion from inches to millimetres;

$i_u$  – average gradient in the size of impermeable areas, %.

For the assumed average decrease in the size of impermeable surfaces (1%), using formula (22), a retention volume of  $d_u = 2.64$  mm was obtained.

## STRUCTURE OF THE PARTIALLY URBANIZED CATCHMENT MODEL

It was assumed that the model of a partially urbanized catchment should reflect several possible development options of the natural catchment. The basic distinction between various types of urbanized areas was founded in the criterion of their location, according to the catchment division into two zones: the maximum extent of *Sb* active areas (variable source areas), in where the direct runoff is formed, and the *Sinf* infiltration zone, where the excess precipitation, after deducting the losses on interception and evaporation can only infiltrate. (see: Fig. 1).

Type A areas are distinguished, which will be located in the zone of direct runoff (*Sb*) and B type areas, which will be located in the areas of the infiltration zone (*Sinf*).

A more detailed breakdown referred to the way rainwater was discharged from urbanized areas.

In the areas located within the *Sb* zone, that is, the maximum range of variable source areas, the following are distinguished:

A1 – urbanized areas with water removal by drainage, or those without channelling, where water flows directly to the riverbed;

A2 – urbanized areas, not channelled, where waters flow to unsealed areas and additionally supply surface runoff.

In the *Sinf* infiltration zone, the following areas were distinguished:

B1 – urbanized areas with water drainage via a rainwater channels directly to the river network;

B2.1 – urbanized areas without channelling, located so close to the *Sb* area that the flow from these areas constitutes an additional supply of that zone;

B2.2 – urbanized areas, not channelled, with the management of rainwater near the occurrence of precipitation (including reservoirs and absorbing wells, infiltration equipment).

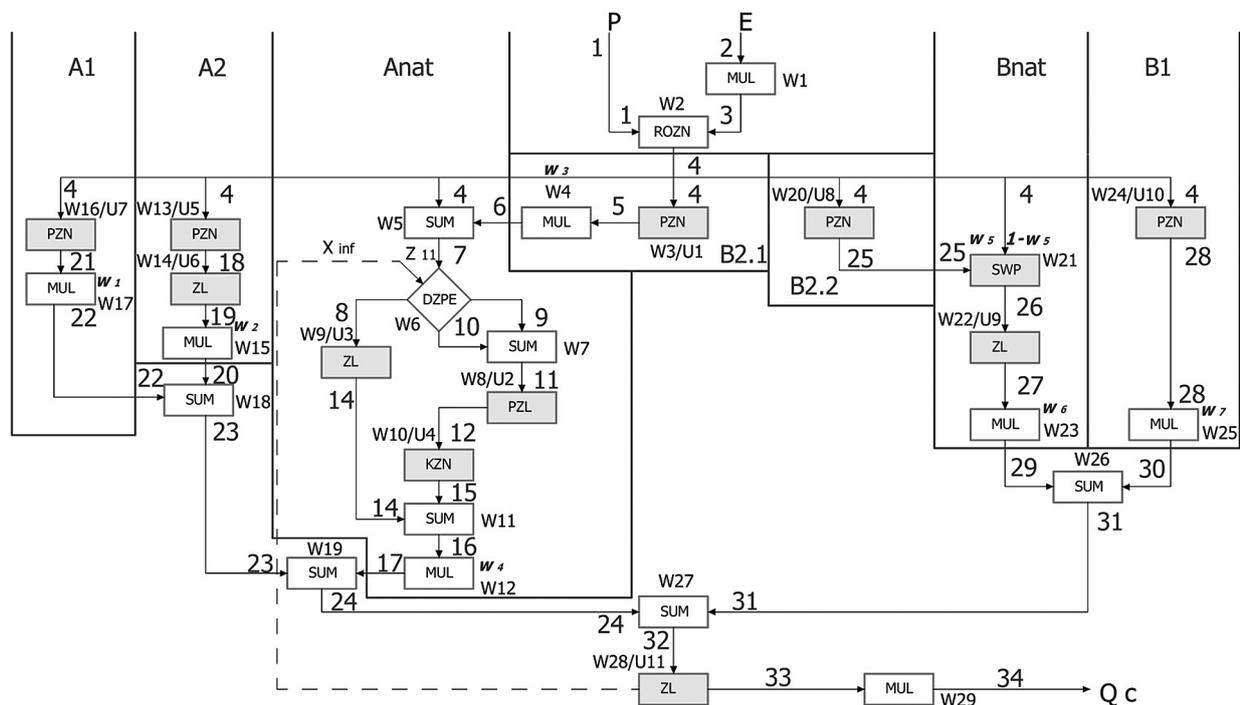
In order to determine the ultimate structure of the model, we had to check whether it was necessary to consider the flow transformation in the sewerage network, while assuming an hourly calculation interval. This is because runoff in the channels is subjected to retention and time shift. Based on the results of the simulations conducted using the SWMM model, it was found that the changes in the shape of the hydrograph are practically irrelevant at the adopted hourly calculation interval. Therefore, the model omits the process of transformation of the flow in the sewerage network.

In the structure of the model, seven modules were distinguished, including two natural catchment modules – Anat for processes occurring within the direct runoff formation zone and Bnat for the infiltration zone, and five modules corresponding to models of the partially urbanized sub-catchment areas of A1, A2, B1, B2.1 and B2.2 types (see: Fig. 7).

As in the case of the natural catchment model, the input signal to each module is a sequence of hourly values obtained by balancing the precipitation with evapotranspiration from the area (signal 4 which is the output from node W2).

The principle of mass conservation has been taken into account by appropriately selecting the weight coefficients before finally adding up the output signals from the individual modules. The Anat module takes into account the processes taking place in the zone of direct runoff formation in areas that have not been subjected to urbanization. It is identical with the corresponding module in the model of a natural catchment. The only difference is that the input signal is additionally increased, by adding the amount of rainfall sewage from the urbanized area of type B2.1. The mass is balanced by out using the  $w_3$  and  $w_4$  weights, according to formulas (25) and (26).

The Bnat module takes into account the supply into the groundwater reservoir, and the formation of groundwater runoff in the part of the *Sinf* infiltration



**Fig. 7.** Schematic model of a partially urbanized agricultural catchment: PZN – non-linear threshold output reservoir, other node symbols as in figure 2; A1, A2,...B2.2 – modules of the model; 1, 2,...34 – numbers of signals; W20/U8 – number of the node and the system;  $w_1, w_2, \dots, w_7$  - weights

Source: own study

zone that has not been subjected to urbanization. The difference compared to the natural catchment model is that the input supply signal is calculated as the weighted sum of the runoff from areas of B2.2 type and signal 4 ( $P-E$ ). The mass is balanced using the  $w_5$  and  $w_6$  weights, according to formulas (27) and (28).

In urbanized areas of all the listed types, the transformation of precipitation into surface runoff is modelled using a single reservoir with nonlinear threshold output (PZN). As a result, surface runoff from these areas, with the exception of area B2.2, feeds into the river network, where it is subjected to the process of flow transformation in the riverbed. The flow from the B2.2 area does not feed into the river network, but it connects to the supply of groundwater reservoir (signal 26) through the weighted sum (SWP) at node W21.

The runoff from the area of B2.2 type is, however, reduced by the surface retention volume of the sealed areas  $d_u$ .

Surface runoffs occurring in the A1 and B1 type areas are discharged directly into the river network via rainwater drainage system. This is reflected in the structure of the model (see: Fig. 7) in such a way that output signals from the A1 and B1 modules, previously balanced with the  $w_1$  and  $w_7$  weights (equations 23 and 29), and numbered 22 and 30 respectively, after adding to the output signals from the other modules at the W18, W19, W26 and W27 nodes, are introduced as input signal No. 32 to node W28 (ZL linear reservoir), performing the function of flow transformation in the riverbed.

Surface runoff from the A2-type urbanized area reaches the river network in the form of sub-surface runoff only after it has been transformed in the unsealed  $Sb$  zone. This transformation is carried out in module A2 at node W14 using a linear reservoir model with an output coefficient equal to the linear reservoir factor ZL, node W9, located in the Anat natural catchment module. The balancing of the output from

the A2 module takes place using weight  $w_2$  (equation 24), by multiplying the signal No. 19 in the node W15 by its value.

Runoffs from individual modules are balanced on the scale of the whole catchment with the help of weighting factors –  $w_1, w_2, w_3, w_4, w_5, w_6$  and  $w_7$ , the values of which depend, among other things, on the assumed parameters characterizing the calculated variants.

The weight coefficients used to balance the runoff from individual modules of the model are determined on the basis of the size of the surface of the distinguished types of urbanized areas:

$$w_1 = \frac{A_{A1}}{A_C} \quad (23)$$

$$w_2 = \frac{A_{A2}}{A_C} \quad (24)$$

$$w_3 = \frac{A_{B2.1}}{A_{OC_{max}} - A_{A1} - A_{A2}} \quad (25)$$

$$w_4 = \frac{A_{OC_{max}} - A_{A1} - A_{A2}}{A_C} \quad (26)$$

$$w_5 = \frac{A_{B2.2}}{A_{inf} - A_{B1} - A_{B2.1}} \quad (27)$$

$$w_6 = \frac{A_{inf} - A_{B1} - A_{B2.1}}{A_C} \quad (28)$$

$$w_7 = \frac{A_{B1}}{A_C} \quad (29)$$

where:

- $A_C$  – total catchment area, km<sup>2</sup>;
- $A_{OC_{max}}$  – surface area of *Sb* direct runoff zone and the maximum range of variable source areas, km<sup>2</sup>;
- $A_{inf}$  – surface area of the *Sinf* infiltration zone, km<sup>2</sup>;
- $A_{A1}$  – surface of urbanized areas of A1 type, km<sup>2</sup>;
- $A_{A2}$  – surface of urbanized areas of A2 type, km<sup>2</sup>;
- $A_{B1}$  – surface of urbanized areas of B1 type, km<sup>2</sup>;
- $A_{B2.1}$  – surface of urbanized areas of B2.1 type, km<sup>2</sup>;
- $A_{B2.2}$  – surface of urbanized areas of B2.2 type, km<sup>2</sup>.

When calculating the values of individual weights, the reduced size of urban areas should be considered, taking into account only the sealed (impermeable) areas. The presented model assumes that they constitute 60% of the total size of a given area.

## IDENTIFICATION OF THE MODEL'S PARAMETERS

In order to identify and verify the model for a natural catchment, five hydrographs of total flow were recorded in the closing profile of the upper-course Mławka river catchment, in the period of flood flows in the years 1995, 2008, 2009, 2010 and 2013, along with their corresponding precipitation hyetographs and sequences of surface evapotranspiration values (see: Table 1). When selecting hydrographs, particular attention was paid to the quality and completeness of the measurement data. The average hourly sums of precipitation for given areas were calculated using the Thiessen polygons based on the results of measurements carried out in five precipitation stations located within the studied catchment area.

The calculations of the daily reference evaporation totals was conducted using the Penman-Monteith method (Łabędzki et al. 2011 quoting: Allen et al. 1998), on the basis of meteorological data obtained from the IMGW PIB weather station (level one) located in Mława, at a distance of 6 kilometres from the profile enclosing the analysed catchment. A simplifying assumption was made for the calculation of evapotranspiration values in hourly intervals. Within one day (24-hour period), only the difference between the total evapotranspiration during the period in which precipitation occurs, and that for a rainless period was taken into account. Based on the analysis of the value of evapotranspiration totals during the period of rainfall, it was found that its average intensity in such conditions for the tested catchment is 0.05 mm · h<sup>-1</sup>. Other hourly values were calculated as the quotient of the residual daily total of evapotranspiration and the number of hours without precipitation, so as to obtain compliance with the previously calculated daily total.

The model of the natural catchment described here has 11 optimized parameters. Two parameters by rule assume the same value ( $c_2 = c_3$ ), hence the final number of optimized parameters amounted to 10:

- $e$  – Penman reference evapotranspiration adjustment index;
- $B$  – divisor in the DZPE procedure for distribution of precipitation and evaporation;
- $b$  – exponent in the DZPE procedure for distribution of precipitation and evaporation;
- $Z_p$  – threshold value for filling the soil-ground retention reservoir, above which the supply of subsurface runoff begins;
- $c_1$  – discharge coefficient of the soil-ground retention reservoir,  $c_1 = c_3$ ;
- $c_2$  – discharge coefficient of the reservoir transforming surface runoff;
- $c_3$  – discharge coefficient from the cascade of reservoirs transforming subsurface runoff;
- $m$  – exponent in the equation of output of the cascade of reservoirs transforming sub-surface runoff;
- $c_4$  – discharge coefficient of the linear groundwater runoff reservoir;
- $w$  – weigh coefficient of the weighted sum of the direct runoff and groundwater runoff;
- $c_5$  – discharge coefficient of the linear reservoir transforming the flow in the riverbed.

The optimum values of the model's parameters were determined using the Hooke-Jeeves method (Hooke and Jeeves 1961). The latter is used for one of the optimization procedures available in the SYSMOR system. However, the original algorithm of the Hooke-Jeeves method has been modified so that it is possible to search for the minimum of the objective function, taking into account the constraints imposed on the variability interval of the optimized parameters. This has been accomplished by loading the objective function with a penalty function that increases the value of the objective function incrementally, in case the iterated value of the parameter exceeds the defined variability range.

In order to identify the model's parameters, the objective function  $F$  was used in the form of the sum of squared deviations between the total flow values calculated with the  $Q_{sym,i}$  model, and the observed flow of  $Q_{obs,i}$ , as follows:

$$F = \sum_{i=1}^N (Q_{b_{sym,i}} - Q_{b_{obs,i}})^2 \rightarrow \min \quad (30)$$

As a result of optimization, optimal parameter values are determined,  $x_1^o, x_2^o, \dots, x, \dots, x_k^o$ , for which the objective function reaches its minimum:

$$F \min = F(x_1^o, x_2^o, \dots, x, \dots, x_k^o) \quad (31)$$

where:  $F \min$  – minimum value of the criterion function;

The SYSMOR optimization procedure, after defining the optimal parameter values, also determines the standard estimation errors  $\sigma_i$  (Szymczak 2003) and 1/2 width of the 95% confidence interval for the given parameter  $x_i^o - HWCI(x_i^o)$ .

An additional parameter of the model is the  $n$  number of reservoirs in the cascade, by means of which the process of sub-surface flow formation is mapped. The application of the Hooke-Jeeves method did not facilitate an automatic optimization of this parameter. For this reason, the optimization process had been repeated multiple times, in order to determine the optimal number of reservoirs in the cascade by the iterative method, and to determine whether individual linear reservoirs are sufficient for the conceptual description of surface runoff processes, groundwater runoff, and flow transformation in the riverbed. The best results in this respect were obtained for the structure of the model that took into account the subsurface runoff cascade consisting of  $n = 5$  nonlinear reservoirs, and of individual linear reservoirs for the other processes mentioned above.

The parameters of the model estimated on the basis of material from observations carried out in the controlled catchments are often characterized by high variance (Allred and Haan, 1991), among other things due to the excessive fit of the model to data burdened with measurement errors. In order to determine the average optimal parameter values, the identification process was carried out simultaneously for three measurement series coming from different time periods. These were flow hydrographs, from 1995, 2008, and 2013 respectively (see: Table 1), and their corresponding precipitation hyetographs as well as evapotranspiration sequences.

The optimization process began with determining the values of the initial parameters and coordinates of state, from  $Z_1$  do  $Z_5$ . The initial values of the  $Z_2$  coordinates (filling the linear reservoir of surface runoff)

**Table 1.** List of parameters of the used total runoff hydrographs

Hydrograph parameters	1995	2008	2009	2010	2013
Beginning of the hydrograph (date, hour)	1995-09-02 02:00	2008-04-29 00:00	2009-07-07 00:00	2010-05-28 14:00	2013-06-02 00:00
End of the hydrograph (date, hour)	1995-09-12 01:00	2008-05-16 23:00	2009-07-15 23:00	2010-06-11 23:00	2013-06-11 23:00
Time duration of the hydrograph – $t_h$ , h	240	432	216	192	240
Beginning of the main flood flow (date, hour)	1995-09-03 01:00	2008-04-30 04:00	2009-07-08 03:00	2010-05-30 14:00	2013-06-06 16:00
Peak of the main flood flow (date, hour)	1995-09-04 12:00	2008-05-03 01:00	2009-07-08 23:00	2010-05-31 19:00	2013-06-07 19:00
Initial streamflow of the hydrograph – $Q_p$ , m <sup>3</sup> /s	0.3989	0.2191	0.1332	0.2123	0.2043
Initial streamflow of the main freshet – $Q_o$ , m <sup>3</sup> /s	0.3791	0.2226	0.1301	0.2151	0.2725
Maximum streamflow – $Q_{max}$ , m <sup>3</sup> /s	3.0221	0.7634	1.0546	0.9091	1.4096
End streamflow of the hydrograph – $Q_k$ , m <sup>3</sup> /s	0.4757	0.1989	0.1438	0.2272	0.2451
Time of the main freshet – $t_p$ , h	35	69	20	29	27
Total precipitation – $\Sigma P$ , mm	129.5	44.2	58.0	72.6	70.2
Precipitation amount causing the main freshet – $\Sigma P_w$ , mm	98.3	41.9	50.2	43.2	36.0
Maximum precipitation – $I$ , mm · h <sup>-1</sup>	12.5	9.3	17.6	7.0	25.0
Minimum/average/maximum daily (24-hour) total evapotranspiration/ Penman index – $E$ , mm	0.24/1.44/3.60	1.44/3.17/4.32	2.16/3.55/4.32	1.68/3.27/5.04	2.88/3.82/4.56
Total runoff layer – $\Sigma H_c$ , mm	13.47	7.16	3.34	4.57	5.52

Source: own study

and  $Z_3$  (filling the nonlinear cascade reservoirs of the subsurface runoff) were assumed to be equal to zero, because the comparative hydrographs were selected so that there was no direct runoff for at least the first several hours. 9 initial values of the coordinates of state (three for each freshet) corresponding to the initial fillings of the soil-ground water reservoir, groundwater reservoir, and riverbed retention reservoir remained to be determined. They were determined together with ten other values of the model parameters, by performing initial optimization with the Monte Carlo method. As a set of input values of model parameters and co-

ordinates of state, a set of variables was adopted, for which after 500 thousand iterations a set of quasi-optimal parameter values was obtained; for the latter, the relative mean square residual error of the model was less than 0.40. This was used as the input set of 19 independent variables, which have been optimized using the Hooke-Jeeves method. A further approximation of the optimal values was thus obtained, which were used again as input values in the next optimization process. However, this time only 10 parameters of the model were optimized for the previously determined initial state coordinate values (see: Table 2).

**Table 2.** Optimum values of initial (input) states of the model's systems

Initial (input) state of the system	Optimum value for the flood flow of the given year, in mm		
	1995	2008	2013
$Z_1$	30.78	41.79	45.12
$Z_4$	409.5	259.1	196.5
$Z_5$	0.3084	0.1653	0.1599

Source: own study

In this way, the final optimal values of 10 model parameters were calculated (see: Table 3). It should be noted that one of the optimized parameters was the weight coefficient  $w$ , the value of which corresponds to the maximum direct supply coefficient  $\alpha_{\max}$ . According to its calculated value (see: Table 3), the maximum range zone of  $Sb$  variable source areas within the studied catchment area constitutes only 8.3% of the total catchment area.

The values of standard errors for estimating model parameters do not exceed 6% of the optimal value.

The lowest accuracy of the estimation was recorded for parameter  $B$  – the divisor in the procedure of distribution of the precipitation balanced with evapotranspiration, between the supply of surface runoff and infiltration  $\delta = 5.75\%$ . The highest accuracy of the estimation is noted for the discharge coefficient in the linear groundwater reservoir  $c_4$ , for which the relative standard error equalled  $\delta = 0.12\%$ .

### VERIFICATION OF THE MODEL

Model verification consisted in the comparison between the simulated and the actually observed hydrographs, and the calculation of the following quality measures:

- $DW$  – relative mean square residual error (coefficient of residual error);
- $EF$  – Nash-Sutcliffe efficiency factor (1970);
- $CRM$  – coefficient of residual mass;
- $SWS$  – ratio of means;
- $SWM$  – ratio of maximums.

The verification was carried out with both dependent and independent material. In the first case, quality measures were determined based on the simulated and

**Table 3.** Optimum values of the model's parameters with measures of their estimation accuracy

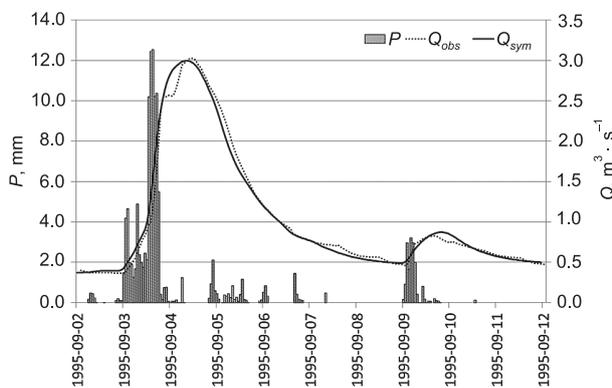
Parameter	Unit	Optimum value $x^0$	Standard error $\sigma$	Relative standard error $\delta, \%$	1/2 width of 95% confidence interval for parameter $HWCI(x^0)$	Upper and lower limit of the confidence interval	
						$x^0 - HWCI(x^0)$	$x^0 + HWCI(x^0)$
$e$	–	1.120	0.02609	2.33	0.0511	1.0693	1.1715
$B$	–	4.573	0.2630	5.75	0.5155	4.0572	5.0882
$b$	–	0.4142	0.009723	2.35	0.0191	0.3952	0.4333
$Z_p$	mm	56.23	0.2732	0.49	0.5355	55.70	56.77
$c_2$	H <sup>-1</sup>	0.1243	0.0065	5.21	0.0127	0.1116	0.1370
$c_1 = c_3$	H <sup>-1</sup>	0.4206	0.0153	3.64	0.0300	0.3906	0.4506
$m$	–	0.7450	0.0250	3.36	0.0491	0.6959	0.7941
$c_4$	H <sup>-1</sup>	0.000546	0.0000006	0.11	0.0000012	0.000545	0.000547
$w$	–	0.08306	0.00069	0.83	0.00135	0.08171	0.08441
$c_5$	H <sup>-1</sup>	0.06530	0.00183	2.80	0.00358	0.06172	0.06888

Source: own study

the observed hydrographs from 1995, 2008 and 2013, used in the identification process (see: Fig. 8, 9 and 10). In order to verify the model with the independent material, simulations of flow hydrographs for the years 2009 and 2010 were performed (see: Fig. 11 and 12). Initial values of the  $Z_4$  and  $Z_5$  coordinates can be calculated for the described model on the basis of the initial flow rates in the river, by applying the equations (16), (17), and (18). The value of  $Z_1$  coordinate was set as identical for both hydrographs, equal to half of the threshold value of soil-ground water reservoir output, amounting to  $Z_1 = 28.1$  mm.

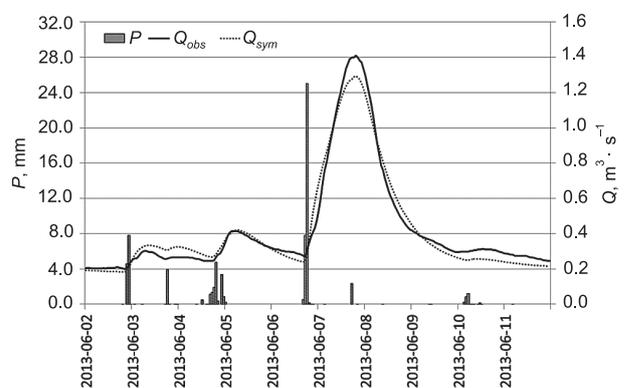
The remaining coordinates of state were assigned initial values of zero, because the initial flow of the comparative hydrographs contained only the groundwater component.

The Nash-Sutcliffe  $EF$  efficiency factor is a statistical measure often used in the context of assessing the correctness of hydrological models. Its value determines the degree of conformity of the simulated process with the actual (observed) course of that process. The closer to 1 the  $EF$  is, the more efficient the model. The lowest value of the efficiency factor  $EF$  was 0.9371 (see: Table 4).



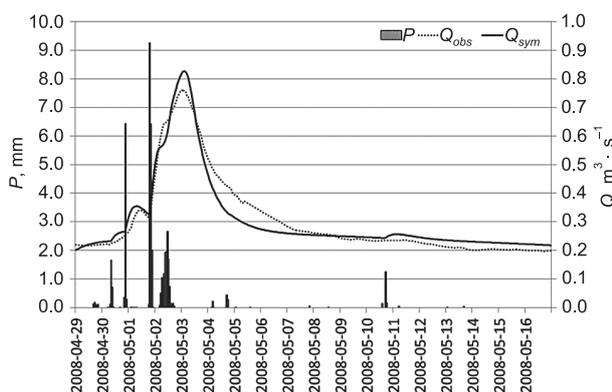
**Fig. 8.** Comparison between the hydrograph of the total observed runoff  $Q_{obs}$  and of the simulated runoff  $Q_{sym}$  for the flood wave of 1995

Source: own study



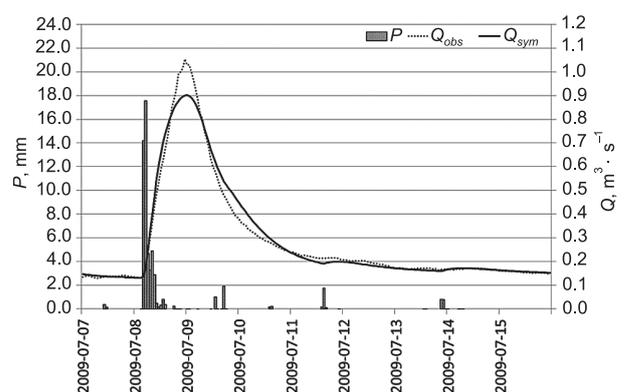
**Fig. 10.** Comparison between the hydrograph of the total observed runoff  $Q_{obs}$  and of the simulated runoff  $Q_{sym}$  for the flood wave of 2013

Source: own study



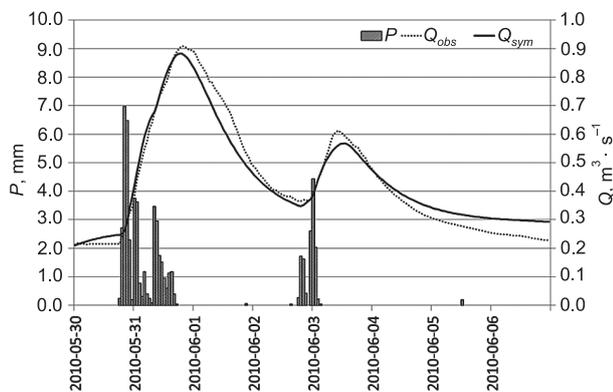
**Fig. 9.** Comparison between the hydrograph of the total observed runoff  $Q_{obs}$  and of the simulated runoff  $Q_{sym}$  for the flood wave of 2008

Source: own study



**Fig. 11.** Comparison between the hydrograph of the total observed runoff  $Q_{obs}$  and of the simulated runoff  $Q_{sym}$  for the flood wave of 2009; verification using independent material

Source: own study



**Fig. 12.** Comparison between the hydrograph of the total observed runoff  $Q_{obs}$  and of the simulated runoff  $Q_{sym}$  for the flood wave of 2010; verification using independent material

Source: own study

Taking into account the criteria given in Table 5, the quality of the model should be considered as excellent.

The assessment of the quality of the model based on the value of the residual error coefficient of variation renders a slightly worse result. Its value varied from 0.0590 to 0.1191. The use of this particular measure to assess hydrological models is recommended by the World Meteorological Organization (WMO). According to the results obtained for this measure, the quality of the model is on the borderline of good and very good. However, this is largely due to the method of calculating the aforementioned error. Its value is dependent and inversely proportional to the intensity of the average flow from the compared hydrograph. The value of the  $DW$  index increases with the number of sections added to the analysed flood flow hydrograph,

**Table 4.** Values of quality measures for the model

Quality measure	Dependent material – hydrographs from the years:			Independent material – hydrographs from the years:	
	1995	2008	2013	2009	2010
Efficiency factor – $EF$	0.9937	0.9371	0.9728	0.9771	0.9594
Relative mean square residual error – $DW$	0.0590	0.1177	0.1173	0.1191	0.0930
Ratio of means – $SWM$	0.989	1.080	0.917	0.854	0.971
Ratio of maximums – $SWS$	1.002	0.999	0.987	1.005	1.010
Coefficient of residual mass – $CRM$	-0.0024	0.0011	0.0131	-0.0048	-0.0098

Source: own study

**Table 5.** Value ranges of the quality measures of the model used for their evaluation

Coefficient of variation of the residual error – $DW$ Value range	Efficiency coefficient – $EF$ Value range	Quality categories of the model
< 0.05	> 0.85	excellent
0.05 – 0.10	0.65 – 0.85	very good
0.10 – 0.20	0.50 – 0.65	good
0.20 – 0.40	0.20 – 0.50	poor
> 0.40	< 0.20	unsatisfactory

Source: own study based on Andersen et al. (2001)

including high or low flows (preceding the flood flow, separating two flood flows or occurring during the runoff recession). Due to the inclusion of the groundwater runoff component in the model, such hydrographs were used for identification, which included prolonged periods with only this form of runoff. Therefore, the obtained values of relative errors  $DW$  are in some cases greater than that it would result from the assessment of the model based on other quality measures. Comparison of the remaining values, that is of the  $SWM$ ,  $SWS$  and  $CRM$  measures with the criteria given in Table 6 leads to the conclusion that the quality of the developed model is most definitely better than satisfactory.

**Table 6.** Values of quality measures determining the satisfactory model

Quality measure	Value of the measure determining the satisfactory model
Ratio of maximums – <i>SWM</i>	$0.75 < SWM < 1.25$
Ratio of means – <i>SWS</i>	$0.75 < SWS < 1.25$
Coefficient of residual mass – <i>CRM</i>	$-0.25 < CRM < 0.25$

Source: own study based on Sarma et al. (1973)

## CONCLUSIONS

The mapping of the process of total river runoff formation from a small lowland river catchment used for agriculture, including its main components, is possible with the help of a simple conceptual model with 10 optimized parameters. Hydrological processes can be described using state-space and output equations. A sufficient set of input data is provided in the sequences of hourly sums of total precipitation averaged for the catchment area and of the evapotranspiration. The hourly sums of evapotranspiration can be determined in a simplified way by the appropriate processing of daily sums of reference evapotranspiration index calculated using the Penman-Monteith method.

The description of the river runoff formation process should take into account the theory of variable source (active) areas, especially in the case of a small lowland drainage catchment with high infiltration capacity. In this case, the model structure should distinguish two types of areas: those where only direct runoff is created, and those where the water coming from precipitation after taking into account evapotranspiration losses, completely infiltrates and supplies the groundwater runoff. Within the studied catchment of the upper-course of the Mławka river, the first type of areas accounted for only 8.3% of its total area. This is very important from the point of view of assessing the scale of impact on river runoff of land sealing in the second zone, occupying 91.7% of the catchment area, where there is no surface runoff under natural conditions.

Considering the manner of affecting hydrological processes, urbanized areas can be classified into five distinctive types.

Based on the results from the verification of the developed model, it can be concluded that with hydro-meteorological data of appropriate accuracy, such a model can simulate total flow hydrographs with an average accuracy of 12% (according to *DW* quality measure) and calculate the maximum flow with an accuracy of 15% (according to *SWM* quality measure). The above statements concern the controlled catchment, and the model after prior parameter identification using optimization methods.

It is possible to expand the model after its identification in natural catchment conditions, by adding components that will serve to model the runoff from partially urbanized sub-catchments, using verified and published calculation procedures with parameters determined on the basis of measurable characteristics of these catchments.

Further work and research efforts should be devoted to adapting the developed model to the conditions of uncontrolled catchments due to the fact that there are very few controlled catchments that would provide data to identify parameters by means of optimization methods. It would be advisable to reduce the number of optimized model parameters, and to determine the correlations between them and appropriately selected characteristics of the catchment, while conducting research in a greater number of facilities. In the case of a catchment with low infiltration capacity, the model should be supplemented with procedures for modelling also the infiltration excess runoff, for instance according to the Horton theory.

The model described in the present paper has been used to carry out simulation studies into the impact of urbanization on changes in the formation of the total runoff and its components as illustrated with the example of the Upper Mławka river catchment (Krężałek 2018).

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## **MODEL PROGNOSTYCZNY ODPLYWU CAŁKOWITEGO I JEGO SKŁADOWYCH Z MAŁEJ NIZINNEJ ZLEWNI CZĘŚCIOWO ZURBANIZOWANEJ**

### **ABSTRAKT**

Celem pracy jest przedstawienie nowego modelu hydrologicznego (opracowanego w Instytucie Technologiczno-Przyrodniczym), który może być stosowany do badań symulacyjnych kształtowania się odpływu całkowitego i jego składowych z małej nizinnej zlewni rolniczej w różnych warunkach meteorologicznych, z uwzględnieniem różnych początkowego stanu retencji zlewni i oddziaływania na ten odpływ obszarów zurbanizowanych.

Opracowanie wiązało się z koniecznością dokonania wyboru rodzaju modelu i sposobu opisu procesów hydrologicznych. Na podstawie studiów literaturowych i wyników wcześniejszych badań oraz obserwacji prowadzonych w zlewni górnej Mławki (lewy dopływ Wkry) przyjęto, że najodpowiedniejszym narzędziem do celów prognostycznych i badań symulacyjnych będzie model konceptualny, uwzględniający w swej strukturze i opisie procesów hydrologicznych teorię obszarów czynnych. Założono, że opracowany zostanie możliwie prosty model, który stwarzałby możliwość symulowania hydrogramów odpływu całkowitego, powierzchniowego, podpowierzchniowego i gruntowego ze zlewni naturalnej oraz zlewni w różnym stopniu zurbanizowanej. Praca obejmuje także przedstawienie propozycji zasad klasyfikacji obszarów zurbanizowanych, uwzględniającej ich lokalizację na terenie zlewni i sposób odprowadzania ścieków opadowych oraz opis odpowiednich modułów modelu służących do symulacji odpływu z cząstkowych zlewni zurbanizowanych.

Model opisany jest równaniami stanu i równaniami wyjść. Pracuje z krokiem czasowym równym jednej godzinie. Wejście stanowi opad całkowity i parowanie wskaźnikowe. Na wyjściu otrzymujemy hydrogram odpływu całkowitego oraz hydrogramy składowych tego odpływu. Model ma 10 optymalizowanych parametrów występujących w procedurach symulujących procesy w naturalnej – nieprzekształconej części zlewni oraz 10 mierzalnych parametrów służących do opisu cząstkowych zlewni zurbanizowanych.

W artykule przedstawiono wyniki identyfikacji parametrów modelu oraz jego weryfikacji. Do tego celu wykorzystano 5 hydrogramów odpływu całkowitego o długości od 192 do 432 godzin obejmujących wezbrania opadowe. Były one zarejestrowane w zlewni badawczej górnej Mławki. Otrzymane wartości funkcji kryterialnych pozwalają przyjąć, że jakość modelu jest bardzo dobra. Model został wykorzystany do badań symulacyjnych wpływu urbanizacji zlewni rolniczej na parametry odpływu rzeczno (Krężałek 2018).

**Słowa kluczowe:** odpływ rzeczny, zlewnia rzeczna, urbanizacja, obszary czynne, model konceptualny