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APPLICATION OF NUMERICAL SIMULATION
IN DESIGN OF INNOVATIVE
KALIPSO-TYPE SEWAGE TANK

Wastewater flow control in sewerage systems is crucial for their effective operation but it is equally important from the point of view of wastewater treatment efficiency and surface waters quality. The paper presents the foundations of mathematical modelling and development of simulation tools for the solutions of a retention reservoir. On the grounds of a case study, possibilities are presented to use the developed program for the dimensioning of reservoirs of a defined type based on theoretical wastewater flow functions and hydrographs obtained on the grounds of the catchment hydrodynamic model.

1. INTRODUCTION

The development of urbanized areas results in tightened requirements in the scope of drainage systems guaranteeing a definite safety level and operational reliability. The increase of watertight surface areas within the catchment area results in intensified runoff of rainwater that consequently feeds sewerage systems.

The phenomenon, commonly observed in contemporary municipal areas, results in a number of negative ecological and hydraulic consequences, in catchment areas as such as well as in sewerage systems, WWTPs and surface waters.

In Western Europe countries, Japan and the United States, the issue of unbalanced rainwater management in municipal catchments was discerned as early as the last century and the countries adopted appropriate legislation, guidelines and standards regulating rainfall wastewater management methods. In Poland, the problem has not been solved properly and completely to date.

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Following the example of developed countries, activities aimed at balanced rainwater management should be focused in the following aspects:

– limitation of watertight surface shares,

– limitation of precipitation wastewaters introduced to surface waters,

– development of calculation methods and technical solutions concerning facilities used for the introduction of rainwater to the ground [1], [2], [3],

– economical utilization of precipitation wastewaters [4], [5], [6], [7], [8],

– rainwater wastewater storage and reducing their outflow rate [9], [10], [11], [12].

The increase of rainwater volumes entering sewerage systems in many cases presents a grave technical and financial problem and is related to the necessity to increase throughput capacity of sewers, extension of existing sewerage facilities and, sometimes, the construction of new ones.

Substantial progress in the area of sewage transport should be associated with the development of the theory and practice of sewage retaining facilities in periods of culminating runoff and the possibilities of controlling the operation of sewerage facilities in real time (RTC) [13], [14], [15].

The paper presents the process of the development of a simulation model and the obtained results of studies on an innovative *KALIPSO*-type retention reservoir solution with the chosen configuration of chambers used for hydraulic relieving of sewerage system in order to protect it against hydraulic overload caused by excess sewage volumes in the course of rainfall wastewater runoff.

2. HYDRAULIC SYSTEM OF *KALIPSO* TANK

The *KALIPSO*-type retention reservoir, according to the original solution presented in patent description [16], is composed of: accumulation chamber *KA*, pumping chamber *KP*, sewage distribution chamber *KR* and discharge chamber *KZ*. The whole hydraulic system conditioning the proper operation of component facilities both in its filling and emptying phases is completed with inter-chamber overflows, pumping systems, gate valves, reflux valves and flow regulators [17]–[19].

For operational reasons, it is favourable to divide the accumulation chamber space into smaller parts through the creation of independently operating sections. They may be connected either in parallel or in series with respect to each other and the remaining chambers. Mutual position of accumulation chamber sections and devices cooperating with them or absence of such equipment determines, to a large extent, hydraulic conditions in the reservoir and design parameters of these devices.

In the paper, research on *KALIPSO*-type retention reservoir is exemplified by accumulation chamber sections connected in parallel to each other. Figure 1 shows a schematic diagram of the hydraulic system of a reservoir with three accumulation sections in top view projection.
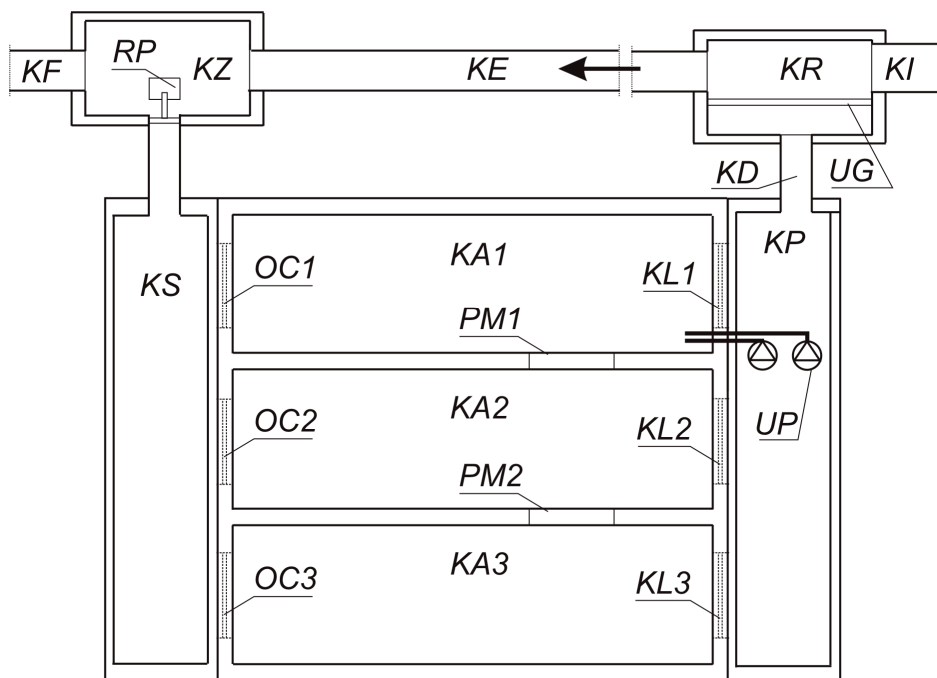
Fig. 1. Hydraulic system of *KALIPSO* retention reservoir with parallel layout
of accumulation chamber sections:
KA1 – accumulation chamber, first section; *KA2* – accumulation chamber, second section;
KA3 – third section of the accumulation chamber; *KD* – inflow channel of pumping chamber *KP*;
KE – outflow channel of the sewage distribution chamber *KR* and inflow channel of
discharge chamber *KZ*; *KF* — outflow channel of discharge chamber *KZ*; *KI* – inflow channel of
wastewater distribution chamber *KR*; *KL1* – reflux valve of the first section *KA1*;
KL2 – reflux valves of the second section *KA2*; *KL3* – reflux valves of the third section *KA3*;
KP – pumping chamber; *KR* – sewage distribution chamber; *KS* – outflow chamber;
KZ – discharge chamber; *PM1* – overflow between first and second sections of the accumulation
chamber; *PM2* – overflow between second and third sections of the accumulation chamber;
RP – sewage flow regulator; *UG* – distribution overflow; *UP* – pumping system

3. DYNAMIC MODEL OF *KALIPSO* TANK

The base for the development of calculational simulation models consists in mathematical description of actual hydraulic processes. According to the law of the conservation of mass, the general mathematical wastewater balance model for all phases significant from the point of view of the operation of *KALIPSO*-type retention reservoir is determined by the following set of equations:

$$\begin{cases} F_{KR} \cdot dhr = Qd(t) \cdot dt - Qe(hr) \cdot dt - Qg(hp, hr) \cdot dt, \\ F_{KP} \cdot dhp = Qg(hp, hr) \cdot dt - Qp(ha_1, hp) \cdot dt - Ql_1(ha_1, hp) \cdot dt \\ \qquad\qquad - Ql_2(ha_2, hp) \cdot dt - Ql_3(ha_3, hp) \cdot dt, \\ F_{KA1} \cdot dha_1 = Qp(ha_1, hp) \cdot dt + Ql_1(ha_1, hp) \cdot dt \\ \qquad\qquad - Qm_1(ha_1, ha_2) \cdot dt - Qc_1(hs) \cdot dt, \\ F_{KA2} \cdot dha_2 = Ql_2(ha_2, hp) \cdot dt + Qm_1(ha_1, ha_2) \cdot dt \\ \qquad\qquad - Qm_2(ha_2, ha_3) \cdot dt - Qc_2(hs) \cdot dt, \\ F_{KA3} \cdot dha_3 = Ql_3(ha_3, hp) \cdot dt + Qm_2(ha_2, ha_3) \cdot dt - Qc_3(hs) \cdot dt, \\ F_{KS} \cdot dhs = Qc_1(hs) \cdot dt + Qc_2(hs) \cdot dt + Qc_3(hs) \cdot dt - Qz(hz) \cdot dt, \\ F_{KZ} \cdot dhz = Qe(hr) \cdot dt + Qz(hz) \cdot dt - Qf(hz) \cdot dt, \end{cases} \qquad (1)$$

where:

F_{KR} – horizontal projection surface area of the distribution chamber KR, m^2;

F_{KP} – horizontal projection surface area of the pump chamber KP, m^2;

F_{KA1} – horizontal projection surface area of the accumulation chamber's first section $KA1$, m^2;

F_{KA2} – horizontal projection surface area of the accumulation chamber's second section $KA2$, m^2;

F_{KA3} – horizontal projection surface area of the accumulation chamber's third section $KA3$, m^2;

F_{KS} – horizontal projection surface area of the outflow chamber KS, m^2;

F_{KZ} – horizontal projection surface area of the discharge chamber KZ, m^2;

dhr – filling level change in distribution chamber KR within time interval dt, m;

dhp – filling level change in pumping chamber KP within time interval dt, m;

dha_1 – filling level change in the accumulation chamber's first section $KA1$ within time interval dt, m;

dha_2 – filling level change in the accumulation chamber's second section $KA2$ within time interval dt, m;

dha_3 – filling level change in the accumulation chamber's third section $KA3$ within time interval dt, m;

dhs filling level change in outflow chamber KS within time interval dt, m;

dhz – filling level change in discharge chamber KZ within time interval dt, m;

Qd – wastewater inflow rate to the distribution chamber KR, m^3/s;

Qe – wastewater outflow rate from distribution chamber KR towards discharge chamber KZ, m^3/s;

Qg – wastewater outflow rate through overflow from distribution chamber KR to pump chamber KP, m^3/s;

Qp – pumping system capacity, m^3/s;

Ql_1 – wastewater flow rate through reflux valve from pump chamber KP to the accumulation chamber's first section $KA1$, m^3/s;

Ql_2 – wastewater flow rate through reflux valve from pump chamber KP to the accumulation chamber's second section $KA2$, m^3/s;

Ql_3 – wastewater flow rate through reflux valve from pumping chamber KP to the accumulation chamber's third section $KA3$, m^3/s;

Qm_1 – wastewater flow rate through overflow from the accumulation chamber's first section $KA1$ to the accumulation chamber's second section $KA2$, m^3/s;

Qm_2 – wastewater flow rate through overflow from the accumulation chamber's second section $KA2$ to the accumulation chamber's third section $KA3$, m^3/s;

Qc_1 – wastewater outflow rate from the accumulation chamber's first section $KA1$ to outflow chamber KS, m^3/s;

Qc_2 – wastewater outflow rate from the accumulation chamber's second section $KA2$ *to* outflow chamber KS, m^3/s;

Qc_3 – wastewater outflow rate from the accumulation chamber's third section $KA3$ to outflow chamber KS, m^3/s;

Qz – wastewater outflow rate from outflow chamber KS to discharge chamber KZ, m^3/s;

Qf – wastewater outflow rate from discharge chamber KZ, m^3/s.

The presented mathematical model of the reservoir includes all hydraulic processes occurring in its chambers and sections and therefore it is subject to modifications and simplifications in individual stages of its operation.

Analyzing the model of *KALIPSO* retention reservoir operating in a sewerage system, one can distinguish the following phases of its functioning in the course of the sewage accumulation process and the corresponding hydraulic boundary conditions that should be fulfilled for these phases:

I. Filling of pump chamber KP. Hydraulic boundary conditions for the phase: $hr > hb$, $hp < hk_1$, $hp < hk_2$, $hp < hk_3$, $hp < Hw_{max}$.

IIA. Filling the first section $KA1$ of accumulation chamber KA through a reflux valve. Hydraulic boundary conditions for the phase: $hr > hb$, $hp \geq hk_1$, $hp < hk_2$, $hp < hk_3$, $hp < Hw_{max}$.

IIB. Filling the second section $KA2$ of accumulation chamber KA through a reflux valve. Hydraulic boundary conditions for the phase: $hr > hb$, $hp \geq hk_2$, $hp < hk_3$, $hp < Hw_{max}$.

IIC. Filling the third section $KA3$ of accumulation chamber KA through a reflux valve. Hydraulic boundary conditions for the phase: $hr > hb$, $hp \geq hk_3$, $hp < Hw_{max}$.

III. Filling the pumping section KP. Hydraulic boundary conditions for the phase: $hr > hb$, $hk_3 \geq hg$, $hp \leq Hw_{max}$.

IVA. Pump-assisted filling the first section $KA1$ of KA accumulation chamber. Hydraulic boundary conditions for the phase: $hr > hb$, $hp > Hw_{min}$, $ha_1 < hu_1$.

IVB. Pump-assisted filling the first section $KA1$ of accumulation chamber KA and the second section $KA2$ of accumulation chamber KA through inner overflow $PM1$ hydraulically operating as a non-drowned one. Hydraulic boundary conditions for the phase: $hr > hb$, $hp > Hw_{min}$, $ha_1 \geq hu_1$, $ha_2 < hu_1$, $ha_2 < hu_2$.

IVC. Pump-assisted filling the first section $KA1$ of accumulation chamber KA and the second section $KA2$ of accumulation chamber KA through inner overflow $PM1$ hydraulically operating as a drowned one. Hydraulic boundary conditions for the phase: $hr > hb$, $hp > Hw_{min}$, $ha_1 > hu_1$, $ha_2 > hu_1$, $ha_2 < hu_2$.

IVD. Pump-assisted filling the first section $KA1$ of accumulation chamber KA, the second section $KA2$ of accumulation chamber KA through inner overflow $PM1$ hydraulically operating as a drowned one and the third section $KA3$ through inner overflow $PM2$ hydraulically operating as a non-drowned one. Hydraulic boundary conditions for the phase: $hr > hb$, $hp > Hw_{min}$, $ha_1 > hu_1$, $ha_2 > hu_1$, $ha_2 > hu_2$, $ha_3 < hu_2$.

IVE. Pump-assisted filling the first section $KA1$ of KA accumulation chamber, the second section $KA2$ of accumulation chamber KA through inner overflow $PM1$ hydraulically operating as a drowned one and third section $KA3$ through inner overflow $PM2$ hydraulically operating as a drowned one. Hydraulic boundary conditions for the phase: $hr > hb$, $hp > Hw_{min}$, $ha_1 \geq hu_1$, $ha_2 \geq hu_1$, $ha_2 \geq hu_2$, $ha_3 \geq hu_2$.

VA. Evacuation of the first section $KA1$ of accumulation chamber KA through an opening hydraulically operating as a pressure orifice. Hydraulic boundary conditions for the phase: $hr \leq hb$, $hp \leq Hw_{min}$, $ha_1 > hr$.

VB. Evacuation of the first section $KA1$ of accumulation chamber KA through an opening hydraulically operating as an orifice with gravitational outflow. Hydraulic boundary conditions for the phase: $hr \leq hb$, $hp \leq Hw_{min}$, $ha_1 \leq hr$.

VC. Evacuation of the second section $KA2$ of accumulation chamber KA through an opening hydraulically operating as a pressure orifice. Hydraulic boundary conditions for the phase: $hr \leq hb$, $hp \leq Hw_{min}$, $ha_2 > hr$.

VD. Evacuation of the second section $KA2$ of accumulation chamber KA through an opening hydraulically operating as an orifice with gravitational outflow. Hydraulic boundary conditions for the phase: $hr \leq hb$, $hp \leq Hw_{min}$, $ha_2 \leq hr$.

VE. Evacuation of the third section $KA3$ of accumulation chamber KA through an opening hydraulically operating as a pressure orifice. Hydraulic boundary conditions for the phase: $hr \leq hb$, $hp \leq Hw_{min}$, $ha_3 > hr$.

VF. Evacuation of the third section $KA3$ of accumulation chamber KA through an opening hydraulically operating as an orifice with gravitational outflow. Hydraulic boundary conditions for the phase: $hr \leq hb$, $hp \leq Hw_{min}$, $ha_3 \leq hr$.

The following symbols were used above:

hb – overflow edge elevation in sewage chamber KR, m;

hk_1 – reflux valve opening level $KL1$, m;

hk_2 – reflux valve opening level $KL2$, m;

hk_3 – reflux valve opening level $KL3$, m;

Hw_{max} – pump system switch-on level, m;
Hw_{min} – pump system switch-off level, m;
hu_1 – $PM1$ overflow edge elevation, m;
hu_2 – $PM2$ overflow edge elevation, m;
hu_3 – $PM3$ overflow edge elevation, m.

The application of the designed mathematical model of a retention reservoir to the development of software tools allowing for the simulation of its operation for arbitrary function describing wastewater flow in the sewerage system required more detailed elaboration of the mathematical description of individual hydraulic processes. At the same stage, the quantitative characterization of the mathematical model parameters was carried out leading to the distinguishment between: examined parameters, resultant parameters and fixed parameters (constants).

Based on the mathematical model, a software tool was developed in Matlab/Simulink environment making it possible to carry out a numerical simulation of wastewater retention process in the *KALIPSO* retention reservoir.

4. CASE STUDY

Calculational capabilities of the program and its usefulness in the process of innovative *KALIPSO* retention reservoir design process were verified on the grounds of a case study concerning a retention reservoir which was to be be constructed in *Zasanie,* a district of the town of Przemyśl. The *Czuwaj* retention reservoir is planned to be located on a by-pass of the town of the main collector of the combined sewerage system. The purpose of the retention reservoir consists in the provision of temporary storage for wastewaters overflowing to the relief channel from the main sewer channeling sewage to the left-bank pumping station which is overloaded in rainfall periods.

Input data for the retention reservoir simulation model were obtained from the sewerage system operator. It was assumed that simulation studies would be carried out for a theoretical wastewater flow rate function [20] and for hydrographs obtained on the grounds of hydrodynamical model of *Zasanie*, with the use of SWMM (*Storm Water Management Model*) EPA program.

The area in which the *Czuwaj* retention reservoir will be located was the subject of meteorological research in the scope of rain height measurements and, to some limited extent, of the quantitative monitoring of sewage flows in the sewerage system. Based on the data, the calibration of the catchment hydrodynamical model was performed.

Bearing in mind the limited availability of precipitation data coming only from the years 2007–2008 it should be admitted that they must not be considered a sufficient base for the dimensioning of the *Czuwaj* retention reservoir. However, they may be taken into account as a reference for calculations carried out for theoretical wastewater flow functions.

For initial selection, 42 rainfalls were adopted with duration periods from 60 to 6480 minutes. Simulation calculations of wastewater flow in the hydraulic model of the catchment were performed for all of them, and then two rainfalls were selected for the retention reservoir operation studies using the criterion of the largest wastewater discharge volumes from collector to relief channel, and then to the retention reservoir. Hydrographs of wastewater flows for these two rainfalls at the node in which the distribution chamber *KR* will be located are presented in figures 2 and 3.
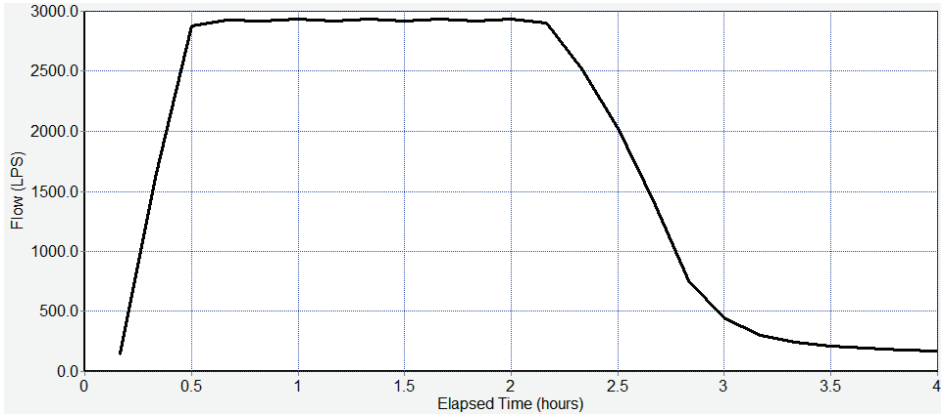

Fig. 2. Hydrograph of wastewater flow at the node in which the distribution chamber *KR* will be located for rainfall occurred on May 13, 2008
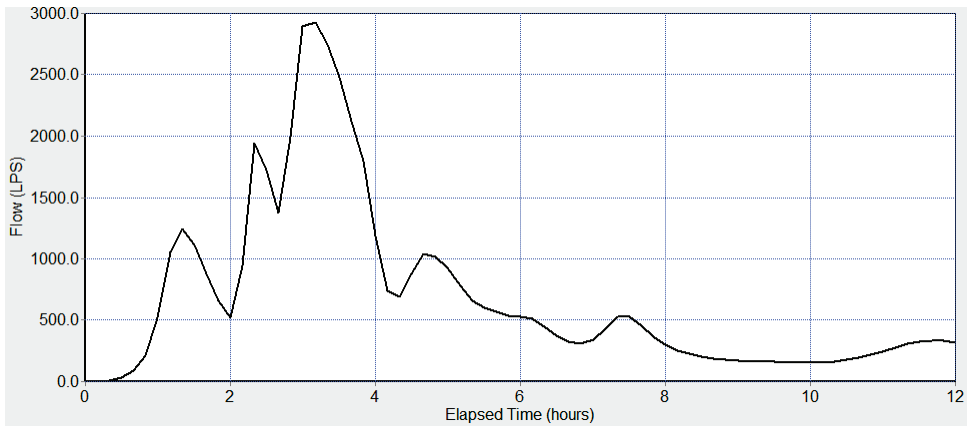

Fig. 3. Hydrograph of wastewater flow at the node in which the distribution chamber *KR* will be located for rainfall occurred on August 18, 2007

Reservoir simulation studies were also carried out for a theoretical wastewater flow profile function. Taking into account the results obtained by the present author

and concerning the selection of profiles of critical theoretical wastewater flow hydrographs for the purpose of retention reservoir design [20], the analysis was carried out with the function defined by equation (2) for time interval $0 < t \leq Tp$, equation (3) for $Tp \leq t \leq Tdm$, and equation (4) for $Tdm < t \leq Tp + Tdm$ [20]:

$$Qd = (qdm_{(Tdm)} \cdot Fzr) \cdot t^2 \cdot Tp^{-2} + Qs \,, \qquad (2)$$

where:

Fzr – reduced catchment surface area, ha;

$qdm_{(Tdm)}$ – rain intensity calculated for the design storm duration period Tdm adopted for retention reservoir design, dm^3/s·ha;

Qs – sanitary sewage outflow from catchment area, dm^3/s;

t – time, s;

Tp – sewage flow time from the most distant catchment point to the determined calculation cross-section, s;

$$Qd = Qs + (qdm_{(Tdm)} \cdot Fzr) \,, \qquad (3)$$

$$Qd = (qdm_{(Tdm)} \cdot Fzr) - ((qdm_{(Tdm)} \cdot Fzr) \cdot (t - Tdm)^2 \cdot Tp^{-2}) + Qs \,, \qquad (4)$$

where Tdm denotes the design storm duration period adopted for the retention reservoir design, s.

The values of the parameters examined and constants used in the *KALIPSO* retention reservoir simulation calculations are listed in the table.

T a b l e

Values of the parameters examined and constants used in calculations concerning the *Czuwaj* retention reservoir

Parameter name	Unit	Value
1	2	3
F_{KZ} – horizontal projection surface area of the discharge chamber *KZ*	m^2	20
F_{KR} – horizontal projection surface area of the distribution chamber *KR*	m^2	20
F_{KP} – horizontal projection surface area of the pump chamber *KP*	m^2	30
F_{KA1} – horizontal projection surface area of the accumulation chamber's first section *KA*1	m^2	500
F_{KA2} – horizontal projection surface area of the accumulation chamber's second section *KA*2	m^2	500
F_{KA3} – horizontal projection surface area of the accumulation chamber's third section *KA*3	m^2	500
F_{KS} – horizontal projection surface area of the outflow chamber *KS*	m^2	40
H – average annual precipitation level	mm	700
c – design storm occurrence frequency	–	5

1	2	3
Tp – sewage flow time from the most distant catchment point to the determined calculation cross-section	s	1600
Tdm – design storm duration period adopted for the retention reservoir design	s	10200
Qs – sanitary sewage outflow rate from the catchment area	dm^3/s	240
Fzr – reduced catchment surface area	ha	63
$qdm_{(Tdm)}$ – rain intensity calculated for the design storm duration period adopted for the retention reservoir design Tdm	$dm^3/s\cdot ha$	
Hb – UG overflow edge elevation	m	1.00
b – UG overflow edge length	m	5.00
μ_1 – UG overflow performance coefficient	–	0.45
μ_2, μ_3 – PM1 overflow performance coefficient	–	0.40
μ_4, μ_5 – PM2 overflow performance coefficient	–	0.40
μ_6, μ_7 – PM3 overflow performance coefficient	–	0.40
μ_8, μ_9, μ_{10} – OC1, OC2, OC3 orifice output coefficients	–	0.60
hk_1 – KL1 reflux valve opening level	m	0.05
hk_2 – KL2 reflux valve opening level	m	0.25
hk_3 – KL3 reflux valve opening level	m	0.35
Hw_{max} – pumping system's switch-on level	m	1.00
Hw_{min} – pumping system's switch-off level	m	0.80
hu_1, hu_2, hu_3 – PM1, PM2, PM3 overflow edge elevations	m	0.70
b_1, b_2, b_3 – PM1, PM3, PM2 overflow edge lengths	m	4.00
fk_1, fk_2, fk_3 – KL1, KL2, KL3 reflux valve surface areas	m^2	4.00
to – opening periods of OC1, OC2, OC3 orifices	s	120
fc – surface area of OC1, OC2, OC3 orifices	m^2	4.00

5. RESULTS

The analysis of the characteristics of rains observed in the catchment area between 2007–2008 revealed that maximum instantaneous rainfall intensities ranged from 0.6 to 0.8 mm/min, which corresponded to rains with 100% probability of occurrence. In the paper, the results of the simulation research are presented for two selected rainfalls with similar intensities but different duration periods and different intensity variations with time. The research work was also performed for a theoretical function representing the profile of wastewater inflow to the reservoir. In figures 4, 5 and 6, the results of the simulation research are presented showing the evolution of wastewater levels in individual chambers of the reservoir, and consequently its required capacity for the rainfalls of May 13, 2008 and August 18, 2007 selected for the analysis as well as for the theoretical block rainfall.
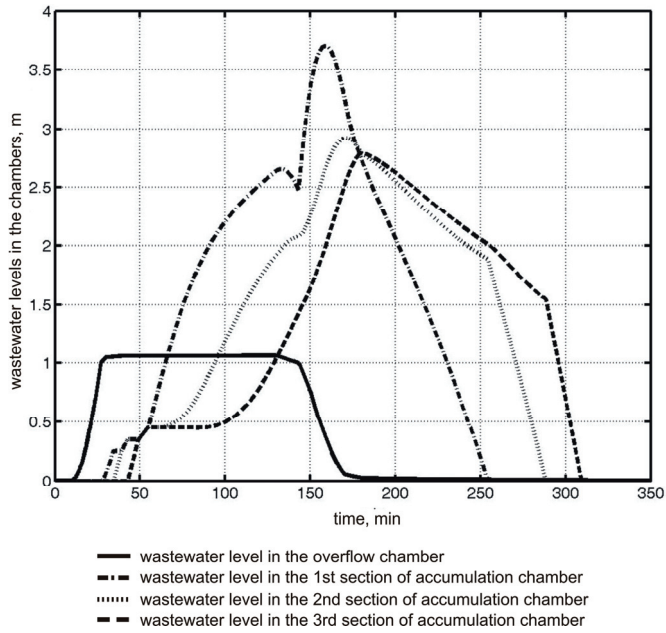
Fig. 4. Wastewater levels in the *KALIPSO* retention reservoir chambers
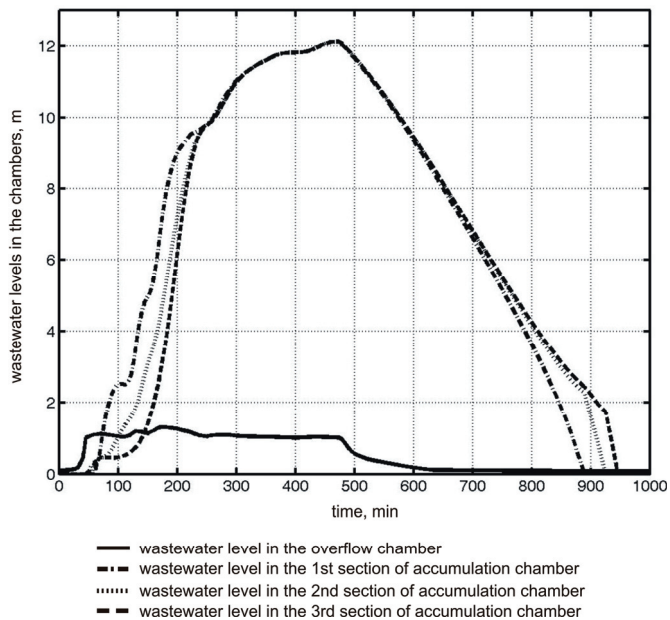for the rainfall of May 13, 2008

Fig. 5. Wastewater levels in the *KALIPSO* retention reservoir chambers
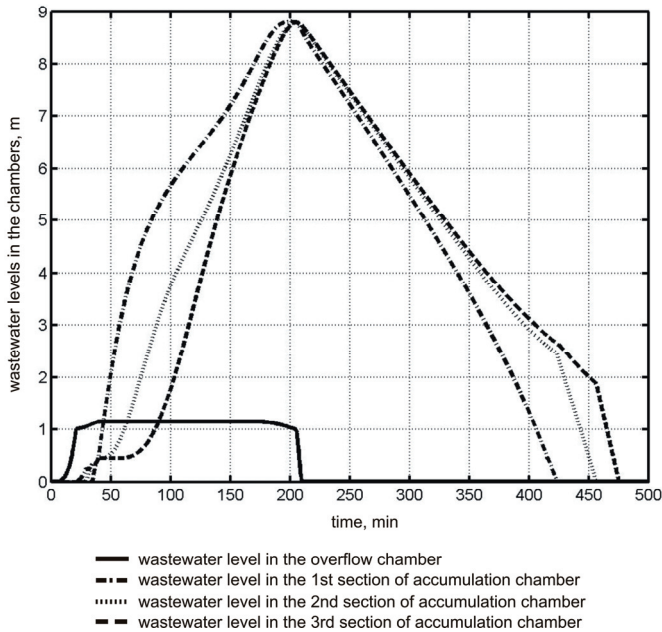for the rainfall of August 18, 2007

Fig. 6. Wastewater levels in the *KALIPSO* retention reservoir chambers
for the theoretical block rainfall

The results of retention reservoir simulation studies for wastewater flow hydrographs obtained on the grounds of the catchment hydrodynamical model show a very significant effect of rainfall duration period on the profile of wastewater level in reservoir chambers. Comparing the obtained maximum wastewater volumes for flow hydrographs of May 13, 2008 and August 18, 2007 and the theoretical hydrograph, one may claim that the required retention capacity value obtained for the latter is insufficient. That is a surprising result all the more because rainfall intensities corresponded to rains with 100% probability, while for the theoretical hydrograph calculations, rainfall with 20% probability was adopted. The result obtained for the theoretical hydrograph may be to some extent affected by the fact that the calculations of the maximum flow rate were carried out by means of disputable yet commonly used Błaszczyk's formula.

In [21], KOTOWSKI presents formulas recommended for use in the calculations of design storm precipitation intensity for the purposes related to retention reservoir design. However, their applicability does not cover the areas in which the investment is located. For that reason, on the ground of KOTOWSKI's studies [21], [22], the adoption of an additional safety factor is hereby proposed to increase the required capacity of the *Czuwaj* retention reservoir by an additional 60%. In that case, the required reservoir capacity would be about 21,000 m^3.

6. CONCLUSIONS

The paper presents basic issues concerning dynamical modelling of innovative *KALIPSO*-type retention reservoir solution and the results of simulation studies for the case study concerning its application to the municipal catchment of the town of Prze-myśl. The research work on the existing sewerage system was aimed at demonstration of capabilities of the program and its usefulness for calculations of the required reservoir capacity and dimensioning of devices and facilities making up its equipment.

The reservoir simulation model allows us to carry out studies based on theoretical wastewater flow function profiles, hydrographs obtained on the grounds of catchment hydrodynamical models and/or actual measurements carried out in sewerage systems.

The results of simulation studies obtained for the case considered above revealed the weaknesses of the commonly used Błaszczyk's formula for design storm intensity calculations when used for the purposes related to sewerage system dimensioning. This confirms the results of analyses concerning rainfall intensity calculation models used for retention reservoir dimensioning studies obtained by Kotowski.

To sum up, it should be emphasized that further development trends in retention reservoir dimensioning methods will be dominated by software tools allowing for the simulation of the wastewater retention process and the choice of design parameters on the grounds of actual measurements and hydrodynamical models of catchments and sewerage systems.

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