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SIZING AND MODELING OF THE SEWAGE SYSTEM IN THE CITY OF WROCLAW

WYMIAROWANIE I MODELOWANIE KANALIZACJI DESZCZOWEJ WE WROCŁAWIU

Abstract: The paper presents verification of selected flow time methods in terms of usability for sewage systems design and sizing on the example of a model municipal flat drainage area of the area of 2 km². Namely, the sewage system network was sized using three methods, that is, MGN with Błaszczyk's formula, MGN with the precipitation model for WROCLAW and MWO with the precipitation model for Wrocław, and then, the network functioning was verified for damming up on the area surface and flooding from drains using the hydrodynamic model SWMM 5.0. The precipitation model of Euler type II was used as the drainage area load in the conditions of Wrocław, for which the probabilistic model of the maximum precipitation was developed. It was shown that the safe flow time method for sewage system sizing is MWO using the criterion for the lack of damming up for the area and flooding from drains.

Keywords: storm water management model, sewer flooding, rain model

Introduction

The sizing of sewage systems or combined sewage systems in Poland presents difficulties resulting from the lack of a reliable precipitation model. The most frequently used model of Błaszczyk from 1954 lowers calculation results for rainfall intensities by 40%, which has been shown on the example of precipitation measured at the IMGW meteorological station in Wrocław in the period of 1960-2009 [1, 2]. This affects the design of drainage areas in Poland according to the latest European standard of BS-EN 752:2008 related to permissible frequencies of sewage system flooding. The safe design of sewage systems aims at ensuring a proper standard of an area drainage, which is defined as a sewage system adaptation to take forecast precipitation water flows with a frequency equal to the permissible - socially acceptable frequencies of flooding occurrences (Table 1).

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Table 1

The recommended design frequencies of rainfall and permissible flooding occurrence frequencies according to BS-EN 752

Design rainfall frequency [1 per C years]	The area drainage standard category	Flooding occurrence frequency [1 per C years]
1 per 1	I. Out of town areas (rural)	1 per 10
1 per 2	II. Residential areas	1 per 20
1 per 5	III. City centers, service and industry area	1 per 30
1 per 10	IV. Underground transportation facilities, underpasses, etc.	1 per 50

Hydrodynamic models for the simulation of sewage system functioning make it possible to account for real precipitation run-offs that are variable both in time and space, as well as variable and transient sewage flow in channels [3-5]. The application of hydrodynamic run-off models for the verification of sewage system functioning requires a previous sizing of drainage systems by the so-called flow time methods [2, 6-9]. In such methods, the sewage system or combined sewage system sizing is based on a number of simplifying assumptions (constant block precipitation in a drainage basin, uniform steady flow in channels). In consequence, the *method of critical intensities* (MGN) used in Poland according to Blaszczyk's formula underrates flooding calculation values in comparison with the *method of delay coefficient* (MWO) used in Germany [10, 11]. Larger sewage systems sized using these methods (in particular, with drainage basin areas of $F > 2 \text{ km}^2$) are now recommended to be verified for hydraulic flow capacity based on calibrated simulation models to meet the requirements of BS-EN 752.

In this paper, the functioning of a sewage system sized using three flow time methods, that is, MGN with Blaszczyk's formula, MGN with precipitation formula for Wrocław and MWO with precipitation formula for Wrocław, has been simulated for a model municipal drainage basin of the area of 2 km^2 on a flat surface. The model precipitation of Euler's type II in hydrological conditions of Wrocław (the land elevation of 120.0 m above sea level) was used as the drainage basin load. The program SWMM 5.0 was used for simulation. The safe method for sewage system sizing was indicated (MWO) with the criterion of the lack of flooding from channels.

Initial assumptions and investigation methods

In the hydrodynamic modeling of sewage systems, the size of integrated partial areas of a drainage basin in urbanized areas should be limited to a few hectares [12]. Thus, partial drainage basins of the areas of 2.25 ha ($150 \times 150 \text{ m}$) were assumed and the model basin of the dimensions of $750 \times 2700 \text{ m}$ and the total area of $F = 202.5 \text{ ha}$ was proposed. The drainage basin consists of 90 modules - integrated partial drainage basins. The schematic diagram of the model drainage basin has been shown in Figure 1. It was assumed that the drainage basin for a residential area is located on a flat surface in Wrocław. The flat drainage basin is a sharp criterion for the assessment of sewage system functioning, since all calculation nodes are potential critical points of the network at the same time. Assuming the mean weighted coefficient of surface run-off from the drainage basin $\psi = 0.25$, its reduced area amounts to $F_{\text{zr}} = 50.625 \text{ ha}$. This corresponds to the assumption that 80% of the partial

drainage basin area is permeable in a different degree (of the coefficient of $\psi_p = 0.075$), while the remaining 20% are impermeable surfaces (with $\psi_n = 0.95$). It has also been assumed that side channels being designed, to the number of 36 (Fig. 1), will have the length of 300 m each (2 design sections, 150 m each). The interceptor will have the total length of 2700 m (18 design sections, each of 150 m in length). The channel wall coarseness is assumed at the level of $n_k = 0.013 \text{ s/m}^{1/3}$.

Fig. 1. The schematic diagram of the model drainage basin of the area of $F = 202.5 \text{ ha}$

The program SWMM 5.0 (*Storm Water Management Model*) has been selected for the hydrological simulation of sewage systems, developed by the American Environmental Protection Agency (www.epa.gov). The application consists of three main modules. The RAIN module is used to input and store rainfall related data, the RUNOFF module is designed for hydrological modeling of a drainage basin in terms of hydrology and quality, while the EXTRAN module can be deployed for hydrological modeling of sewage networks using the system of Saint Venant's differential equations [13].

In SWMM 5.0, a partial drainage basins is represented with a rectangle of a set area and width (hydraulic). To calculate the run-off, the drainage basin is basically divided into two surface types: permeable, where infiltration occurs, and impermeable. The run-off is calculated separately for each surface to be summed later on. The stream of rainfall sewage flowing into a network design node is calculated using the model of a non-linear reservoir:

$$Q_m = W \frac{(d - d_p)^{5/3}}{n} i^{1/2} \quad (1)$$

where: Q_m - model run-off from a drainage basin [m^3/s]; W - hydrological width of a drainage basin [m]; d - precipitation amount [m]; d_p - surface retention height [m]; n - Manning's surface roughness coefficient [$\text{s/m}^{1/3}$]; i - mean drainage basin area slope [-].

The parameters W , i , n of an integrated partial drainage basin represent retention properties of a drainage basin. For symmetrical drainage basins in relation to a rainwater sewage channel, it is assumed that the hydraulic width is twice as large as the channel length. This is equivalent to the assumption of a design run-off length from a drainage basin, equal to the distance from the drainage basin boundary to the discharge channel, which, in turn, corresponds to a half of the geometrical drainage basin width (the drainage basin dimension in the direction perpendicular to the channel). Since integrated partial areas (modules) of the model drainage basin are $150 \times 150 \text{ m}$ squares and the rainfall sewage

discharge channel is located in the middle of the drainage basin, the hydraulic width $W = 300$ m of such a drainage basin has been assumed in SWMM [14]. The graphic interpretation of the hydraulic width determination method of the integrated partial drainage basin has been shown in the Figure 2.

Fig. 2. The graphical representation of the hydraulic width determination method for a partial drainage basin

The following has been assumed for hydrological simulations: a substitute surface slope value $i = 1\%$ a substitute roughness coefficient (for Manning's formula) for sealed drainage basin surfaces $n = 0.02 \text{ s/m}^{1/3}$ [15-21].

Sizing of example rainwater sewage systems

The network of rainwater sewage system for the proposed model drainage basin has been sized using three flow time methods. When designing channel pits, 1.4 m has been assumed as a minimum soil covering for side channels and 1.9 m for an interceptor. Channels of a circular cross-section, arranged with a minimum bottom slope selected from the formula $1/D$ have been used. The channel connection in design nodes (connection chambers) has been designed according to the sewage table leveling rule. When leveling the sewage tables, the channels were connected by their bottoms, when the ordinate of the sewage discharge channel bottom was higher than the ordinate of the supply channel.

The hydrological and hydraulic computations have been conducted for three network sizing variants (three flow time methods):

- MGN with Blaszczyk's precipitation model ($H = 590$ mm for Wrocław),
- MGN with the maximum precipitation model for Wrocław,
- MWO with the maximum precipitation model for Wrocław.

In the MGN method used in Poland, it is assumed that the design volume flow of rainwater sewage (Q) in the channel cross-section in question occurs with a certain delay in relation to the precipitation beginning moment (after a dry period), by the time necessary for: terrain concentration (t_k), channel retention (t_r) and flow in the channel (t_p) - from the beginning to the design cross-section. Hence, the rainwater run-off time from the drainage basin is assumed to be equal to the dependable rainfall duration: $t_{dm} = t_k + t_r + t_p$. In any channel cross-section, the design volume flow Q (in $[\text{dm}^3/\text{s}]$) is written by the following formula:

$$Q = q(t_{dm}) \cdot F_{zr} = \frac{6.631 \sqrt[3]{H^2 C}}{t_{dm}^{2/3}} F_{zr} \quad (2)$$

where: $q(t_{dm})$ - a rainfall intensity for a dependable duration according to Blaszczyk's formula [$\text{dm}^3/(\text{s} \cdot \text{ha})$]; H - a mean annual precipitation amount (from the minimum of 30 years) [mm]; C - a design rainfall frequency [years]; F_{zr} - a reduced rainfall drainage basin area [ha].

In the MWO method used in Germany, the time of sewage flow (t_p) in a channel is assumed as a dependable rainfall duration (t_d). Thus, terrain and channel retention times are omitted mainly due to the network operational safety at precipitation occurrence frequencies C more seldom than the design frequencies. The rainwater run-offs determined in this manner are higher in comparison with values computed according to MGN [22]. The volume flow Q [dm^3/s] in MWO is calculated using the following formula:

$$Q = q(t_d) \cdot F_{zr} \quad (3)$$

where $q(t_d)$ - a rainfall intensity for a duration equal to a flow time ($t_d = t_p$) [$\text{dm}^3/(\text{s} \cdot \text{ha})$].

The probabilistic model of maximum precipitations in the conditions of Wrocław has been formulated for the range of $t_d \in [5; 4320]$ minutes and occurrence probability $p = 1/C \in [1; 0,01]$ in the following form [23]:

$$h(t_d) = -4.58 + 7.41t_d^{0.242} + (97.11t_d^{0.0222} - 98.68)(-\ln p)^{0.809} \quad (4)$$

where $h(t_d)$ - the maximum rainfall amount (for the duration t_d) [mm].

The breakdown of initial assumptions for 3 different design variants of the rainwater sewage network has been shown in the Table 2.

Table 2

The initial assumptions for sizing example networks of a rainwater sewage system

Design variant	Design rainfall frequency C [years]		Terrain concentration time t_k [min]		Channel retention time t_r [min]	Minimum dependable rainfall duration t_d [min]
	side channels	interceptor	side channels	interceptor		
I.	1	2	10	5	$0.2 t_p$	10
II.	1	2	10	5	$0.2 t_p$	10
III.	2	2	0	0	0	15

Hydraulic computations of the rainwater sewage network with the selection of channel diameters and gradients for the assumed design assumptions has been shown (Table 2). In the I. design variant, side channels consist of 2 sections with diameters K0.30 and K0.40 m and are buried to the depth from 1.70 to 2.62 m below ground level. The interceptor consists of 18 sections from K0.80 to K1.60 m, buried to the depth from 2.80 to 5.99 m below ground level. The design flow time amounted to 45.6 minutes, while the rainwater run-off stream $Q_{(I)} = 1.948 \text{ m}^3/\text{s}$. The channel volume of the sized network is 4849 m^3 , hence the volume rate per one hectare of the drainage basin area is: $4849/202.5 = 23.9 \text{ m}^3/\text{ha}$. In the II. variant, side channels have diameters of K0.40 and K0.50 m, while the interceptor from K0.80 to K2.0 m. The burial depths range from 1.80 to 5.91 m below ground level. The flow time amounted to 43.85 minutes and the design sewage stream to $Q_{(II)} = 3.049 \text{ m}^3/\text{s}$. The channel volume amounts to 7234 m^3 , hence the volume flow rate is $35.7 \text{ m}^3/\text{ha}$. In the III. variant, side channels have diameters of K0.40 and K0.60 m, while the interceptor from K0.80 to K2.20 m. The burial depths range from 1.80 to 5.33 m below ground level. The flow time amounted to 43.33 minutes and the sewage stream to

$Q_{(III)} = 3.700 \text{ m}^3/\text{s}$. The network volume is 9825 m^3 , hence the rate is $48.5 \text{ m}^3/\text{ha}$. The cumulative breakdown of sizing results has been given in the Table 3.

Table 3

The sizing results of example sewage networks [14]

Design variant	Run-off stream $Q [\text{m}^3/\text{s}]$	Design flow time [min]	Channels and interceptor dimensions [m]	Channels and interceptor burial depths [m bgl]	Network volume $V_K [\text{m}^3]$	Network volume rate $[\text{m}^3/\text{ha}]$
I.	1.948	45.60	K0.3-K1.6	1.70-5.99	4849	23.9
II.	3.049	43.85	K0.4-K2.0	1.80-5.91	7234	35.7
III.	3.700	43.33	K0.4-K2.2	1.80-5.33	9825	48.5

Taking the design volume flow of rainwater sewage run-off from the I. variant of the sewage network (Table 3): $Q_{(I)} = 1.948 \text{ m}^3/\text{s}$ as the relative basis for comparisons (100%), then, the flow in the III. variant: $Q_{(III)} = 3.700 \text{ m}^3/\text{s}$ is higher by as much as 90%. In the case of the rainwater sewage system located on a flat land and sized in the III. variant, despite larger side channels and interceptor diameters (up to K2.2 m) compared with the I. variant (up to K1.6 m), for example, the construction costs of such a network may turn out to be lower (or comparable) due to the shallowing of the interceptor in the III. variant.

Creation the model precipitation of Euler's type II

The concept of the model precipitation is to yield a typical precipitation distribution of an intensity variable in time in a manner close to reality. Model precipitations are then synthetic rainfall hyetographs created on the basis of local precipitation amount curves (DDF) or precipitation intensity curves (IDF). An example of model precipitations is the Euler type II model, recommended for sewage modeling in Germany [10, 24]. The model is based on the observation that the highest momentary rainfall intensity occurs at the end of the third part of its duration. According to this rule and with the 5-minute time step of the discretization, the precipitation of the highest unit intensity after the period of 0.3 of the precipitation duration is determined. The subsequent highest precipitations in time intervals are determined in the decreasing order - to the left of the maximum precipitation, until the precipitation beginning time is reached. Subsequent precipitations are compiled in the non-increasing order - to the right of the highest precipitation, until the precipitation ending time is reached. The Euler model precipitation is recognized as corresponding to real measured series of annual storm rainfalls, which are typically not easily accessible for a designer.

Table 4

The damming up frequencies for verifying computations of sewage systems being designed recommended according to ATV A-118

Land development type	Damming up frequencies [1 per C years]
I. Out of town areas (rural)	2
II. Residential areas	3
III. City centers, service and industry area	Less than 5
IV. Underground transportation facilities, underpasses, etc.	Less than 10

In order to verify the occurrence of damming up in channels, in example designer networks, the model drainage basin should be loaded with rainfall of the occurrence frequency of $C = 3$ years (Table 4) and with duration twice as long as the network flow time according to the recommendations ATV-A118. Since the mean flow time in designed model drainage basins is of the order of 45 min, the model precipitation with the duration of $t = 90$ min (Fig. 3) has been developed based on the formula for the maximum precipitation amount in Wrocław.

Fig. 3. The model precipitation creation of Euler type II for Wrocław ($C = 3$ years and $t = 90$ min): a) the rainfall intensity in 5-minute intervals; b) the model precipitation of Euler type II

The model precipitation developed for the conditions of Wrocław for $t = 90$ min and $C = 3$ years ($p = 0.33$) is characterized by the maximum intensity of 101.71 mm/h, occurring between the 25th and 30th minute. The mean precipitation intensity amounts to 17.83 mm/h, which corresponds to 26.75 mm of the precipitation height.

Modeling the operation of example sewage networks

In order to verify the hydraulic flow capacity - sized in the 3 variants - of the rainfall sewage system, the drainage basin has been loaded with the model precipitation of Euler type II for the conditions of Wrocław. From the total precipitation amount (26.75 m), its forth part ($\psi = 0.25$) has been transformed into the surface flow and reached the sewage system. As a result of hydrodynamic simulations carried out, the information on volume flows and fills in specific sewage sections in the time of the model precipitation duration was obtained. The interceptor profile in a selected precipitation duration time (the 31st minute) for the sewage system sized in the I. variant has been shown in Figure 4.

The rainwater sewage system designed in this variant does not have appropriate hydraulic flow capacity to discharge modeled rainwater run-offs without damming up to the ground level. As it has been shown in Figure 4, except for the last section, practically the entire interceptor operates under pressure during the given modeling task. The damming ups of a few meters - including to the ground surface and flooding - occur in most interceptor design nodes. In case of side channels, we also have numerous damming ups to the ground level. In the beginning side channels, the pressure reaches the ground level along the entire channel length. In case of further side channels, where the interceptor, to which they are connected, is located deeper, the damming ups to the ground level occur only on the beginning sections of the side channels.

Fig. 4. The interceptor profile in the 31st minute of the precipitation duration modeled in the I. variant

The total sewage volume, which was not contained or flowed out of the network, amounts to 1291 m³ (Table 5) during the model precipitations. The most of the volume consists of rainwater sewage that flowed out from the beginning network sections - in the design nodes, where the interceptor burial depth is the lowest. The highest noted flooding occurred in nodes J6 and J10, and amounted to 42 m³. The flooding was observed in the total of 71 design nodes, that is 71 integrated partial drainage basins.

Table 5

The breakdown of flooding volumes from the sewage system in the I. variant

Node	Flooding volume [10 ³ m ³]	Node	Flooding volume [10 ³ m ³]	Node	Flooding volume [10 ³ m ³]	Node	Flooding volume [10 ³ m ³]	Node	Flooding volume [10 ³ m ³]
J1	0.041	J19	0.017	J37	0.001	J55	0.022	J73	0.000
J2	0.032	J20	0.039	J38	0.004	J56	0.020	J74	0.000
J3	0.009	J21	0.037	J39	0.001	J57	0.000	J75	0.017
J4	0.032	J22	0.011	J40	0.030	J58	0.003	J76	0.017
J5	0.041	J23	0.002	J41	0.028	J59	0.000	J77	0.000
J6	0.042	J24	0.011	J42	0.001	J60	0.020	J78	0.000
J7	0.029	J25	0.037	J43	0.004	J61	0.018	J79	0.000
J8	0.002	J26	0.035	J44	0.001	J62	0.000	J80	0.017
J9	0.029	J27	0.008	J45	0.028	J63	0.001	J81	0.018
J10	0.042	J28	0.004	J46	0.025	J64	0.000	J82	0.000
J11	0.040	J29	0.008	J47	0.001	J65	0.018	J83	0.000
J12	0.020	J30	0.035	J48	0.006	J66	0.017	J84	0.000
J13	0.001	J31	0.033	J49	0.001	J67	0.000	J85	0.017
J14	0.020	J32	0.004	J50	0.025	J68	0.000	J86	0.017
J15	0.040	J33	0.006	J51	0.022	J69	0.000	J87	0.000
J16	0.039	J34	0.004	J52	0.001	J70	0.017	J88	0.000
J17	0.017	J35	0.033	J53	0.006	J71	0.017	J89	0.000
J18	0.002	J36	0.030	J54	0.001	J72	0.000	J90	0.017

The design nodes, in which a flooding occurred, have been shown schematically in Figure 5. Only in the case of 19 of 90 nodes, the damming ups did not reach the ground level in any time moment of the model precipitation duration.

Fig. 5. Locations of damming ups above the ground level in the I. variant

In the I. variant of the sized sewage system, the maximum model volume flow in the last interceptor section amounted to $Q_{m(I)} = 5.160 \text{ m}^3/\text{s}$. The sewage flow hydrograph on the interceptor section has been shown in Figure 6.

Fig. 6. The sewage flow hydrograph on the last interceptor section J88-out in the I. variant

The model precipitation lasted 90 minutes and the rainwater run-off and channel emptying time ended after approximately 4.7 hours from the moment of the model precipitation beginning (the volume flow in the interceptor drops then below $5 \text{ dm}^3/\text{s}$).

In order to verify the hydraulic flow capacity of the sewage system designed in the II. variant, the drainage basin has been loaded with the model precipitation similarly to the I. variant. The inceptor profile with fills in the selected time of the precipitation duration (the 33rd minute) has been shown in the Figure 7.

Fig. 7. The interceptor profile in the 33rd minute of the precipitation duration modeled in the II. variant

As it can be seen in Figure 7, the damming ups in the beginning interceptor sections - in nodes J23 and J28, in the presented time moment of the simulation, reach the ground level (flooding). The middle and partially ending interceptor sections operate under small pressure. In case of the beginning side channels, we also have damming ups to the ground level, however for the further side channels, where the interceptor is buried considerably deeper, the damming ups to the ground level do not occur. The total sewage volume, which flowed out from the sewage system during the model rainfall, amounts to 20 m^3 only. The highest simulated flooding occurred in nodes J1, J5 and J18 - 3 m^3 each. All in all, the flooding was observed in 12 nodes only.

The nodes, in which a flooding occurred, have been shown schematically in Figure 8. In the case of 78 of 90 integrated partial drainage basins, the damming ups did not reach the ground level in any time moment of the model precipitation duration.

Fig. 8. The locations of damming ups to the ground level in the II. variant

The maximum model volume flow reached $Q_{m(II)} = 6.112 \text{ m}^3/\text{s}$. Its variability in time on the last interceptor section has been shown in Figure 9. The time of rainwater sewage run-off and channel emptying ended after approximately 5.1 hours from the precipitation beginning time.

Fig. 9. The hydrograph of the sewage flow on the last interceptor section J88-out in the II. variant

In order to verify the hydraulic flow capacity of the sewage system designed in the III. variant, it has also been loaded with the model precipitation of Euler type II. The interceptor profile with fills in the selected time of the precipitation duration (the 34th minute) has been shown in Figure 10.

Fig. 10. The interceptor profile in the 34th minute of the precipitation duration modeled in the III. variant

As it follows from Figure 10, practically the entire interceptor operates with a free sewage table during the set model rainfall, except for the interceptor section J23-J28 operating with a several-centimeter damming up above the channel vaulting. Damming up to the ground level are also absent in the case of all side channels - the lack of flooding from channels. Only damming up of a several dozen centimeters in relation to vaults occur on the first two sections, the two side channels.

Fig. 11. The hydrograph of the sewage flow on the last interceptor section J88-out in the III. variant

The maximum model volume flow (in the III. variant) on the last interceptor section reached $Q_{m(III)} = 6.947 \text{ m}^3/\text{s}$ (Fig. 11). The simulated rainwater run-off and channel emptying time in the III. variant ended after approximately 5.3 hours from the model precipitation beginning. The cumulative breakdown of the operation analysis of the model sewage systems (sized in the three variants - for $i = 1.0\%$ and $n = 0.02 \text{ s/m}^{1/3}$) has been shown in Table 6.

The maximum values of the simulated volume flows rise with the increase in the network flowability, that is, the channel diameters and network volume (Table 6): from $Q_{m(I)} = 5.160 \text{ m}^3/\text{s}$ - in the I. variant (with numerous flooding) to $Q_{m(III)} = 6.947 \text{ m}^3/\text{s}$ (without flooding). The high volume of the storm channels in the III. variant of the sewage system sizing $V_{K(III)} = 9825 \text{ m}^3$ is then a considerable buffer („retention reservoir”) in

relation to the water run-off volume of the model precipitation from the drainage basin:
 $V_{op} = h_{op} F_{zr} = 0.02675 \cdot 506250 = 13542 \text{ m}^3$.

Table 6

The results of operation modeling of example sewage systems for $i = 1.0\%$ and $n = 0.02 \text{ s/m}^{1/3}$

Design variant	Model flow $Q_m [\text{m}^3/\text{s}]$	Number of flooding	Volume of flooding $[\text{m}^3]$	Run-off and network emptying time [hours]
I	5.160	71	1291	4.7
II	6.112	12	20	5.1
III	6.947	0	0	5.3

For authentication of obtained results of analysis, assessment of influence of parameter values i and n for modeling of the sewage system was done and simulation of operation of dimensioned sewage systems for $i = 0.5\%$ and $n = 0.01 \text{ s/m}^{1/3}$ was repeated. The cumulative breakdown of modeling results has been given in Tables 7 and 8.

Table 7

The results of operation modeling of example sewage systems for $i = 0.5\%$ and $n = 0.02 \text{ s/m}^{1/3}$

Design variant	Model flow $Q_m [\text{m}^3/\text{s}]$	Number of flooding	Volume of flooding $[\text{m}^3]$	Run-off and network emptying time [hours]
I	5.080	68	1194	4.8
II	6.008	6	9	5.3
III	6.824	0	0	5.5

Table 8

The results of operation modeling of example sewage systems for $i = 0.5\%$ and $n = 0.01 \text{ s/m}^{1/3}$

Design variant	Model flow $Q_m [\text{m}^3/\text{s}]$	Number of flooding	Volume of flooding $[\text{m}^3]$	Run-off and network emptying time [hours]
I	5.211	71	1349	4.5
II	6.223	15	48	4.9
III	6.958	0	0	5.1

The third rainwater sewage system sizing variant - according to MWO with the maximum precipitation model for Wrocław - ensures the safe network sizing results for the model drainage basin in the conditions of Wrocław (Tables 6-8).

Summary and conclusions

The investigations described in this paper have been carried out to verify the selected flow time methods (MGN with Blaszczyk's formula, MGN with the model precipitation for Wrocław and MWO with the model precipitation for Wrocław) for usability in the safe designing of sewage systems. The verification was conducted on the example of the model drainage basin located on the flat land in the hydrological conditions of Wrocław. To simulate the functioning of sewage networks sized in this manner, the application SWMM 5.0 was deployed, in which the precipitation model of Euler type II was assumed as the precipitation load, while damming up and flooding were assumed as the criterion of the correct network operation.

The analyses carried out have unequivocally shown that the safe sewage system sizing method is the MWO with the maximum precipitation model for Wrocław due to the lack of damming up to the ground level and flooding from the sewage system. The assumed model drainage basin on the flat ground was the sharp criterion for the assessment of the sewage operation, since all design nodes were critical network points.

Blaszczyk's formula and MGN in general considerably underrates the storm sewage volume flow necessary for sewage system sizing, causing in consequence numerous damming up to the ground level and flooding (Tables 6-8). The number of damming ups and flooding volumes from the sewage system sized in the I. variant amount to: 71 and 1291 m³, respectively. The attempt to substitute Blaszczyk's formula in MGN with the new maximum precipitation formula for Wrocław (in the II. variant) has also failed to give satisfying results - the number of damming ups and flooding volume have only decreased (12 and 20 m³, respectively). The third rainwater sewage system sizing variant - according to MWO with the maximum precipitation model for Wrocław - ensures the safe network sizing results for the model drainage basin in the conditions of Wrocław - the lack of damming ups to the ground level and flooding (Tables 6-8).

When the model precipitation has decreased, the rainwater run-off time (terrain retention), and mainly channel emptying time (channel retention) keeps lasting for a few hours from the moment the precipitation ends - the longer the time, the bigger the channel diameters and lower required bottom slopes. In this time, the sewage system (sized for $C = 2$ years at $t_d = t_p = 45$ minutes) is not fully ready to take the rainfall of similar intensity and duration as the model precipitation ($C = 3$ years and $t_d = 2t_p = 90$ minutes), since channels are still partially filled. Thus, flooding from channels is inevitable, even when using correct flow time methods for the sizing of drainage networks, and then for the verification of their operation in hydrodynamic modeling (SWMM) due to the stochastic (random) nature of rainfall. Thus, this breaks down to the limiting of damming up and flooding occurrence probability, which can only be obtained in safely sized sewage systems.

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WYMIAROWANIE I MODELOWANIE KANALIZACJI DESZCZOWEJ WE WROCŁAWIU

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Abstrakt: W pracy symulowano działanie kanalizacji deszczowej zwymiarowanej trzema metodami czasu przepływu, tj.: MGN z wzorem Błaszczyka, MGN z wzorem opadów dla Wrocławia i MWO z wzorem opadów dla Wrocławia, dla modelowej zlewni miejskiej o powierzchni 2 km² w terenie płaskim. Do symulacji hydrodynamicznych systemów kanalizacyjnych wybrano program SWMM 5.0 (Storm Water Management Model), opracowany przez Amerykańską Agencję Ochrony Środowiska (US EPA). Jako obciążenie zlewni zastosowano opad modelowy Eulera typ II w warunkach hydrologicznych miasta Wrocławia (rzędna terenu 120,0 m nad poziom morza). Wskazano na bezpieczną metodę wymiarowania systemów kanalizacyjnych (MWO) przy kryterium braku wylewów z kanałów.

Słowa kluczowe: model zarządzania wodami opadowymi, wylania z kanalizacji, opady modelowe