ORIGINAL ARTICLE

Potential of storm water storage tank outflow construction in the prevention of sewerage overload

Wojciech Dąbrowski[1] · Michał Nowak[1]

Abstract

The impact of a storm water storage tank outflow construction on its required volume is discussed. A dimensioning of the tank by a rational method applied for small sewerage systems is presented. For large systems, subroutines should be developed to take into consideration the construction details of complex storage tanks in the software enabling real time modelling of sewerage systems. Such subroutines can be tested under simple conditions using rational methods of sewerage modelling, including the approach described in the paper. In the example discussed here for two different positions of the sewer delivering to and collecting stormwater out of the tank both the tank required volume and the outflow as a function of time visibly depended on the sewer position. This was proofed for a rectangular chamber storage tank of the same bottom surface area. However, if the bottom surface area and the storage tank height were calculated in such a way to enable the same maximum value of an outflow from the tank for two different positions of the sewer both the tank volume and the outflow as a function of time were proved to be very similar. Concluding the tank volume depends visibly on the construction of outflow, but the height of the tank can be adjusted in such a way to keep the same maximum outflow for different details of the outflow construction. After this adjustment, the volume of the tank was proved to be almost independent of the construction of outflow from the tank.

Keywords Storm water storage tank · Storm water · SWMM · Tank outflow construction · Computing of storage tanks volume

Introduction

Retention tanks are commonly used in combined sewerage systems to control the frequency and the total load of pollution discharged to the nearby rivers. In some countries the limitations set on these discharges refer to the frequency of discharge, the COD load discharged annually from a unit impermeable surface of catchments, on the initial dilution coefficient and on the surface water quality (Butturi et al. 2020). According to the standard EN 752:2008 real time modelling is required for describing the impact of large, combined sewerage overflows (CSO) on discharge to the receiving surface waters. In some countries, especially those with polders, accumulation of runoff occurring until a specified depth of precipitation is required (Butturi et al. 2020).

The most reasonable position of a retention tank constructed for controlling pollution discharges through CSO is between the sewer and the final overflow to the surface water. The first overflow is located at the sewer, to deliver wastewater to the retention tank only when the sewer is flowing full. In this way, the accumulation of combined wastewater is avoided if the sewer has enough capacity to transport the whole amount of wastewater towards the wastewater treatment plant (WWTP). When the sewer is overloaded the excess of wastewater flows through the first overflow filling the tank. When the tank is full the CSO starts to discharge to the surface water. Sedimentation occurs in the storage tank resulting in decreasing of suspended solids together with heavy metals, microorganisms, COD, BOD and other pollutant parameters (Ashley Dąbrowski 1995). In separate sewerage systems stormwater green retention is an alternative to grey retention giving some more benefits to the local society (Dąbrowski, Zielina, McGarity 2021). However, not always it is possible to grow enough green infrastructure in

✉ Wojciech Dąbrowski
  wdabrow@pk.edu.pl

1 Environmental and Power Engineering Department, Cracow University of Technology, 31-135 Kraków, Poland

town centers and sometimes grey and green mix of infrastructure is an efficient solution.

The subject of this paper refers to storage tanks constructed for a different purpose and serving storm water drainage systems. Sometimes a sewer is too small to transport the amount of storm water delivered from the upper sewers. The choice is between rebuilding the sewer or constructing a storage tank at its top. The decision should be economical. Now the purpose of retention is totally different than of CSO's but the philosophy behind the construction details design is quite similar to that described previously. At the beginning of the storm, the flow should be directed totally towards the WWTP and the accumulation in the storage tank starts as soon as the capacity of the sewer is not high enough. This rule of operation has a visible impact on the required storage tank volume.

The storage tank is a part of a storm water disposal system, so the hydraulics calculations for large scale should be based on real time modelling principles and include the cooperation of the tank with the whole system. However, storage tanks are of different constructions impacting the required volume, which usually is not to be modelled by commercially available software. This software is unable to model most complex tanks constructions, such as (Dziopak, Niemczynowicz 1999), (Słyś 2010), (Starzec et al. 2018) single chamber tank Canalis, two-chamber tank Contract, multi-chamber tank Complement, Wiskarb, Complex, tabular detention tank Magnus, multi-chamber tank Commodus-S, three-chamber detention tank Mirus, Mirus-S, multi-chamber tank Parkus-S, Conses-S, Kalipso-type sewage tank, and others. Because of this, it is recommended to use programs of available source code enabling to develop own subroutines. US Environmental Protection Agency Storm Water Management Model (SWMM) is such an example. Although, the effectiveness of day to day modelling is the subject of several limitations when applied to storage tank modelling. Probabilistic models can be another example (Szeląg, Kiczko 2014; Szeląg, Bąk 2016). New subroutines should be tested in advance to apply them in professional programs. Such simple tests can be performed starting from one of the rational methods. Using such tests has been presented here for a simple rectangular tank to investigate the impact of the storage tank outflow construction on the required tank volume.

Purpose of the study

The purpose of the study is to present the importance of construction details, on the required storage tank volume, to prove that some subroutines describing the impact of these details should be developed and implemented in the software used for real time modelling design of sewerage systems with sophisticated storage tanks constructions. A very simple rectangular tank is used here as an example. Unfortunately,

companies developing commercially available numerical programs do not give access to the source code of their software, so although the EPA's SWMM (Storm Water Management Model) is not and will not be supported with some convenient commercial links to databases, such as GIS it should be chosen in such cases when more complex operated storage tanks are to be included in the modelling.

Rain precipitation model

An example of a simple sewerage system was chosen for the numerical tests. The Błaszczyk formula (1) for the rain precipitation intensity q [l/(s·ha)], as a function of: the average height of an annual rain precipitation H [mm/year], storm recurrence C [years], and the rainfall duration t_d [min], was adopted in the computing example.

$$q = \frac{6.631\sqrt[3]{H^2 C}}{t_d^{2/3}} \tag{1}$$

Equation (1) was published in 1954 from rainfall observations collected from one place in Warsaw.

The observations had covered the period from 1837 to 1925. Recently this formula is criticized for delivering too small values of q (Szeląg, Bąk 2016; Stirrup, Marchandt 2002) because of two reasons. First of all, in the time period of recording the data only 37 years of observations were available, but all 67 years were taken into account for the interpretation (Węglarczyk 2013). Moreover, climate changes tend to increase extreme weather events (Szeląg, Bąk 2016) although the rising trend of the annual rain height precipitation is not so well documented and, in some countries, such as Japan, this trend is the opposite. Concluding, the Błaszczyk formula underrepresents the real values of the rain precipitation intensity but it is still commonly used in Poland for the whole area of the country (Starzec et al. 2018). More recently equations describing the depth of precipitation in time have been developed by the Polish Institute of Meteorology and Water Management and individual equations have been developed for some large towns in Poland, for example Wroclaw (Kaźmierczak and Kotowski 2012). An atlas of rain events has been developed for the whole country. The Błaszczyk formula was used here in the computations only as an example of rain precipitation events.

Sewerage design

The inflow of storm water to the storage tank has been calculated from a rational method applicable for the catchments larger than 50 ha. First, in this method, the duration

of precipitation is assumed as being equal to the period of time taken by water to flow from the outermost point of the catchment to the end of any sewer "i." Then this duration of precipitation is extended by 20% compensation for the delay of flow through sewers resulting from the apparent fact that at the beginning of storm the sewers were empty. The index "i" takes values of 1 to "n", where n is the number of sewers along the flow path. The assumption that the largest flow results from the storm duration equal to the period time of flow through the sewerage system was tested by Kotowski and Kaźmierczak (2013), and Starzec M. et al. (2018) using SWMM software. In their computations, the largest flow rates through sewers computed by the SWMM software occurred for the storm duration up to 25% shorter, which suggests that the 20% of delay for filling in the sewers with storm water may be neglected in this rational method of sewers design. The flow through the sewer "i = n" located just above the storage tank was computed for several storm durations t_{di}, being first equal to and then up to several times longer than t_{dd}, because the duration of rain representative in this rational method for predicting the storage tank volume is unknown at the beginning of computations. Calculations were run based on assumptions shown in Table 1.

Table 1 Input data for computations

H =	600	[mm/year]	- Assumed normal precipitation height
C =	2	[years]	- Incidence of design rain
A =	592	[-]	- Błaszczyk method coefficient
$\sum F$ =	50	[ha]	- Total catchment area
ψ =	0.7	[-]	- Avarage surface runoff coefficient
μ =	0.6	[-]	- Suppression factor
h_{max} =	2.4	[m]	- Maximum depth of water in tank
t_p =	30	[min]	- Designed rainfall flow duration
S =	0.0029	[-]	- Outlflow pipe slope
d =	0,35	[m]	- Outflow diameter
A_{zb} =	1000	[m^2]	- Tank bottom slab area
n =	0.013	[s/m$^{1/3}$]	- Manning roughness coefficient

Construction details

In the calculations, the same circular sewer delivers storm water to the storage tank and drains it. Two different levels of the sewer bottom were considered. In the first case (Fig. 1a) the bottom of the sewer is located by its radius below the tank bottom, and in the second case (Fig. 1b) by the diameter below the tank bottom. It was assumed that the bottom of the storage tank has a slope along the direction of flow equal to the slope of the sewer.

Mathematical models

Before performing numerical tests using SWMM software, a simple mathematical model of a rectangular stormwater tank has been developed to verify the correctness of the SWMM software in these specific computations. This simplified model, developed in MS Excel, applies the rational method of computing the sewerage system above the tank and the Błaszczyk formula (1) for predicting the average intensity q of rain precipitations as a function of the rain durability. Both outflow locations presented in Fig. 1a and b were considered. The inflow to the tank was predicted from the rational method of computing the sewerage system and the outflow using the Manning's Eq. (2) first and then the equation describing an outflow through a small, submerged hole (3). The depth of stormwater accumulated in the tank was calculated from the mass balance Eq. (4) as a function of time t. The computations were done for the system of Eqs. (1), (2), (3), (4) for the data from Table 1 and then repeated for the same set of data using SWMM.
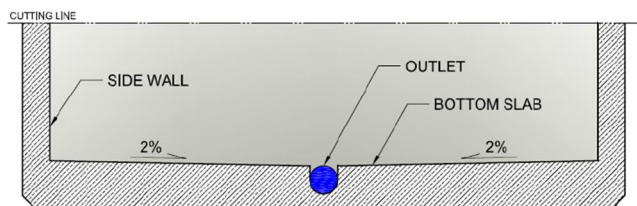
$$v = \frac{1}{n} \cdot R^{\frac{2}{3}} \cdot S^{\frac{1}{2}} \tag{2}$$

$$Q_{outf}(t) = \mu \cdot A_o \cdot \sqrt{2 \cdot g \cdot h(t)} \tag{3}$$

The friction to flow through sewers was calculated using the Manning formula (2), where v corresponds to
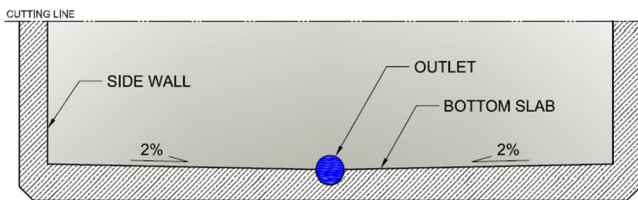
(a) First location of the sewer in comparison with the tank bottom (basic solution).

(b) Second location of the sewer in comparison with the tank bottom (alternative solution).

Fig. 1 a First location of the sewer in comparison with the tank bottom (basic solution). **b** Second location of the sewer in comparison with the tank bottom (alternative solution)

the cross-sectional average velocity of flow [m/s], n is the Gauckler-Manning roughness coefficient [s/m$^{1/3}$], R stands for the hydraulic radius [m], and S is the adopted hydraulic slope [-], A_o–surface area of the hole [m^2], h –head [m], g–acceleration due to gravity [m/s^2]. The storm water mass balance in the storage tank is described by Eq. (4), in which V is the volume of the tank occupied by stormwater [m^3], t–time [s], Q_{inf}, Q_{outf} – inflow to and outflow from the tank [m^3/s]. The rectangular shape of the tank was considered, so $V = A \cdot h$, where A is the surface area of the bottom, and h is the depth of storm water level in the tank.

$$\frac{dV}{dt} = Q_{inf} - Q_{outf} \tag{4}$$

The outflow from the tank up to the top of the sewer was calculated from the Manning Eq. (2) assuming uniform flow with the depth equal to h inside of the tank. For h higher than the outflowing sewer diameter d, the outflow was calculated from Eq. (3) describing the gravitational outflow from the tank through an opening of a diameter d [m], and the section surface A_o [m^2].

Results and discussion

In Fig. 2, an example of two single computations is presented. The inflow in time is represented by a trapeze, but the outflows slightly differ one from the other because the first represents the outflow for the position of the sewer illustrated in Fig. 1a and the second in Fig. 1b. The first is called the basic solution and the second the alternative solution. In both cases the rain precipitation is of the same duration equal to $3t_{dd}$ and the bottom surface area of the tank is unchanged. The volume V occupied by storm water in the first case is proportional to the surface area bounded by the line A, B, C, D and in the second case by the line A*, B, C, D *. For duration longer than t_D the outflow from the tank is larger than the inflow so the volume V decreases in time. Both the outflows from the storage tank and the maximum volume of storm water accumulated in the tank differ because of the different positions of the sewer in the basic solution (Fig. 1a) and the alternative solution (Fig. 1b).

The rain precipitation duration for which the volume V and the depth h are the highest should be used as a design storm. This period is unknown at the beginning of calculations and should be predicted by the trial and error method as illustrated in Fig. 3. In this figure, the storm of a duration $3 \cdot t_{dd}$ results in the highest required tank volume V, so this duration should be considered in the computations. This is illustrated in Fig. 3 for the basic position of the outflow from the tank (Fig. 1a).

The outflow from the tank depends on the depth h, so on both the volume V and the tank bottom surface A. Because of this assuming different values of A one receives in calculations different required volumes of the tank V.
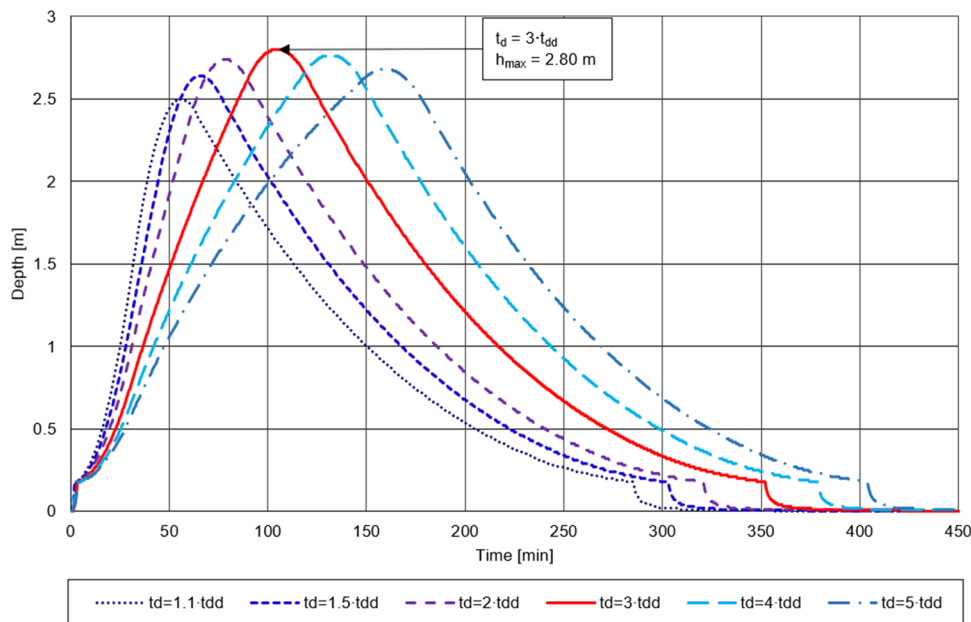
Simplified model versus SWMM software

Before using the SWMM software for testing the outflow vertical position impact (Figs. 1a and b) on the accumulation of stormwater in the tank, some tests were run for verifying if very close results are obtained using SWMM and the simplified mathematical model described in the

Fig. 2 An example of the inflow and two different outflow solutions as a function of time
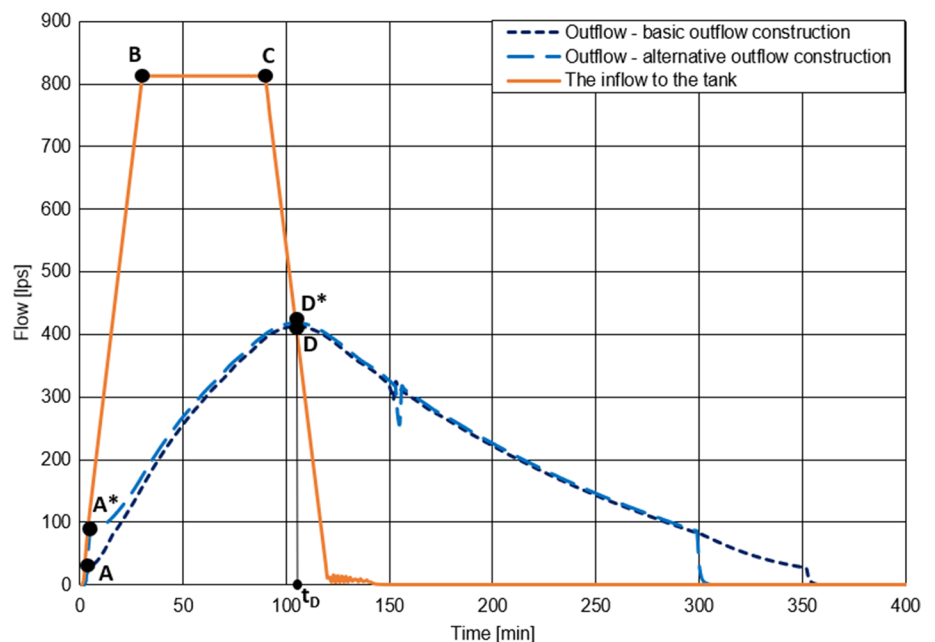
Fig. 3 The trial and error method applied for predicting the storm duration period, which gives the largest (required) depth of water in the tank h_{max}
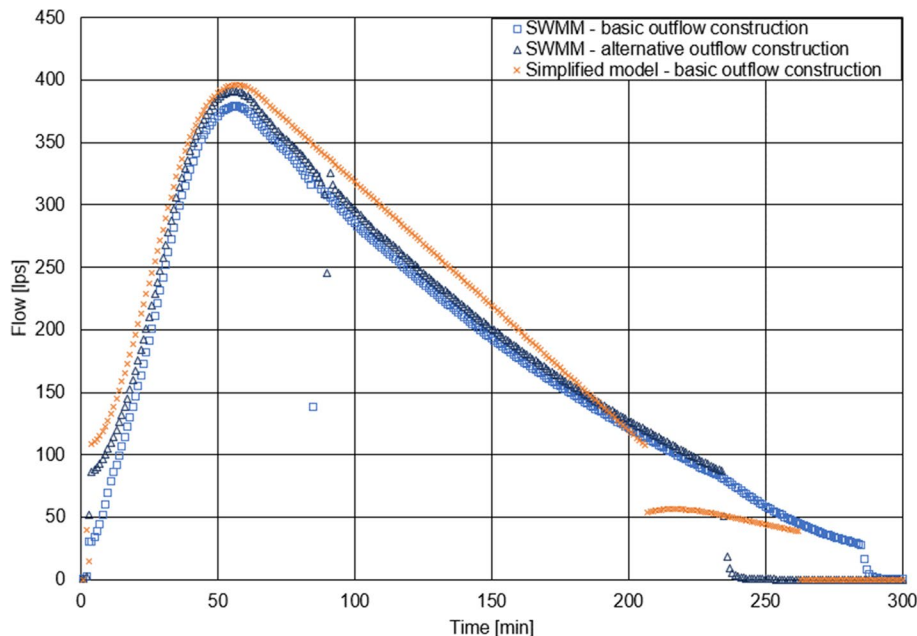

previous paragraph. Two examples of such tests are presented in Figs. 4 and 5.

The results obtained using SWMM and our own simplified mathematical model are similar, so the SWMM software was chosen for the computations as giving probably much better precision of computations for low depth h of storm water in the tank and allowing to make computations for variable rain intensity.

Application potential

Models like the one presented, which is based on SWMM, can be applied in all rational methods of small sewerage design as well as in real time modelling of sewerage systems equipped with storage tanks, which purpose is to reduce the maximum possible outflow. The maximum value of the outflow from the tank should be equal to the capacity of the outflow receiving sewer. This means that the maximum flow (in point D from Fig. 2) should be the same for any construction

Fig. 4 Outflow from the tank in time calculated for the data presented in Table 1, for both positions of outflow (basic Fig. 1a and alternative Fig. 1b), and for duration of precipitation equal to $1.1 \cdot t_{dd}$. For the SWMM computations the length of the outflow pipe = 30.0 m and its slope $S = 2.9‰$ were assumed
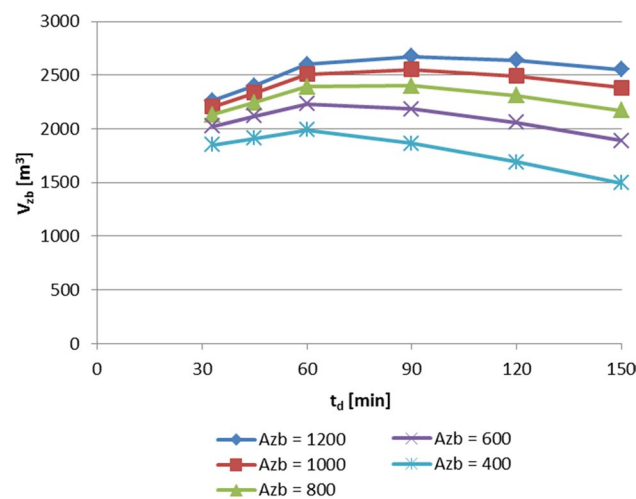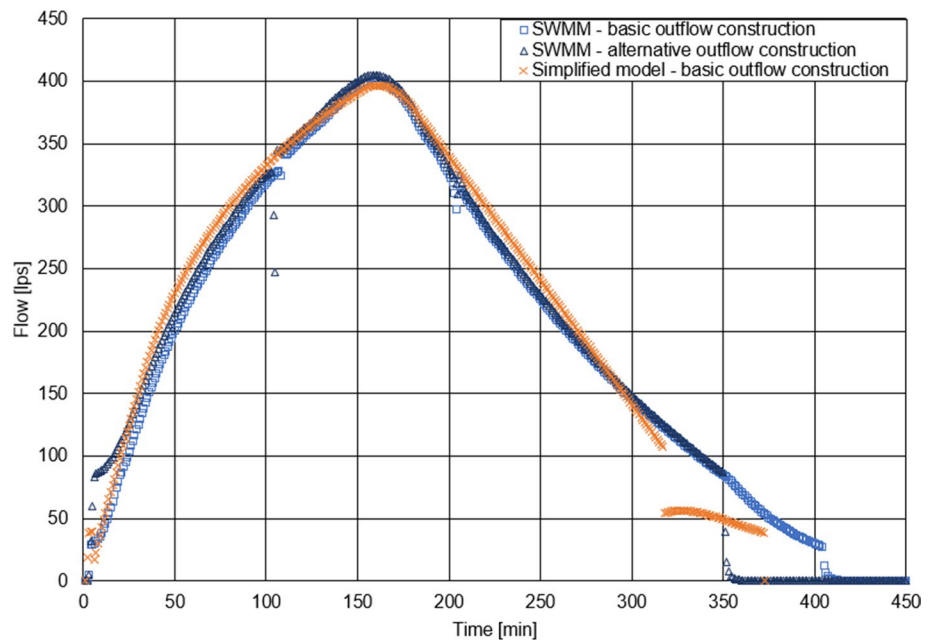
Fig. 6 Volume of storm water accumulated in the tank V_{zb} as a function of the storm duration t_d and the bottom surface area A_{zb} of the tank expressed in m²

details of the outflow, so in this example for both the case from Fig. 1a and b. Providing the computations with several different bottom surface areas A of the tank (see Fig. 6 as an example), it is possible to find such solutions for the storm durations $t_{d(Vmax)}$ predicted as presented in Figs. 6 and 7 that for different constructions of outflow from the tank (for example such as presented in Fig. 1a and in Fig. 1b) both curves describing Q_{outf} will cross the same point D laying on the side of the trapeze describing the inflow to the tank. In this way, for two different constructions of outflow and for two differently selected bottom surface areas A, we receive the same maximum value of the ratio $\beta_{max} = Q_{outf}/Q_{inf}$.

Similarities and differences

To investigate in which situations the vertical position of outflow impacts significantly both the outflow itself and the volume of stormwater accumulated in the tank, several tests were conducted. Each time the data from Table 1 were applied. All computations were done using SWMM exclusively because our simplified model gave very similar results (see Figs. 2, 4, 5), but SWMM much better describes the outflow conditions from tanks. In Fig. 8, the computed depth of stormwater in the tank h was presented for both vertical positions of outflow (basic Fig. 1a and alternative Fig. 1b) and for different surface of the tank bottom. In computations done by SWMM a 30.0 m long sewer installed at the outflow in variable slopes was assumed each time. Depths h presented in Fig. 8 refer to storm durations selected by the trial–error method, as shown in Fig. 3, to give the highest maximal depths h. Obviously larger bottom surface A_{zb} results in lower h values and more significant differences between h values computed for basic (Fig. 1a) and alternative (Fig. 1b) vertical positions of outflow from the tank. The impact of the sewer slope on the depth of stormwater in the tank is rather marginal.

In Fig. 9, the outflows in time were compared for both (basic and alternative) vertical positions of outflow for the surface area of the tank bottom $A_{zb} = 2500$ m² and several outflow slopes. The impact of the outflow vertical position from a tank is not negligible from the economical and technical point of view what can be observed in Fig. 9. The relative differences between maximum outflows from a tank are about 7‰ for the same duration of storm, the

Fig. 7 Another example of the inflow and two different outflows from the tank as a function of time. Bottom tank area in the alternative version was adjusted to gain the same maximum outflow values (D-point)
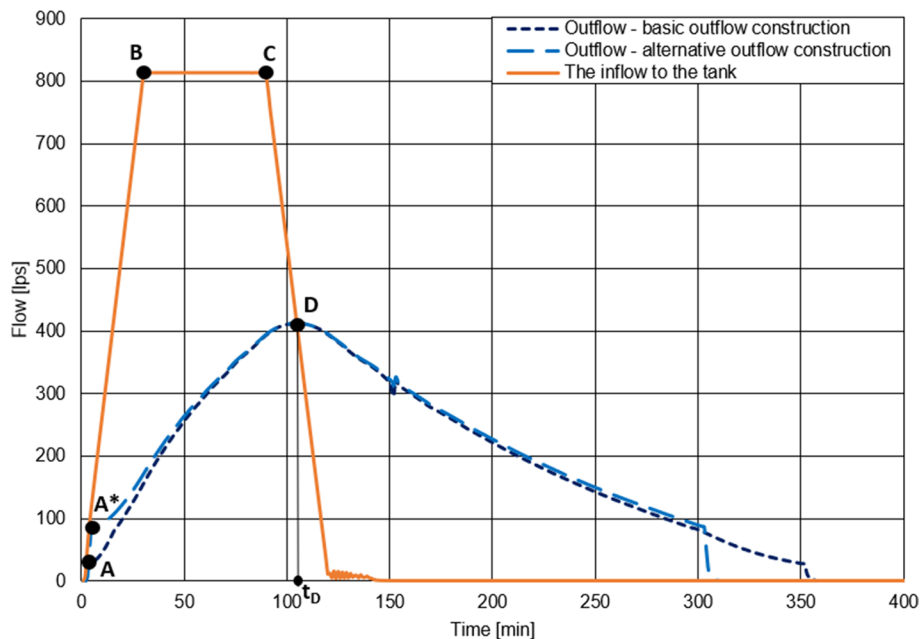
Fig. 8 Depth of stormwater in the tank for different bottom surface areas and different slopes of outflow
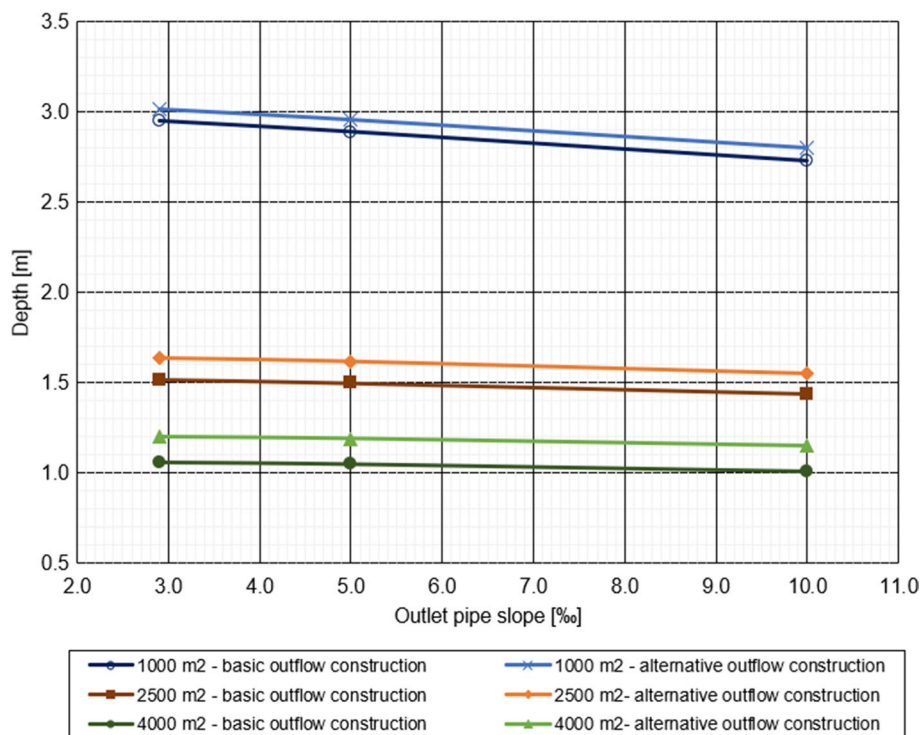
same bottom surface of a tank and the slope of outflow 10‰ but for the basic and alternative position (see Fig. 1) of outflow in the tank. These relative differences are up to 19% for the same position of outflow and for different slopes. The absolute differences between flow rates Q are visibly higher for larger flows referring to higher h.

ISO standard impact

Until 2008, the sewerage design in Poland was based on the national guidelines, according to which the size of channels and their slopes were selected so that the pressure line for the applied rainfall recurrences C was located no

Fig. 9 Impact of vertical position of outflow from the tank on outflow rates in time, for the surface area of the tank bottom $A_{zb} = 2500$ m^2 and for the storm duration equals to $3 \cdot t_{dd}$
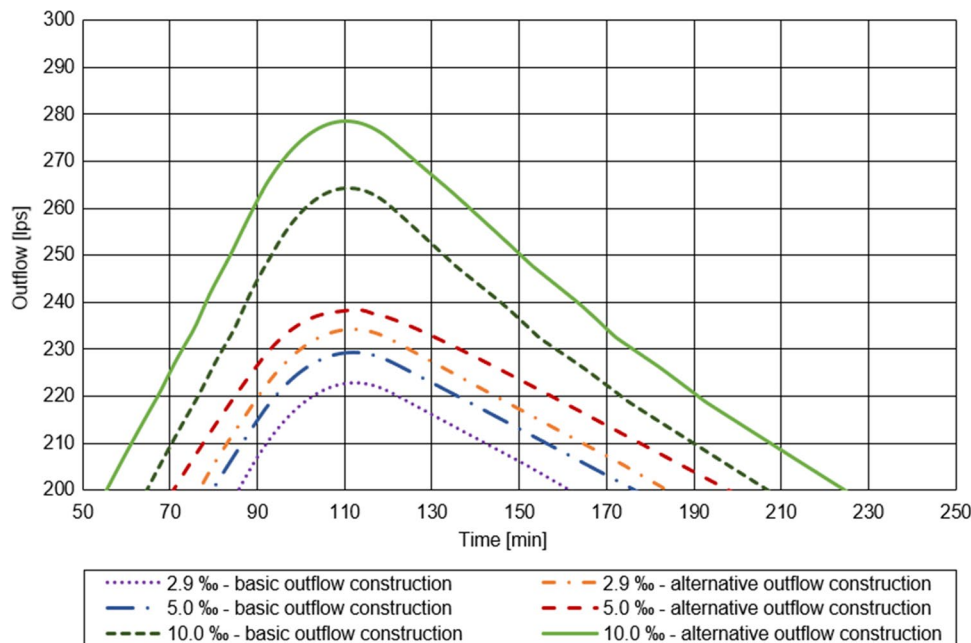

higher than 0.5 m below the surface of the ground. These C frequencies for storm sewers were 1 year in undeveloped areas, 2 years in residential areas, 5 years for city centers, and 10 years for underpasses. In 2000, and again in 2008, the Polish Committee for Standardization adopted the standards PN-EN 752–2:2000 and PN-EN 752–2008, which were updated in 2017 to EN 752:2017.

According to this standard, the rainfall recurrence intervals C, for which it is necessary to check whether a sewer does not flow under pressure, are identical or nearly identical to those that have been used in national guidelines for many years. However, there are new requirements to verify that in large sewer systems, for the new rainfall recurrence intervals C, the sewer is not overflowing or unable to receive surface water runoff. This time, suggested in the standard rainfall recurrences C are significantly higher and are equal in stormwater systems in undeveloped areas to $C = 10$ years, in residential areas $C = 20$ years, in urban centers $C = 33$ years, and for underground infrastructure $C = 50$ years.

Admittedly, this new requirement in Poland refers to large sewer systems and furthermore, the standard gives precedence to national regulations, if any exists, but to maintain similar safety of drainage operation in Poland, as in most European countries, it would be necessary to face these new requirements for rainfall recurrence rates such as those suggested in PN-EN 752–2017. Where the flat terrain forces the sewers to be significantly recessed, one can hope to increase the flow by allowing a sufficiently high hydraulic gradient. For a fully filled channel cross section, the flow is proportional to approximately the square root of the hydraulic gradient (Nalluri, Dąbrowski 1994). On the other hand, the rainfall intensity, of course

partially transformed into surface runoff according to the time-varying value of the runoff coefficient, is according to Blaszczyk's Eq. (1) proportional to the third root of the rainfall recurrence rate.

However, in terrain conditions that allow the shallow laying of sewers, one cannot count on a significant increase in hydraulic gradient above the terrain slope and then this new condition for checking the ability to collect rainwater from the terrain surface is a significant problem. In order to check how much it may cause the need to increase the volume of retention tanks, calculations were carried out for the same initial data as before, but for rainfall recurrence rates of once in 5, 10, and 20 years. The results of the calculations are shown in Fig. 10.
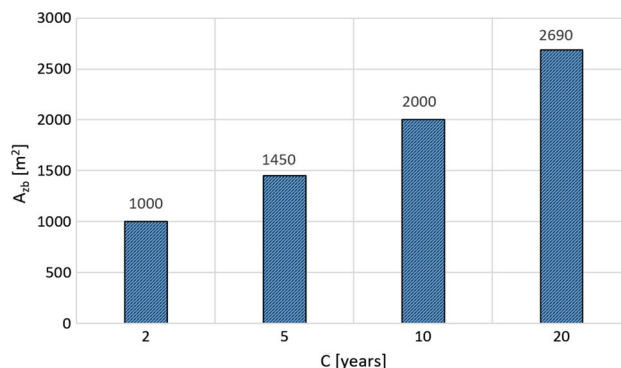
Fig. 10 Required increase in designed storage tank bottom area necessary to withstand higher values of rainfall recurrence rates

What is not considered in the models?

Simple sewerage rational models are based on a static image of rainfall as if the precipitation cloud hovers over the catchment and waits in place until it precipitates and turns into rain. In reality, wind speeds at 10 km altitude can be 200, 300, and sometimes up to 400 km/h. As the altitude decreases this speed also reduces and at an altitude of two kilometers it rarely exceeds 100 km/h. At low cloud ceilings, the speed of the moving rain wave can be so low that it approaches the velocity of the flow through the rain sewers. Then, the wind direction can have a real impact on the sewer flow rate values. While the gutter systems are dimensioned for rainfall intensities of 0.03 l–0.035 l/(s•m^2) in temperate climates, the short, single sewers are dimensioned for rainfall duration of 10 min, which corresponds to a design rainfall of about 132 l/(s•ha) in central Poland. On the other hand, the main sewers are dimensioned for long design rainfalls, which results in design rainfall intensities of 5–10 l/(s•ha). According to the calculations, such sewers should not fill faster than after several hours of continuous rainfall. Despite this, there are known rare cases of monitoring flows in such sewers with full cross section after time shorter than even half an hour. This can be explained by the movement of a front of short-period heavy rainfall along the direction of storm water flow in the main sewers at a speed similar to the flow velocity through these sewers.

A second phenomenon that is not usually considered is the probability that the retention tank is partially filled at the beginning of the next rain. If the role of such a tank is to reduce the frequency of storm overflows in the combined sewer system, the reservoir remains partially filled with combined sewage after the rain has stopped, which is systematically directed to the sewage treatment plant. In this case, this probability can be relatively high. If, as here, the sizing of the storage tank is considered due to the relief of an existing sewerage system, then when the relief concerns one of the main sewers sized for long-term rainfall this probability can be estimated by the methods used for this purpose (Becciu G., Raimondi A (2012), (2015); Raimondi A., Becciu G. (2015)). On the other hand, if the tank relieves a short sewer lying at the edge of the catchment, it is dimensioned for short representative rainfall durations in the range of 10 min to 1 h wherein the probability that it is partially filled at the very beginning of the next rainfall is significantly lower.

Main conclusions

Even small construction details of inflow and outflow to and from storage tanks may be important for computing their required volume not only for multi-chamber, or other complex tanks (Słyś 2010; Dziopak, Niemcynowicz 1999), but even for simple tank constructions through which stormwater flows directly. In the example discussed here for two different vertical positions of the outflow from the tank (Fig. 1a and b) both the tank required volume and the outflow as a function of time (Fig. 2) depended on the vertical outflow position. That was especially true for large tank bottom surface area, so for low maximum depth of flow. This was proofed for a rectangular chamber storage tank of the same bottom surface area. However, if the bottom surface area and the storage tank height were calculated in such a way to enable the same maximum value of an outflow from the tank for two different positions of the sewer (Fig. 1a and b) both the tank volume and the outflow as a function of time were proved to be much more similar (Fig. 7).

The second conclusion refers to the relatively new requirement that all drainage systems in Poland, and wider in Europe, should be able to fulfil. For large cities, the sewerage systems should be able to collect entire runoff and transport storm water for high precipitation recurrence values, much higher than specified for verifying the condition of the full depth flow. This additional requirement is specified in the actual standard EN 752:2017. The results of computations presented in Fig. 10 indicate that in this example it would be necessary to enlarge the storage tank volume by 45% for $C = 5$ years, 100% for $C = 10$ years, and by about 170% for $C = 20$ years to fulfil this additional requirement. Before 2000 no such requirement was applied in designing sewerage systems, so it is one of the reasons why the drainage systems in Poland are below the common European level standard.

Declarations

Conflict of Interest The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Ethical Standards The authors declare that this paper complies with generally accepted ethical standards.

Ethical Conduct The authors confirm that they followed the rules of ethics in preparing the article and conducting the research.

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