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Comparative Analysis of Methods for Dimensioning of Storage Reservoirs in Sewage Systems

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1. Introduction

Storage reservoirs are one of the basic and efficient component used today in stormwater management. A crucial issue in a design process of storage reservoir is to calculate the optimal volume considering both: operational requirements (admissible frequency of flooding) and reduction of investment costs.

Storage volume is estimated by calculating the differences between the inflow and outflow hydrographs. The basic equation for these calculations is [11]:

$$V_R = \int_0^{t_0} \left(Q_{in} - Q_{out}\right) dt \quad (1)$$

where: V_R – required storage volume,
t_0 – time when the outflow hydrograph intersects the inflow hydrograph,
Q_{in} – inflow rate,
Q_{out} – outflow rate.

The design inflow into detention facility is usually defined by the design storm which is used to calculate an inflow hydrograph. The volume of detention storage reservoir can be calculated in many different ways. The following are some of the categories of methods that are used to calculate storage volumes [11]:

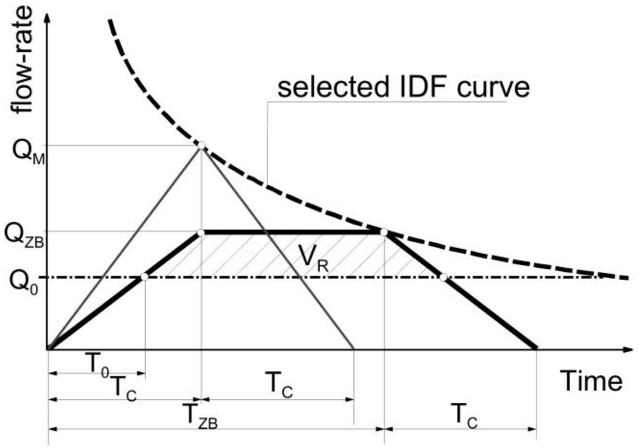
- calculations based on Intensity-Duration-Frequency (IDF) relationship or Intensity-Duration-Frequency-Area (IDFA) relationship,
- rain point diagram method,
- simple rainfall-runoff models (i.e. time-area method),
- detailed hydrodynamic models (i.e. SWMM).

The choice of the method depends on: required accuracy, available precipitation data for the site and also on time and financial sources planned for design phase.

2. Design methods – overwiev

2.1. Intensity-Duration-Frequency relationship

The required storage volume is determined by finding the maximum difference between the areas under trapezoidal or triangular inflow hydrographs and the desired basin release discharge rate Q_0 (see Figure 1).



Rys. 1. Obliczanie objętości retencyjnej w oparciu o krzywe IDF
Fig. 1. Storage volume calculation using IDF relationship

The trapezoid is constructed by drawing a horizontal line back from the value on IDF curve found at duration T_{ZB}(duration of rainfall resulting max. required volume) to T_C (time of concentration). For this method crucial issue is to select the proper IDF relationship. In Poland two IDF relationship are commonly used for engineering purposes [5]:

a) Blaszczyk equation (data collection period 1890–1960):

$$q = \frac{471 \cdot \sqrt[3]{c}}{T_d^{0,667}} \quad \left[\frac{dm^3}{s \cdot ha}\right] \tag{2}$$

where: T_d – rainfall duration [min],
c – return period) [a],

Even the relationship is based on data collected in Warsaw only, the equation was recommended for a whole country;
b) equation proposed by Polish Institute of Meteorology and Water Management (data collection period 1961–1990, 25 stations):

$$P = 1,42\ T_d^{0,33} + \alpha\left(-\ln p\right)^{0,548} \quad [mm] \tag{3}$$

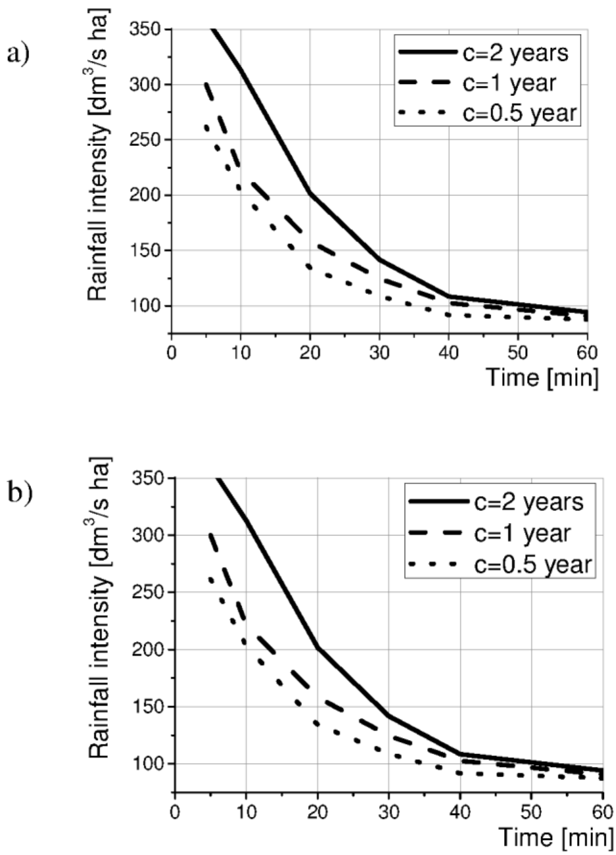
where: T_d – rainfall duration [min],
p – probability of exceedance (i.e. if return period is 10 years then p = 0,1),
α – dimensionless geographical coefficient (separate equations for two regions of Poland).

Usually the ready-to-use nomograhps are available to quick calculations. The data required for dimensioning are:desired basin release discharge rate Q_0, time of concentration T_C, return period c and total impervious area F_{IMP}. Frequently the release rate is given as unit value $q_0 = Q_0/F_{IMP}$, so the calculated storage volume is also unit value: VJ in m^3 on hectare of impervious area. Required total volume (V_R) is then calculate as:

$$V_R = VJ \cdot F_{IMP}\ [m^3] \tag{4}$$

2.2. Intensity-Duration-Frequency-Area relationship

Method based on IDF curves doesn’t take into consideration temporal and spatial variability of rainfalls [1,14]. Thus for a large catchments it may introduce significant error in calculations [8]. This error can be reduced by using IDFA or ARF (Areal Reduction Factor) relationships carried out for local precipitation data. In years 2007–2009 author was collecting rainfall data on five raingauges located at the urban catchment of total area 12.5 km^2 [5]. Obtained data made possible to define IDFA relationship for Czestochowa city (sample curves can be seen on Figure 2 for catchment of area 2.5 km^2 and 12.5 km^2). The way to calculate required storage volume is exactly the same as for standard IDF relationship (Fig. 1), the only difference is pre-selection of curve with respect of the catchment area. Local IDFA seems to be more reliable hydrologic source in comparison to a country-scale universal IDF equations.

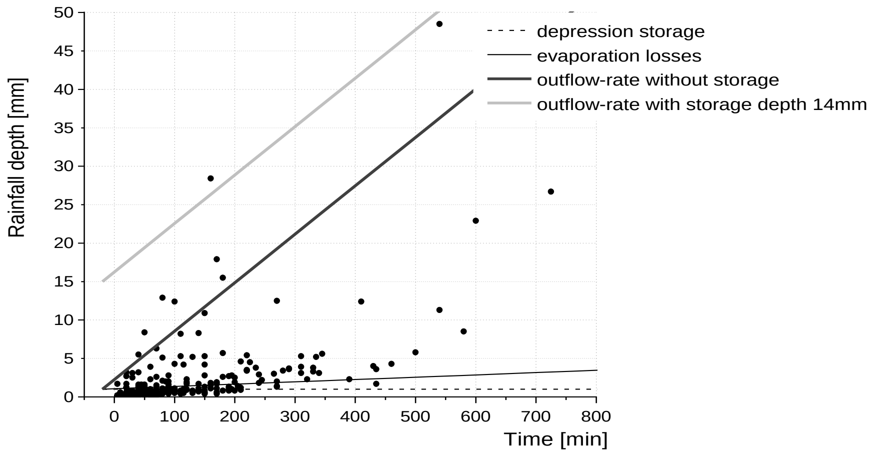


Rys. 2. Krzywe IDFA dla Częstochowy: a) F = 2.5 km^2 (1 deszczomierz), b) 12.5 km^2 (5 deszczomierzy) [7]

Fig. 2. IDFA curves for Czestochowa: a) F = 2.5 km^2 (1 rain gauge), b) 12.5 km^2 (5 rain gauges) [7]

2.3. Rain point diagram method

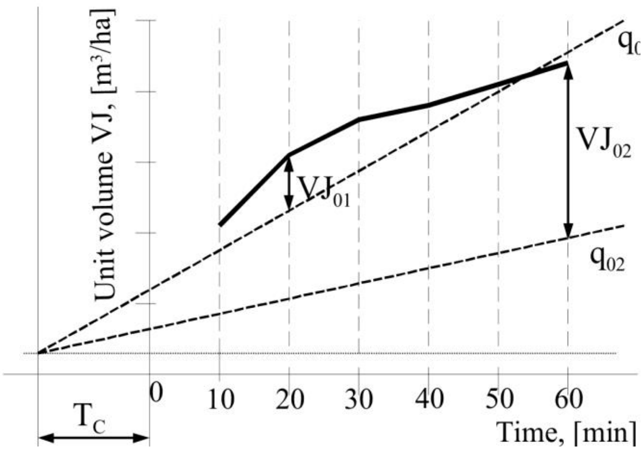
This method requires the preparation of diagram by plotting total rainfall volume against rainstorm duration for required period [10]. The main advantage is that actual rainfall data for the town is used instead of IDF curves being only statistical representation of historical rainfalls occurred at country-scale [11]. The main assumption of this method is the runoff from unpaved surfaces is equal zero, alike in method based on IDF curves. This assumption may be serious error on areas characterized by a low infiltration rate, when during high intensity rainfalls the runoff is generated also from pervious surfaces because depression storage is limited.



Rys. 3. Przykład zastosowania metody diagramu opadów do wymiarowania zbiornika retencyjnego
Fig. 3. Sample of rain point diagram method for dimensioning of storage tank

Depression storage of the impervious surfaces range from 0.5 mm to even 2.5 mm according to average slope, 1 mm is commonly used by authors. Evaporation losses are often neglected because their effect during short duration rainfall events is negligible. Figure 3 presents an example of sizing procedure based on the rain point diagram plot for following input data: maximum outflow-rate q_0 = 10 $dm^3/(s ha_{imp})$, time of concentration T_C = 24 min, depression storage 1,0 mm.Because the rain point method ignores the temporal and spatial distribution of the rainfall intensity, error of calculations for particular event may be significant.

Therefore author propose to construct diagram including rainfall characteristics instead of one-point value. The characteristics is constructed by selection of the highest rainfall depths for incremental time steps ($h_{max10min}$, $h_{max20min}$, $h_{max30min}$ etc.) – it is especially important when the release discharge rate q_0 is higher than 30 dm^3/sha. An example on Figure 4 shows that for high value of release rate (q_{01}) a 20 minutes period of selected rainfall event is critical, while for low release rate (q_{02}) it is 60-minutes respectively. If the sufficient continuous data are available this method may be an alternative for time-consuming hydrodynamic simulation modelling [6].



Rys. 4. Przykład zastosowania metody charakterystyk opadów do wymiarowania zbiornika retencyjnego [5]
Fig. 4. Sample of rain characteristics method for dimensioning of storage tank [5]

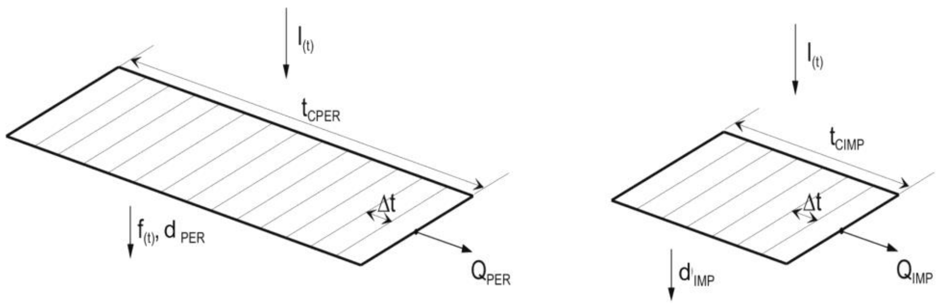
2.4. Conceptual rainfall-runoff model

Simple and comprehensive models require limited effort and data input to compute accurate runoff hydrographs. There are many simple rainfall-runoff models described in literature at different level of simplifications and limitations [12, 13]. Therefore author proposed modified rational method (called TEO) described in details in [4]. The proposed method uses the time of concentration to take the physical properties of a catchment (slope, roughness coefficient, flow path length etc) into account. The method is based on the linear system theory described earlier by Guo [3] and on the rational hydrograph method proposed by

Crobeddu et al. [2]. The following assumptions are considered in the model (see Figure 5):

- the catchment consists of a pervious and an impervious part, both are rectangular in shape (each time-step Δt contribute the same portion of the catchment),
- the time of concentration is different for the pervious (t_{CPER}) and impervious (t_{CIMP}) areas, and is independent from rainfall intensity ($I_{(t)}$)and duration,
- the rainfall is uniform on the catchment area,
- Horton equation is used to calculate the infiltration rate $f_{(t)}$,
- hydrologic losses are represented by the storage depth (d_{IMP} and d_{PER}) and are filled first during rainfall.

Model TEO gives accurate results but the parameter t_{CIMP} have to be calibrated – other parameters may be assumed without significant influence on final results [3].



Rys. 5. Schematogólnymodelu TEO [4]
Fig. 5. General scheme of TEO model [4]

The model transforms given rainfall precipitation into outflow hydrograph from the catchment. If the release discharge rate is constant in time (q_0 = const) the required storage volume can be easily calculated using computer spreadsheet according to equation(1).

2.5. Hydrodynamic models

Hydrodynamic models are the most accurate methods when consider sizing and evaluating the operational performance of storage facilities. Although the computer models perform a large number of hydraulics calculations each second, their reliability and accuracy depends entirely on calibration against recorded simultaneous rainfall and runoff data.

For comparative analysis author chose EPA SWMM5 [9], the most widely-known rainfall-runoff simulation model used for single event or long-term (continuous) simulations. Based on a numerical maps and aerial photographs the whole catchment was divided into homogeneous subcatchment taking into account their shape, slope, land-use type and soil conditions.

The main parameters of the hydrodynamic model of the catchment and sewer system:

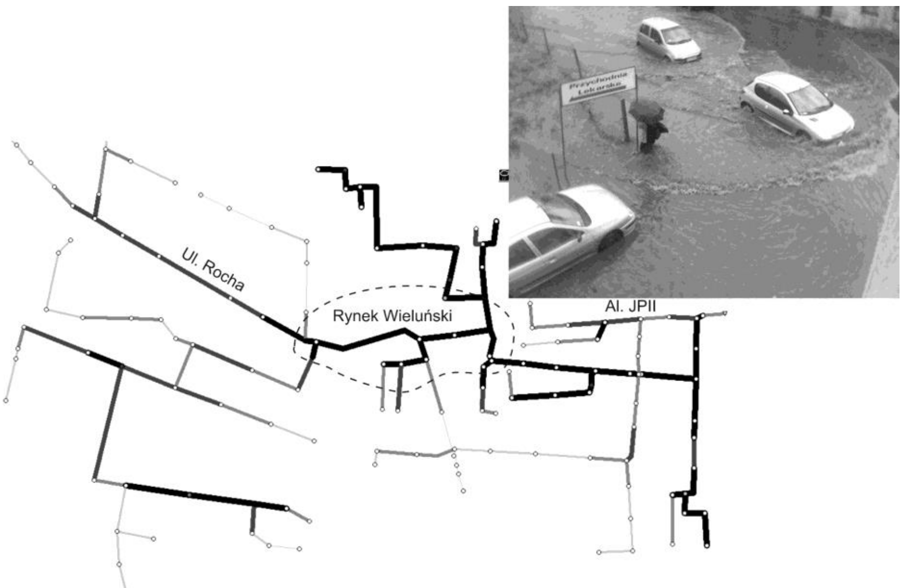
- the catchment area covers 550 ha of which approximately 30% are impervious (in case A only a part of the catchment is used)
- the whole catchment have been divided into 200 subcatchments of area from 0.2 to 10 ha, slopes of the subcatchments between 0.1% to 6%,
- the whole network consist 415 links (diameters between 250 mm and 2000 mm), 410 nodes and one outfall,
- total length of the sewer channels is approximately 35 km, whole network is made of concrete pipes (Manning's coefficient 0.013 was used),
- based on a literature values the depression storage for impervious areas was assumed as 7–12 mm while for pervious areas ranged 1.0–2.0 mm.
- The Horton infiltration equation (f0 = 60 mm/h, fc = 105 mm/h and k = 3 hr-1) was used to estimate the infiltration of stormwater on the pervious portions of the catchment.

Dynamic wave was selected as routing method with time step 1 second. For the hydraulic validation, a very accurate adjustment in terms of time and variation of flows was obtained, and the total volume simulated presented only a difference of 7% with respect to the measured volume. Although the observed peak flows were greater than simulated ones by 10–15%. The simulation gives an accurate results also for a flooding locations and their scale – the comparison was based on a photo documentation of the flooded streets.

3. Comparison of sizing methods

3.1. Case A

In this site the main aim of storage facility is to protect the part of main catchment (area of 127 ha, 24 ha impervious) against frequent flooding (Fig. 6). Storage tank is required to reduce unit outflow rate to value q_0 = 50 dm^3/sha (ha of impervious surface) and return period c = 2 years (allowable flooding frequency: once a two years). Selected 14 rainfall events (recorded on 5 raingauges at Czestochowa) of high intensity were tested on the developed model. The results were classified in descending order according to the volumes that overflow from the system. Because one flooding is allowable, the second result of hydrodynamic simulations was selected as the reference value (V_R = 1250 m^3, thus unit volume VJ = 52 m^3/ha).



Rys. 6. Schemat sieci kanalizacji deszczowej dla przypadku A
Fig. 6. Scheme of drainage system in case A

Calculations based on IDF and IDFA relationships required pre-calculations of the design rainfall duration and intensity (T_C and q_C).It had to be found by trial and error method with 2 minutes step for each

IDF equation separately. For Blaszczyk equation the critical rainfall had parameters T_C = 26 min and q_C = 67.4 dm^3/sha, while for IMiGW equation it is T_C = 22 min and q_C = 113.5 dm^3/sha. The required storage volume VJ is then equal: for Blaszczyk equation VJ = 7,4 m^3/ha (significant underestimation!) and for IMiGW equation VJ = 42.5 m^3/ha (underestimation 18% in comparison to the reference volume). Local IDFA curve presented on Fig. 3a (point raingauge, F = 2.5 km^2) was used to calculate VJ. The final result VJ = 72 m^3/ha overestimates the required volume by 40%.

Rain point diagram was constructed using selected 14 rainfall events of high intensity. Because there are no point above outflow line so it means that no storage volume is required (VJ = 0 m^3/ha (!)) using this method. This case clearly shows the limitations of rain point diagram method. Modification proposed by author – construction of the individual characteristics for each event – significantly changed the results in comparison to a standard (point) diagram. The required storage volume was equal to 41 m^3/ha.

Rainfall-runoff model TEO – the simplified model was used with the following parameters: one subcatchment, t_{CIMP} = 21 min (calibrated), t_{CPER} = 42 min (assumed as $2 \cdot t_{CIMP}$), depression storages d_{IMP} = 1.5 mm and d_{PER} = 7 mm (assumed). The simulations, using spreadsheet, for each event gave outflow hydrographs and consequently, for constant release rate (q_0 = 50 dm^3/sha) then required volumes were calculated. Similarly like for hydrodynamic simulations the second highest value in descending order was the required storage volume Obtained value, VJ = 50,1 m^3/ha, is very close to the reference value. This result is specially valuable when consider time effort needed to construct hydrodynamic model of the sewer system and assigned catchment.

3.2. Case B

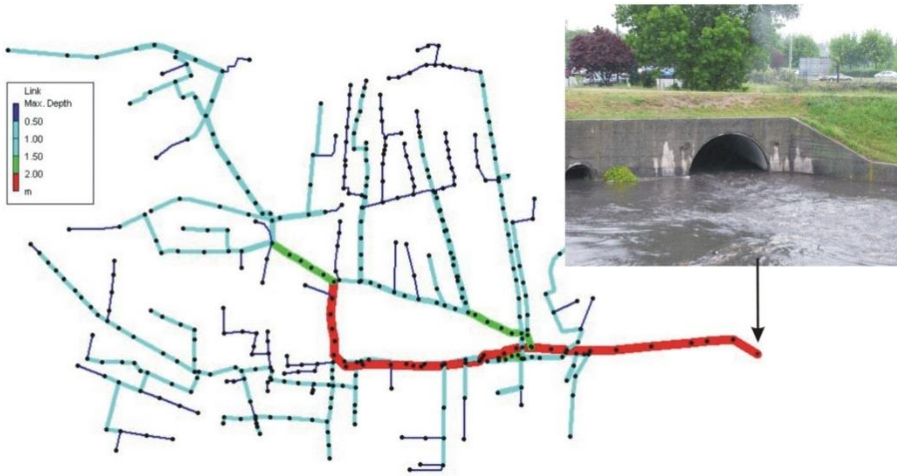
In the second case the storage reservoir is designed as the part of treatment train at the outlet of the urban drainage system (fig. 7). The volume had to be calculated considering two factors:

a) unit outflow-rate to be treated is equal to 15 dm^3/sha,
b) return period is equal to 1 year (during 3 years 3 overflows are allowable).

Similarly like in case A results of hydrodynamic simulations have been sorted descending and the fourth result has been chosen as the reference value related to impervious surface (VJ = 116 m^3/ha).

For calculations based on IDF and IDFA relationships similarly like in case A the first step was to estimate the design storm parameters(T_C and q_C).Then it was possible to estimate the unit volumes: $VJ_{Blaszczyk}$ = 42 m^3/ha and VJ_{IMiGW} = 8 m^3/ha. Local IDFA curve presented on Fig. 3b (five rain gauges, F = 12.5 km^2) was used to calculate VJ and obtained value (VJ = 101 m^3/ha) underestimates volume by about 15%.

Standard rain point diagram gave underestimated volume again (VJ = 74 m^3/ha), even though the spatial variability was taken into account – whole catchment was divided into 5 subcatchments with assigned raingauge. Rainfall characteristics diagram significantly improved this result (VJ = 102 m^3/ha).



Rys. 7. Schemat sieci kanalizacji deszczowej dla przypadku B
Fig. 7. Scheme of drainage system in case B

Usage of rainfall-runoff model TEO was more complicated due to siye of modeled catchment. It was divided into 8 subcatchments with individual times t_{CIMP} (3 were calibrated, 5 assumed). Similarly like in case A simplified rainfall-runoff model was the most accurate method

(VJ = 109,2 m^3/ha) for storage volume estimation in relation to the hydrodynamic simulations.

To show the influence of the release discharge rate q_0 on the demanding storage volume the calculations were done also for q_0 = 7.5 dm^3/sha and 30 dm^3/sha (see Table 1).

Tabela 1. Obliczone jednostkowe pojemności retencyjne VJ [m^3/ha] przy zastosowaniu rożnych metod wymiarowania
Table 1. Estimated unit storage volume VJ [m^3/ha] using different dimensioning methods

q_0 [dm^3/sha]	Model						
	Model hydrodynamic	IDF Blaszczyk	IDF IMiGW	Local IDFA	Diagram standard	Diagram modified	Model TEO
7.5	150.5	77.0	20.8	120.5	129.0	136.3	146.5
15.0	116.0	42.0	8.0	101.0	74.0	102.0	109.2
30.0	60.7	10.0	0.0	65.0	32.1	51.2	56.0

Analysing results contained in the table 1 following relationships can be formulated:

- for unit outflow-rates close to 7.5 dm^3/sha there is significant differences regardless of used method (underestimated results achieved only for universal IDF),so it indicates that even for large catchments a lumped methods can be applied to properly estimate storage volume;
- for q_0 >15 dm^3/sha and large catchments the universal IDF relationships should not be used, but surprisinglygood results were obtained using local IDFA relationships.
- for $q_0 \geq 15$ dm^3/sha rain point diagram generates significant error while rain characteristics methodgives results comparable to rainfall-runoff models. The error of 10–20% in comparison to reference model (SWMM) are acceptable taking into account decisively lesser involvement of people and equipment (data collection, calibration process etc.).

3. Conclusions

The analysis of the obtained results makes possible to formulate following general conclusions and recommendations:

- universal IDF curves significantly underestimate the required storage volume and are inappropriate tool for sizing the storage tanks,
- local IDFA curves give better results than IDF ones but uncertainty of the results is still high so it can be used rather in a pre-design phase,
- rain point diagram in a standard version has limited applicability, specially for high values of q_0, and tends to underestimation ofstorage volume. The proposed rainfall characteristic diagram significantly increase accuracy of this method.
- rainfall-runoff model TEO bring a reliable results, similar to obtained from detailed hydrodynamic models although calibration of t_{CIMP} is required. Considering the costs and time needed to develop a numerical model, the model TEO is an promising alternative.

Obviously it's difficult to compare IDF relationships based on 30-years (Blaszczyk's or IMGW equation) observations to only 3-years period. Thus final conclusions have general form and should be verified in future investigations.Availability of precipitation data seems to be the key problem during the design process of storage facilities. If available data characterize spatial and temporal variability of rainfalls, a detailed hydrodynamic model can be superseded by more simple tools (i.e. rainfall characteristic diagram or TEO model) without significant loss of accuracy.

References

1. **Berne A., Delrieu G., Creutin J.-D., Obled C.:** *Temporal and spatial resolution of rainfall measurements required for urban hydrology*.Journal of Hydrology.Vol. 299. Issues 3–4, 166–179 (2004).
2. **Crobeddu E., Bennis S., Rhouzlane S.:** *Improved rational method*. Journal of Hydrology.Vol. 338, 65–72 (2007).
3. **Guo J.C.Y.:** *Rational hydrograph method for small urban watershed*. Journal of Hydrology Engineering.Vol. 6 (4), 352–356 (2001).
4. **Mrowiec M.:** *Modeling of stormwater runoff from urbanized areas.* Polish Journal of Environmental Studies. Series of Monographs.Vol.1, Water management systems in agricultural and industrial regions. 38–44 (2009).

5. **Mrowiec M.:** *Efektywne wymiarowanie i dynamiczna regulacja kanalizacyjnych zbiorników retencyjnych*. Publishing House of Czestochowa University of Technology. Czestochowa, 2009.
6. **Mrowiec M.:** *The application of improved flow diverter for first flush management*. Water Science and Technology, vol. 62, no 9. 2167–2174 (2010).
7. **Mrowiec M., Malmur R.:** *Wpływ przestrzennej zmienności opadów na niezawodność systemów kanalizacyjnych*. Czasopismo Techniczne – Środowisko. Zeszyt 1, rok 108. 1-Ś/2011 (2011).
8. **Niemczynowicz J., Bengtsson L.:** *What practitioners need from theoreticians?* Atmospheric Research. Vol. 42, 5–17 (1996).
9. **Rossmann L., Dickinson R., Schade T., Chan C., Burgess E., Huber W.:** *SWMM5: The USEPA's newest tool for urban drainage analysis*. Proceedings of 10th ICUD, Copenhagen.pp 1–8 (CD edition), 2005.
10. **Schilling W.:** *Rainfall data for urban hydrology: what do we need?* Atmospheric Resources.Vol. 27, 5–21 (1991).
11. **Stahre P., Urbonas B.:** *Stormwater detention for drainage, water quality and CSO management*. Prentice-Hall. New Jersey, 1990.
12. **Thorndahl S., Schaarup-Jensen K.:** *Comparative analysis of uncertainties in urban surface runoff modeling*. 6th Int. Conference NOVATECH Sustainable Techniques and Strategies in Urban Water Management, Lyon. 1631–1638 (2007).
13. **Urbonas B.:** *Stormwater runoff modeling; Is it accurate as we think?* International conference on Urban Runoff Modeling: Intelligent Modeling to Improve Stormwater Management, Arcata USA, 1–12 (2007).
14. **Vaes G., Willems P., Berlamont J.:** *Areal rainfall correction coefficients for small urban catchments*. Atmospheric Research. Vol. 77, 48–59 (2005).

Analiza porównawcza metod wymiarowania zbiorników retencyjnych w systemach kanalizacyjnych

Abstract

Praktycznym problemem przy projektowaniu zbiorników pozostaje procedura ustalania ich wymaganej objętości. W Polsce dominuje podejście skrajnie uproszczone, polegające na wykorzystywaniu uniwersalnych wzorów, których głównym przeznaczeniem było wyznaczanie chwilowych natężeń przepływu do wymiarowania przekrojów kanalizacyjnych nie zaś ustalanie kształtu hydrogramu dopływu do zbiornika. Dodatkowym problemem jest nadal brak aktualnych i wiarygodnych danych o opadach, opracowanych dla warunków krajowych.Dla

potrzeb porównania dokładności metod wymiarowania, jako metodę najdokładniejsza (wartość referencyjną) przyjęto skalibrowany model hydrodynamiczny, wykonany w oparciu o rzeczywiste dane, pochodzące z sieci pluwiografów.W odniesieniu do wyników uzyskanych dla modelu numerycznego zarówno metoda oparta na wzorze IDF wg Błaszczyka jak i wg IMiGW dawały wyraźnie zaniżone wyniki wymaganej objętości retencyjnej. Kluczowe znaczenie dla wiarygodnych obliczeń zbiorników retencyjnych ma korzystanie z lokalnie zarejestrowanych danych o opadach. Zastosowanie opracowanychlokalnych krzywych IDFA (uwzględniających zasięg opadu) do wymiarowania zbiorników dało zdecydowanie lepsze wyniki niż wg uniwersalnych krzywych IDF.

Przeprowadzona analiza wykazała także, że metoda diagramu opadów w swej oryginalnej postaci ma bardzo ograniczoną użyteczność w stosunku do wymiarowania zbiorników odciążających sieć kanalizacyjną. Zaproponowano metodę polegająca na opracowaniu charakterystyk opadowych dla każdego ze zdarzeń, co umożliwia uwzględnienie zmiennego w czasie natężenia opadów i ma zasadnicze znaczenie dla zwiększenia dokładności uzyskiwanych wyników.Uzyskiwany stopień niedoszacowania objętości retencyjnej na poziomie nie przekraczającym 20% pozwala na wstępne rekomendowanie tej metody dla projektowania zbiorników retencyjnych.

Zastosowanie autorskiego modelu transformacji opadu w odpływ (TEO), bazującego na metodzie racjonalnej, umożliwiło uzyskanie hydrogramów dopływu do zbiornika obliczonych w oparciu o zarejestrowane na zlewni hietogramy. Porównanie wyników uzyskiwanych w oparciu o model TEO do modelu nieliniowych zbiorników, zastosowanego w programie SWMM wykazało jego dużą dokładność, przy mniejszej liczbie wymaganych parametrów wejściowych.W rozpatrywanych symulacjach obliczone objętości różniły się o mniej niż 10% od wartości referencyjnej. Możliwość uzyskania wiarygodnych wyników obliczeń bez konieczności wykonywania szczegółowego modelu hydrodynamicznego pozwala na znaczące skrócenie czasu jak i kosztów ponoszonych na etapie projektowania zbiorników retencyjnych.