DOI: https://doi.org/10.14256/JCE.2317.2018

Primljen / Received: 6.1.2018.
Ispravljen / Corrected: 22.6.2018.
Prihvaćen / Accepted: 20.7.2018.
Dostupno online / Available online: 10.9.2018.

Methodology for defining the design storm for sizing the infiltration system

Authors:

Nino Krvavica, PhD. CE
University of Rijeka
Faculty of Civil Engineering
nino.krvavica@gradri.uniri.hr

Krešimir Jaredić, MCE
University of Rijeka
Faculty of Civil Engineering
jaredic.kresimir@gmail.com

Assist.Prof. Josip Rubinić, PhD. CE
University of Rijeka
Faculty of Civil Engineering
jrubinic@gradri.uniri.hr

Preliminary report

Nino Krvavica, Krešimir Jaredić, Josip Rubinić

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A methodology aimed at defining design storm for sizing infiltration systems, as a part of the stormwater management according to the low impact development approach, is presented in the paper. The proposed analyses are based on numerical models for simulating the runoff and discharge of rainfall into infiltration systems. Preliminary design-storm curves for Rijeka are developed, and results of the numerical model sensitivity analysis are presented. An example of the infiltration system design is also shown. The importance of successive analysis of different durations of rainfall, so as to define a critical design situation, is emphasized.

Key words:
LID; BMP, stormwater management, design storm, kinematic wave, rainfall duration

Prethodno priopćenje

Nino Krvavica, Krešimir Jaredić, Josip Rubinić

Metodologija definiranja mjerodavne oborine za dimenzioniranje infiltracijskih sustava

U radu je predstavljena metodologija definiranja mjerodavne oborine za dimenzioniranje infiltracijskih sustava za odvodnju oborinskih voda prema integralnom pristupu. Predloženi proračuni temelje se na numeričkim modelima za proračun otjecanja i infiltracije oborine u upojnim građevinama. Definirane su preliminarne krivulje mjerodavne oborine za područje Rijeke, te su prikazani rezultati analize osjetljivosti numeričkog modela. Također, dan je primjer dimenzioniranja infiltracijske građevine. Naglašena je važnost sukcesivnog proračuna na oborine različitih trajanja, kako bi se sustav dimenzionirao na kritični slučaj.

Ključne riječi:
LID, BMP, odvodnja oborinskih voda, mjerodavna oborina, kinematički val, trajanje oborine

Vorherige Mitteilung

Nino Krvavica, Krešimir Jaredić, Josip Rubinić

Methodologie zur Bestimmung relevanter Niederschläge für die Dimensionierung von Infiltrationssystemen

In der Abhandlung wird die Methodologie zur Bestimmung relevanter Niederschläge für die Dimensionierung von Infiltrationssystemen für die Ableitung von Niederschlägen nach dem integralen Ansatz vorgestellt. Die vorgeschlagenen Berechnungen basieren auf den nummerischen Modellen für die Berechnung des Abflusses und der Infiltration der Niederschläge in saugfähigen Objekten. Definiert werden vorläufige Kurven relevanter Niederschläge für das Gebiet von Rijeka und es werden die Ergebnisse der Analyse der Empfindlichkeit des nummerischen Modells präsentiert. Darüber hinaus wird ein Beispiel der Dimensionierung des Infiltrationsobjektes dargestellt. Betont wird die Wichtigkeit der sukzessiven Berechnung für Niederschläge von unterschiedlicher Dauer, um das System für einen kritischen Fall zu dimensionieren.

Schlüsselwörter:
LID, BMP, Ableitung von Niederschlägen, relevanter Niederschlag, kinematische Welle, Niederschlagsdauer

1. Introduction

Over the past several decades, extreme rainfalls and floods have been increasingly registered in Croatia, with pronounced extremes at the Adriatic coast [1, 2]. Urban areas are especially at risk from pluvial flooding, which occurs because of a combination of rapid and unplanned urbanization, poor maintenance of storm drainage systems, and more frequent occurrence of extreme rainfalls [3]. However, results obtained by a detailed analysis of short-term extreme rainfalls for the cities of Split and Varaždin, representing maritime and continental precipitation regimes in Croatia, point to a statistically insignificant increase of heavy rainfalls in these areas [4]. It was therefore concluded that the cause of more frequent problems with storm drainage systems should be investigated outside of the domain of possible climate change and its influence on short-term heavy rainfalls.

The Sendai Framework for Disaster Risk Reduction for the 2015-2030 period emphasises the importance of preventing adverse effects of urbanisation, and reducing vulnerability of urban areas to flooding risks [5]. Negative aspects of urbanisation and traditional approaches to the design of storm drainage systems are well known and documented in the literature: the runoff volume and velocity increases [6], the time of concentration decreases [7], and the stormwater quality deteriorates [8]. An alternative to traditional stormwater management is the integral approach – namely the Low Impact Development (LID) or the Best Management Practice (BMP) – which is based on the principle of making minor interventions in space in order to restore natural hydrological conditions, the rainwater infiltration capability in particular [9].

The LID approach benefits have been examined in detail under various conditions and at different locations [7, 9-11]. Structural solutions are regularly based on the implementation of infiltration systems, such as bioretentions [12], permeable pavements [13], and infiltration tanks and trenches [14]. In a larger part of Europe, and especially in the USA and Australia, the LID approach has been the first choice in the stormwater management and drainage system design for some time now. On the other hand, this integral approach has only recently become recognized, considered, and (rarely) implemented in Croatia [15-19].

To predict and prevent negative effects of extreme rainfall events, the implementation of the LID approach should be based on reliable hydrological and hydraulic models, where computed values are accurately quantified. On the one hand, overestimated size of drainage systems may generate negative economic effects, and sometimes result in inappropriate concentration of excessive rainwater discharges. On the other hand, underestimated size would not only endanger the urban landscape by uncontrolled rainwater overflow, but would also cause negative environmental effects. In the case of extreme rainfall, and when the storm drainage system is improperly maintained or under capacitated, the polluted stormwater is directly, i.e. without any treatment, released into the environment [20]. Coastal karstic areas are especially vulnerable to water pollution [21]. Furthermore, according to the EU Water Framework Directive (WFD), all European countries are advised to actively work on the prevention of common sources of water pollution, which also includes pollution from stormwater [22].

Unfortunately, outdated and inappropriate drainage-system design methods still prevail in the engineering practice, especially in Croatia, and particularly for infiltration systems, which are a vital part of the LID approach. In this respect, calculations of infiltration wells according to Pönninger [23] can very often be encountered, although they are based on a simple empirical formula that completely neglects rainfall characteristics as an input parameter. Also, there are calculations based on the rational formula for computing the peak runoff discharge [24], combined with a formula for estimating the required infiltration system storage volume (e.g., [25]). Such computations are based on the balance of total rainfall amounts averaged over time, which neglects temporal variability of the rainfall intensity, runoff discharge, and water levels in infiltration systems. Moreover, these computations are conducted for relatively short duration of storm events, which is very often inconsistent with critical scenarios. Modular infiltration systems based on the German standard DWA-A 138E have recently been introduced and implemented [26]. Although such systems involve a series of computations for different storm durations to identify a critical scenario, they still suggest a simple computation of the total volume averaged over time.

This paper presents a new methodology for an optimum design of infiltration systems as an integral part of the storm drainage system. The proposed approach is based on a combination of numerical models, where the input data are defined by a temporal distribution of rainfall intensity – the design storm. Furthermore, this methodology includes a series of computations for different storm durations, so as to design the infiltration systems for a critical scenario. This approach has been tested on an infiltration system implemented in the Rijeka area, which is characterized by high rainfall intensities for all durations up to 24 hours [27]. In this context, the main aim of this work is to:

- define preliminary design storm curves for the Rijeka area
- compare differences between traditional approach based on uniform intensity (HDF curves) and the contemporary approach that considers temporal distribution of rainfall intensity, and investigate its influence on the resulting surface runoff hydrograph and water level changes in an infiltration system
- analyse the model sensitivity to main input parameters (storm duration, watershed area, infiltration velocity)
- provide an example of infiltration system design based on the proposed methodology.

Finally, the method of an optimum infiltration system design is additionally analysed by examining functionality of such a system during a registered extreme storm event.

2. Methodology

The methodology for an optimum design of infiltration systems involves definition of appropriate numerical models for simulating the surface runoff discharge and water volume changes in infiltration systems, with an appropriately defined design storm for a given watershed. The complete model, including all computational algorithms, was implemented in Python 3.6 [28].

2.1. Design storm

Unlike uniform rainfall intensities obtained from HDF or IDF curves, the design storm is characterized by temporal distribution of rainfall intensity, so that an actual storm event can be presented more realistically. Watt and Marsalek [29] recently gave a very detailed review of several design storm approaches. As to situation in Croatia, only several attempts have so far been made (including this paper) to define a design storm. According to Bonacci [30], one of the first attempts was the paper published in 1979 by Bonacci and Stupalo. According to the first author of that study, the analysis was based on "a modest sample of storm events" obtained from the ombrograph stations Split-Marjan, Sinj, Zadar, and Knin. Only short storm durations (up to 60 min) were analysed in that study, and the results were obtained by averaging all recorded heavy rainfall events. Longer durations, for time spans ranging from 20 minutes to 24 hours, were analysed in a separate study [31], where data from the Split-Marjan and Split-Airport stations were analysed in a simple way - by averaging all recorded storm events with durations ranging from 8 to 24 hours. Additionally, Maričić et al. [32] derived a design storm for the city of Osijek, also by averaging characteristics of selected storm events. Therefore, an appropriate design storm has still not been derived for any city in Croatia.

The methodology and preliminary results of the design storm analysis for the city of Rijeka in Croatia are presented in this paper. For that purpose, the Average Variability Method [33] was applied because it provides deterministic design-storm values for a given storm duration and frequency (return period), which is more readily applicable for practical engineering problems, as compared to probabilistic approaches, such as Huff's Quartile Curves [34]. In addition, it provides a satisfactory approximation of temporal distribution of characteristic rainfall intensities. The proposed methodology for processing short-term heavy storm events according to the average variability method is defined as follows [33]:

- Each individual storm event of a given duration (10 minutes to 24 hours in the present study) is first divided into a finite number of equal parts (time periods).
- Total rainfall depths are then ranked for each time period, and the total rainfall depth percentages are assigned to each rank.
- Finally, a new rank is assigned to each time period according to the average value of each rainfall, and a percentage value corresponding to each rank is assigned.

In this paper, the computation and definition of design storm is based on a limited sample of storm events combined with HDF curves for Rijeka [27]. Since the proposed method recommends the use of representative events only, ten years of ombrographic observations from the period of 1961-1990 were selected from the Rijeka station for this preliminary analysis [35]. Out of these ten years, a total of 230 storm events of different durations were selected for further processing according to the predefined lower limits of rain intensity [30]. All storm events were categorized into four duration classes, namely $t < 1$ hr, 1 hr $\leq t < 3$ hr, 3 hr $\leq t < 6$ hr, and 6 hr $\leq t < 24$ hr. The result obtained using the average variability method is a dimensionless curve for each duration class. Finally, the design storm is computed by assigning the total rainfall depth of a given duration and the frequency (return period) from known HDF curves.

Although the design storm represents the temporal variability of a real storm event more realistically than a uniform intensity, certain elements of this approach are still not completely defined. One of such elements, and perhaps the most important one in the computation and design of infiltration systems, is the critical storm duration [36]. It should be noted that this uncertainty is present in engineering practice regardless of the method that is used for representing the actual storm. In urban areas, the surface runoff is usually computed for storm durations equal to the time of concertation, i.e., the response of the watershed to a rain event, which results in a maximum runoff discharge [24]. However, for some elements of storm drainage systems, such as infiltration systems, a more important parameter is the runoff volume. In this case, the critical storm duration for sizing the infiltration system storage volume is not clearly defined, because it is difficult to estimate in advance the nonlinear interaction between all governing parameters, such as the temporal distribution of the rainfall intensity, the time of concentration, the required volume of the infiltration storage, and the infiltration velocity [37]. Therefore, it is necessary to run a series of computations for a range of different storm durations and then select the critical scenario.

2.2. Surface runoff model

Hydraulic models for surface runoff are based on mathematical equations derived from conservation laws of mass and linear momentum, which define physical processes of the time-dependant shallow water flow [24]. The method of kinematic wave [33] is a regular choice for hydraulic computation of surface runoff from small watersheds (< 2.5 km^2), and especially for surface runoff when discharge measurements are unavailable [39]. In general, the kinematic wave method is considered accurate for computing surface runoff from watersheds defined by steeper slopes and for subcritical flow [40].

This method is derived from Saint-Venant equations for a time-dependant shallow water flow in which the inertial and pressure forces have been neglected, i.e., it is assumed that the slope of the energy line is equal to the channel bed slope [38]:

$$\frac{\partial A}{\partial t}+\frac{\partial Q}{\partial x}=q \tag{1}$$

$$S_f - S_0 = 0 \tag{2}$$

where A (m^2) is the wetted cross-section area of the channel, Q (m^3/s) is the discharge, $q = r_{eff} B$ (m^2/s) s the source term defined by rainfall intensity, r_{eff} (m/s) s the effective rainfall intensity, B (m) s the channel breadth, x (m) s the axial coordinate along the channel, t (s) is the time, S_f (-) is the energy line slope and S_0 (-) s the channel bed slope. The water velocity can be computed using the Manning equation [41]:

$$v = \frac{1}{n} R^{2/3} S_f^{1/2} \tag{3}$$

where R (m) is the hydraulic radius, and n (s m$^{-1/3}$) is the Manning roughness coefficient. Combining the Manning equation (3) and Eq. (2) gives:

$$Q = \frac{1}{n} R^{2/3} S_0^{1/2} A \tag{4}$$

For a uniform channel cross-section, Eq. (4) can be simplified to:

$$Q = \alpha A^m \tag{5}$$

where coefficients α and m depend on the channel geometry. In this work, the analysed watershed is an impermeable surface of a prolonged rectangular shape defined by longitudinal slope S_0 and cross-section slope I_0 (Figure 1), so that α and m an be written as:

$$\alpha = \frac{1}{n}\left(\frac{I_0}{2}\right)^{1/3} S_0^{1/2}, \quad m = \frac{4}{3} \tag{6}$$

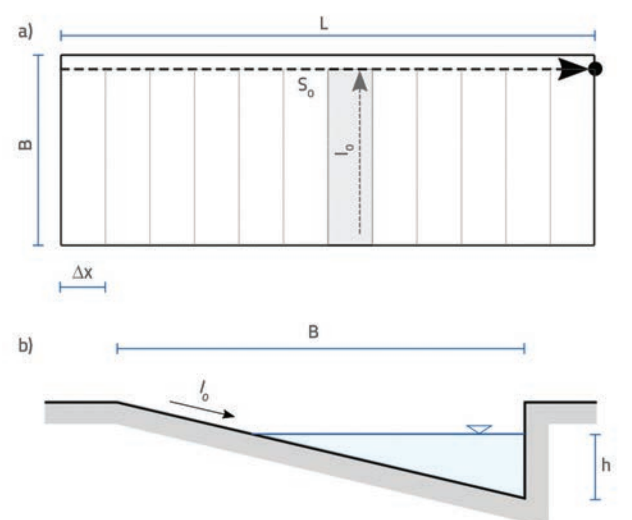

Figure 1. Schematic illustration of: a) plan area of rectangular watershed; b) characteristic cross-section in the direction of runoff sliva

Finally, by introducing Eqs. (6) and (5) in the continuity equation (1), the surface runoff is completely defined by the following governing equations:

$$\frac{\partial A}{\partial t} + \alpha m A^{m-1} \frac{\partial A}{\partial x} = q \tag{7}$$

Since α and m depend on the channel geometry only, the surface runoff is simplified to the calculation of the wetted cross-section area $A(x, t)$ at different locations along the channel, and at different time steps. Equation (7) can be solved by the finite difference method (FDM) that approximates partial derivations. Although several explicit and implicit numerical schemes have been developed for this purpose, the *Forward-time Backward-space* (FTBS) method [42] was used here. This method reduces Eq. (7) to a finite number of algebraic equations of the following form:

$$\frac{A_i^{j+1} - A_i^j}{\Delta t} + \alpha m \left(\frac{A_i^j + A_{i-1}^j}{2}\right)^{m-1} \frac{A_i^j - A_{i-1}^j}{\Delta x} = \frac{q_i^{j+1} + q_i^j}{2} \tag{8}$$

where index i denotes the position in space, index j denotes the position in time, Δt (s) is a time step, and Δx (m) is a spatial step.

2.3. Modelling water level change in infiltration system

Water level changes in infiltration systems – modular infiltration tank (Figure 2) – are defined by the continuity equation [24]:

$$\frac{dV}{dt} = Q_{ul} - Q_{iz} \tag{9}$$

where V (m^3) is the volume of water, Q_{ul} (m^3/s) is the inflow discharge and Q_{iz} (m^3/s) is the outflow discharge. The water volume V inside the infiltration tank can be defined as the product of the tank bottom area A_b (m^2) and the total water depth h (m). Furthermore, the sum of the overflow (crest) height $H_{overflow}$ (vertical distance between the bottom of the tank and the overflow pipe axis) and the overflow head H_p (vertical distance between the overflow pipe axis and the total water level) is equal to the total water depth $h = H_{overflow} + H_p$ (Figure 2). The inflow discharge Q_{in} is defined by the runoff hydrograph computed by the kinematic wave method, and the outflow discharge is defined as the sum of the infiltration flow rate through the tank bottom Q_{up} (m^3/s) and the overflow discharge $Q_{overflow}$ (m^3/s). The infiltration flow rate is computed as follows:

$$Q_{up} = q_{up} A_b \tag{10}$$

where q_{up} (m/s) is the infiltration velocity, and A_b (m^2) The infiltration velocity depends on terrain characteristics, which should be obtained from field tests in various saturation

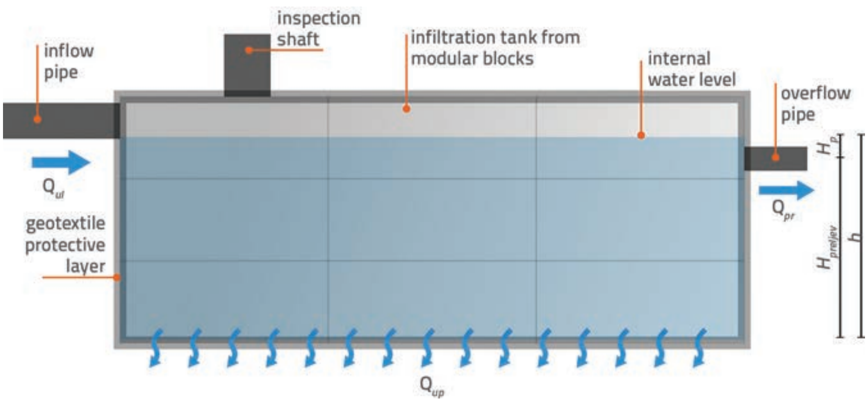

Figure 2. Schematic illustration of modular infiltration tank

conditions. Preliminary values can be derived from known values of hydraulic conductivity K (Table 1). At that, it should be noted that the relationship between q_{inf} and K may be defined by the Darcy law for the one-dimensional and stationary flow of fluid through a porous medium [24]:

$$q_{up} = K \cdot I \tag{11}$$

where $I = dh/dz$ is the hydraulic gradient, h is the head, and z is the vertical coordinate. The hydraulic gradient is usually assumed to be constant and equal to 1 for vertical infiltration in saturated and porous mediums [43].

TTable 1. Ranges of hydraulic conductivity values (according to [44])

Terrain type	Description	Hydraulic conductivity K [m/s]
Soil	Fine sand	10^{-5} do 10^{-4}
	Coarse sand	10^{-4} do 10^{-3}
	Fine gravel	10^{-3} do 5×10^{-1}
	Coarse gravel	10^{-2} do 5×10^{-1}
Rock	Dense limestone	10^{-9} do 10^{-7}
	Karstic limestone	10^{-5} do 10^{-3}

Of course, in field conditions, the medium is not always completely saturated, and the hydraulic parameter should be determined by more complex time-dependant models. However, the former assumption is justified in most cases because I becomes equal to 1 in a relatively short time after the start of infiltration, when compared to the total infiltration time [43]. For a more realistic description of water infiltration into the ground, which includes partially saturated and time-dependant conditions, the Green-Ampt approximate model or Horton empirical expression may be used instead [24], or even a numerical computation of the full Richards quation may be considered [24]. In Croatian practice, the values ranging from 10^{-4} to 10^{-3} m/s are usually used in karst areas and in the planning phase. These values correspond to the hydraulic conductivity of well permeable rocks, i.e., the upper class of the values for highly karstic limestones (Table 1) [44].

The discharge through the safety overflow pipe is determined by the expression [45]:

$$Q_{pr} = \mu A_p \sqrt{2gH_p} \tag{12}$$

where μ is the discharge coefficient, A_p is the cross-section pipe area, g is the acceleration of gravity, and H_p is the overflow head (Figure 2). For round openings with sharp edges, and where the length of the pipe is around 3 to 4 times the pipe diameter, the discharge coefficient is μ = 0.62 [45]. The crest height is usually positioned near the top of the tank, and it is determined according to design conditions, which means that the water overflows from the tank only during extreme storm events.

To solve Eq. (9), FDM can be applied once more to approximate the time derivative of the volume. For appropriately small steps, the forward Euler method is accurate and reliable enough, and it can be written as follows:

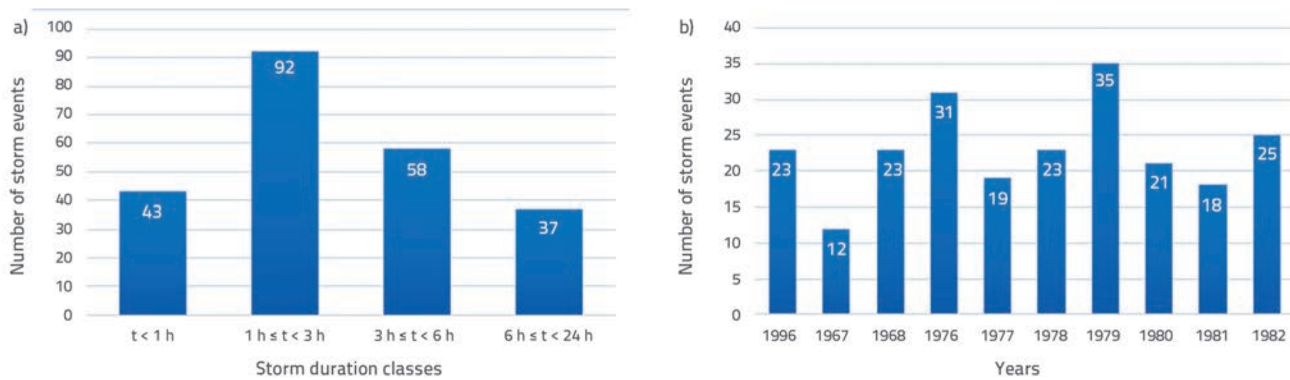

Figure 3. Number of storm events that exceed lower values used in secondary analyses, according to: a) duration class, b) year of occurrence

$$\frac{V^{j+1} - V^j}{\Delta t} = Q_{ul}^j - Q_{up}^j - Q_{pr}^j \quad (13)$$

where, similarly as in the previous numerical scheme, index j is the position in time, and Δt is the time step.

3. Results

3.1. Design storm curves for Rijeka area

Storm events, observed at the Rijeka station during ten years selected from the 1961-1990 period, which exceed the lower values used for secondary analysis, are presented in Figure 3 according to the duration class and the year in which they occurred. Figure 3 shows that the largest number of representative storm events occurred in 1979 when the highest annual rainfall depth was recorded (H_{yr} = 1973.4 mm), whereas the smallest number of rain events occurred in 1967, which is characterised by the lowest annual rainfall depth in the observed period (H_{yr} = 1476.9 mm). Also, it can be observed that the largest number of storm events lasted between 1 and 3 hours, whereas the lowest number of storm events lasted longer than 6 hours. Figure 4 shows the dimensionless design storm curves and statistical indicators (interquartile range) obtained from individual cumulative storm events, for each duration class. The results suggest that the shortest storm events (under 1-hour) have the most pronounced S-shape of the cumulative curve

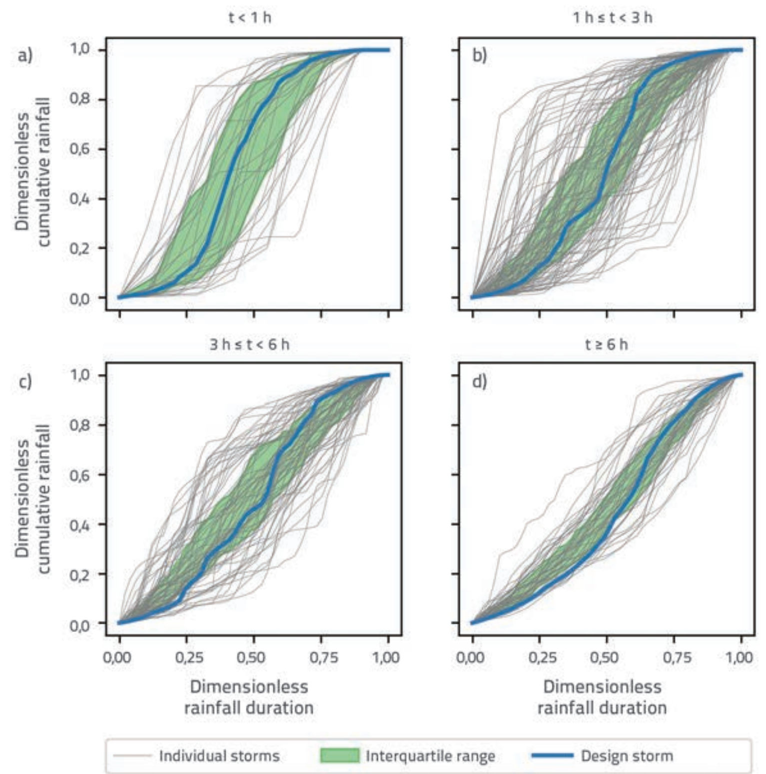

Figure 4. Cumulative rainfall curves for individual storms, interquartile range, and design storm curve for different duration classes: a) less than 1-hour, b) between 1 and 3 hours, c) between 3 and 6 hours, d) between 6 and 24 hours

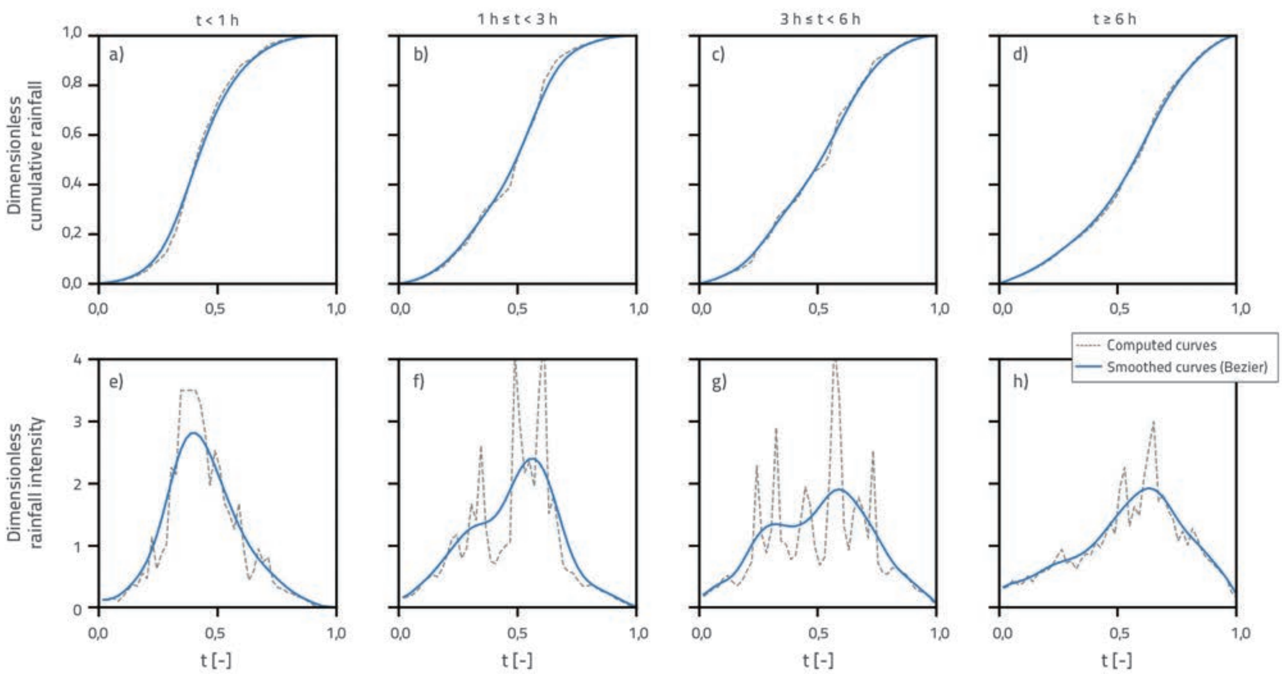

Figure 5. Dimensionless shape of design storm curves for Rijeka, before and after smoothing by Bezier curves, shown as: a) – d) cumulative rainfall depth, e) - h) rainfall intensity

(Figure 4.a), whereas the curve for storm events longer than 6 hours is much closer to a uniform intensity shape (Figure 4.d). This result is expected since the temporal variability is usually more pronounced in shorter, generally convective precipitations, in comparison to longer cyclonic precipitations.

Furthermore, Figure 4 shows that the computed curves are somewhat "noisy", which may result in unrealistic oscillation and peaks when the rainfall intensity is computed. Therefore, smoothing by fitting Bezier curves [46] is additionally performed. The rainfall intensity is obtained from a discrete derivation of the cumulative rainfall depth in time. The design storm curves for different duration classes together with intensity curves, after post-processing, are shown in Figure 5.

3.2. Comparison of design storm and uniform intensity approaches

Various storm durations, i.e. t_k = 30 min, 1-hour, 3 hours, and 6 hours, defined by uniform intensity and by design storm, were analysed in order to compare various approaches for defining an appropriate storm for infiltration tank sizing. The corresponding surface runoff discharges, and volume changes inside the infiltration tank, were also computed. In both cases, the total rainfall depths were computed from HDF curves with a 20-year return period for Rijeka [27]:

$$H_{20g} = \begin{cases} 76,876 \cdot t_k^{\,0,6288} & za \quad t_k < 1,52h \\ 88,307 \cdot t_k^{\,0,2982} & za \quad t_k \geq 1,52h \end{cases} \tag{14}$$

The surface runoff hydrograph was computed for a small, regular, and impermeable watershed, defined by: length L = 600 m, width B = 10 m, constant longitudinal slope S_o = 1.0%, constant cross-section slope I_p = 3.5%, and Manning's roughness coefficient n = 0.015 s m$^{-1/3}$. This watershed can represent various urban areas, such as flat building roofs, parking lots, road sections, squares, etc. All losses in the rainfall-runoff process were ignored, i.e., it was assumed that r_{eff} = i. The infiltration tank bottom area was

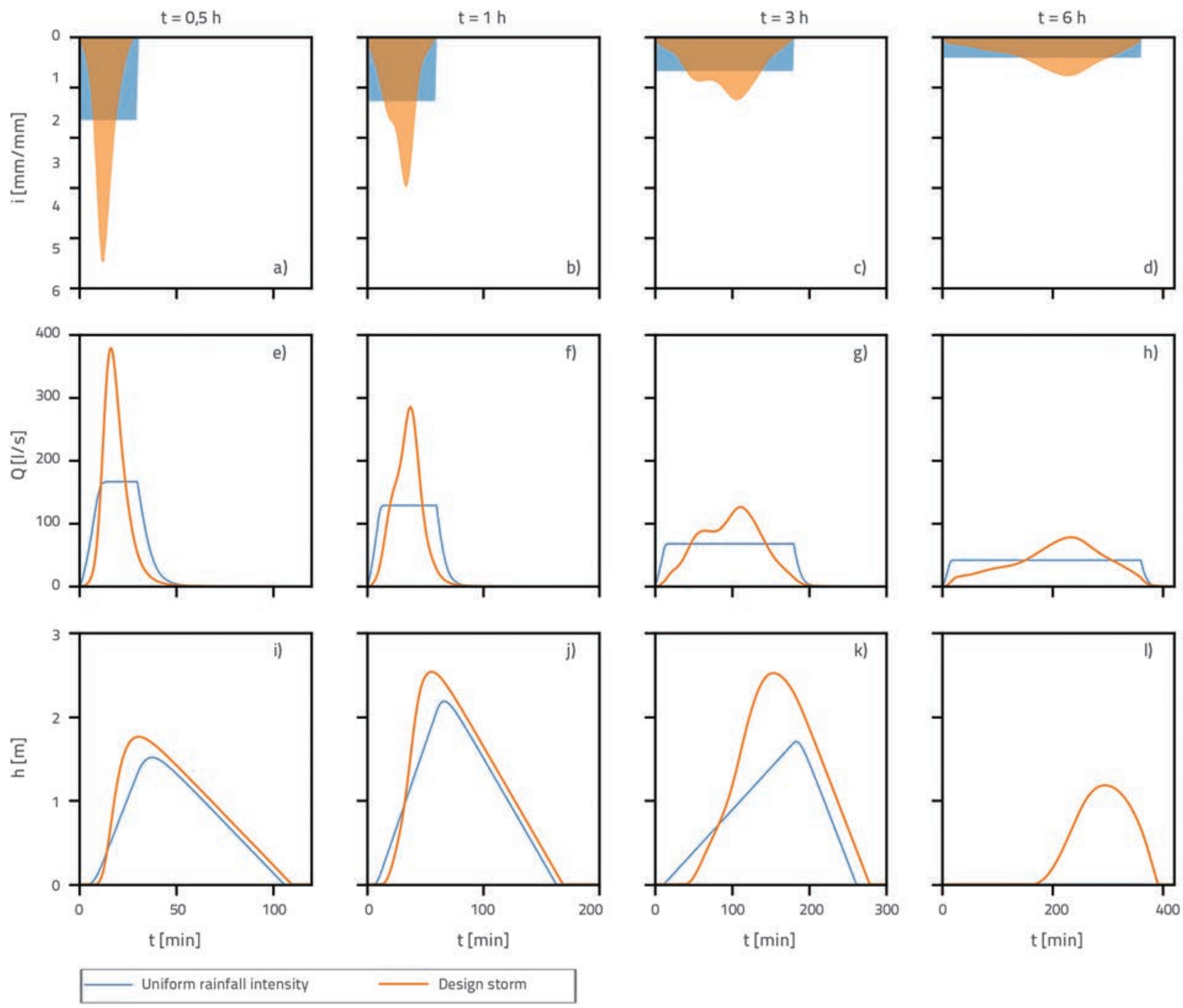

Figure 6. Comparison of uniform intensity and design storm approaches, for different storm durations, illustrated as: a) – d) rainfall intensity, w) – h) runoff discharge, and i) – l) water level changes in infiltration tank

defined as 2% of the watershed area A_b = 120 m^2, the overflow height was $H_{overflow}$ = 3.0 m, and the average infiltration velocity amounted to q_{inf} = 0.4 mm/s.

Figure 6 shows the results obtained by both approaches. Differences between the two can already be noticed in the temporal distribution of rainfall intensity, where the differences are more pronounced for shorter durations (Figures 6a. - 6.d). For example, for 30-minute duration, the peak intensity in the design storm is over two times larger than the average intensity (Figure 6.a). Similarly, the peak runoff discharge generated by the design storm is almost two times larger than the one generated by uniform rainfall intensity (Figure 6e. -6.f). As expected, the maximum discharge was registered for the shortest storm duration (Figure 6.e), because the time of concentration amounted to t_c = 13 min in that particular case. Both approaches, for each storm duration, predict that the same total volume of rainfall is discharged into the infiltration tank. However, because of different temporal distributions, the maximum water level in the tank will be higher in the design storm approach in comparison to the uniform intensity (Figure 6i-l). This difference is more pronounced for storm durations of 3 and 6 hours (Figure 6.k, 6.l). Furthermore, it is interesting to note that the critical storm duration, which is defined by the maximum water level in the tank, is 1-hour for the uniform intensity approach, in comparison to the slightly longer 1 to 3-hour duration for the design storm approach. It seems that the uniform intensity approach underestimates, not only the peak runoff discharge, but also the required volume of the infiltration tank storage.

3.3. Model sensitivity analysis

This section presents the sensitivity analysis of the proposed model for computing the water level changes in infiltration systems, and the critical storm duration considering the main governing parameters: the watershed area that affects the total runoff volume, and the infiltration velocity that affects the required tank volume. As in the previous case, various durations (t_k = 30 min, 1 hour, 3 hours and 6 hours) were considered in the analysis. The time distribution was calculated from the design storm curves (Figure 5), and the total rainfall depth was computed using Eq. (14).

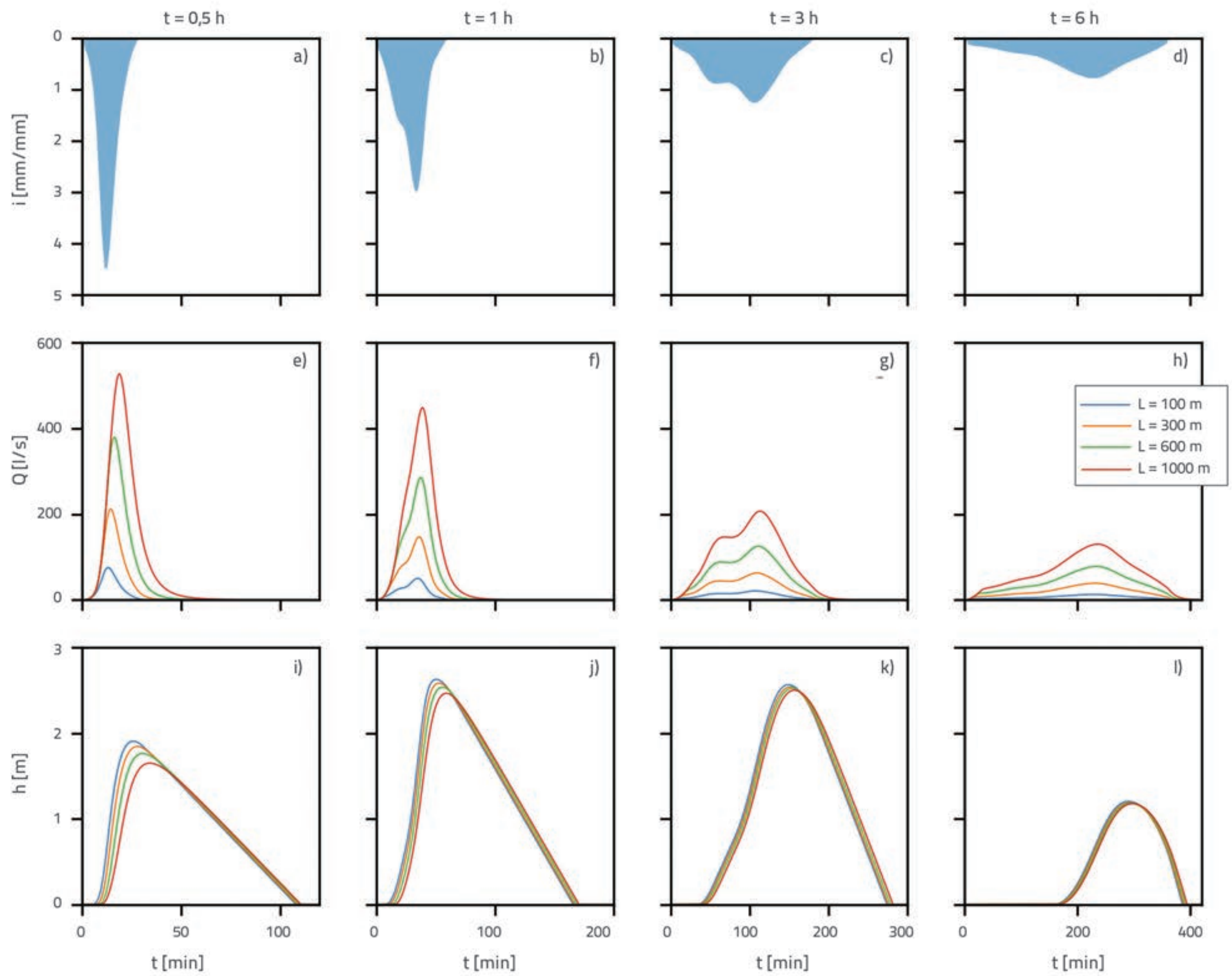

Figure 7. Effect of watershed length on: a) – d) temporal distribution of rainfall intensity, e) – h) runoff discharge, and i) – l) water level changes in infiltration tank, for various storm durations

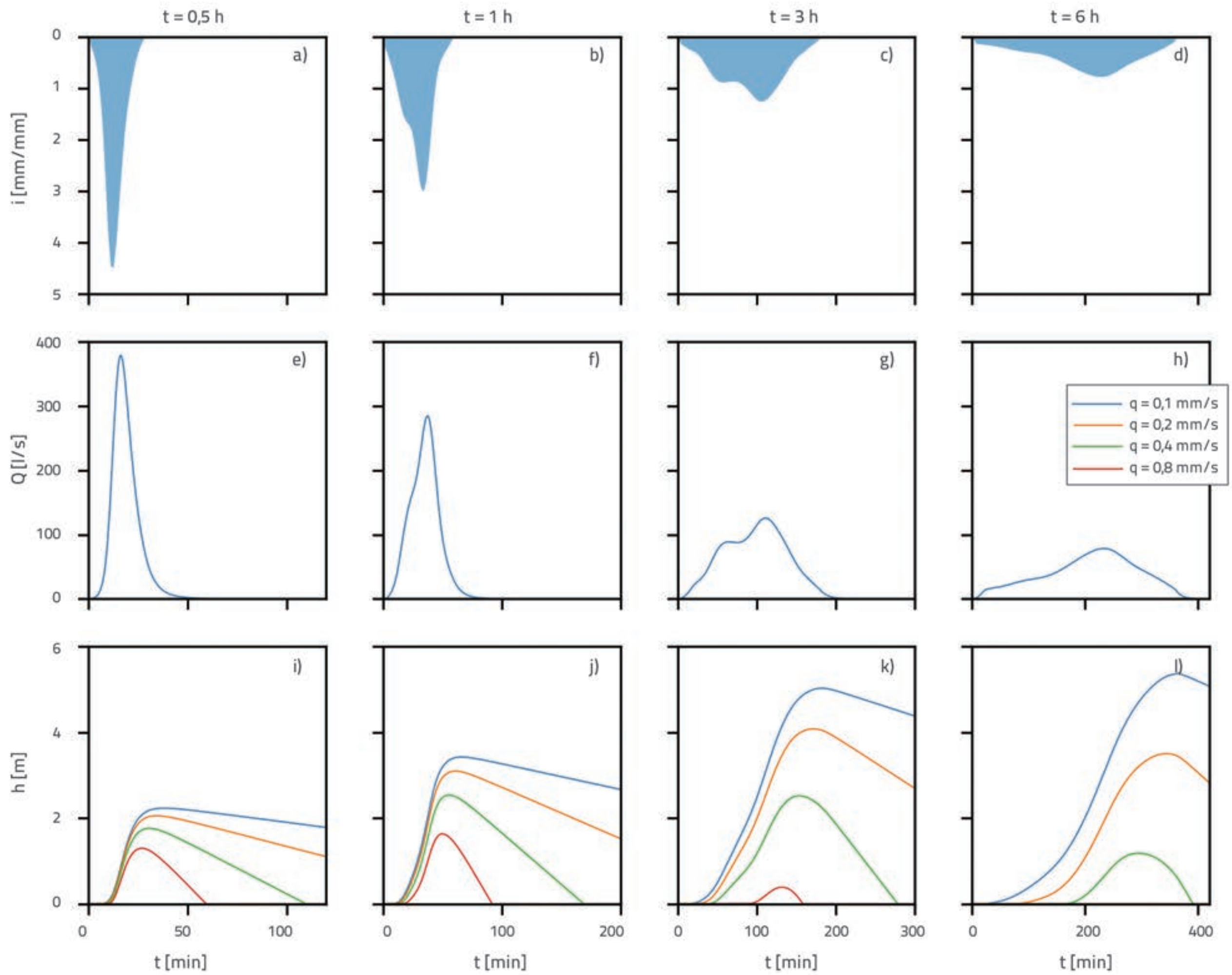

Figure 8. Effect of infiltration velocity on: a) – d) temporal distribution of rainfall intensity, e) – h) runoff discharge, and i) – l) water level changes in infiltration tank, for various storm durations

The effects of varying the watershed area were analysed in the first test. The same parameters as in the previous example were used to compute the runoff hydrograph. Only the watershed lengths were varied L = 100, 300, 600, and 1000 m. The infiltration tank bottom area was once again defined as 2% of the watershed area, which amounts to A_b = 20, 60, 120, and 200 m^2, respectively. The overflow height was $H_{overflow}$ = 3.0 m, and the average infiltration velocity was q_{inf} = 0.4 mm/s.

Figure 7 shows the runoff hydrograph and water level changes in the infiltration tank for various watershed areas and storm durations. A clear dependence of the peak runoff discharge on the watershed area can be observed (Figures 7e. - 7.h). However, although discharges and total runoff volumes vary with watershed area, total water levels in the infiltration tank are almost constant when the tank area is sized proportionally to the watershed area (Figure 7.i - 7.l). Furthermore, the critical storm duration ranges from 1 to 3 hours, regardless of the watershed area.

The effects of infiltration velocity were analysed in the second test. The same parameters as in the previous test were used for watershed characteristics and infiltration tank. Only the infiltration velocities were varied q_{inf} = 0.1, 0.2, 0.4, and 0.8 mm/s.

Figure 8 shows the runoff hydrograph and water level changes in the infiltration tank for various infiltration velocities. It can be noted that the infiltration velocity has a significant effect on water level changes in the tank (Figures 8i-l). It seems that the corresponding water levels will be lower for higher infiltration velocities, regardless of storm duration. Additionally, the critical storm duration depends on the infiltration velocity; for a low infiltration q_{inf} = 0.1 mm/s the critical storm duration is the longest and amounts to 6 hours (Figure 8l), whereas for a high infiltration q_{inf} = 0.8 mm/s it is the shortest and amounts to 1 hour (Figure 8j). Clearly, the process of defining the critical storm duration in advance is not straightforward; however, a strong trend has been observed suggesting that low infiltration velocities correspond to longer critical storm durations. Furthermore, these results emphasize the importance of conducting a series of computations on the infiltration tank storage volume for different storm durations in order to define a critical scenario.

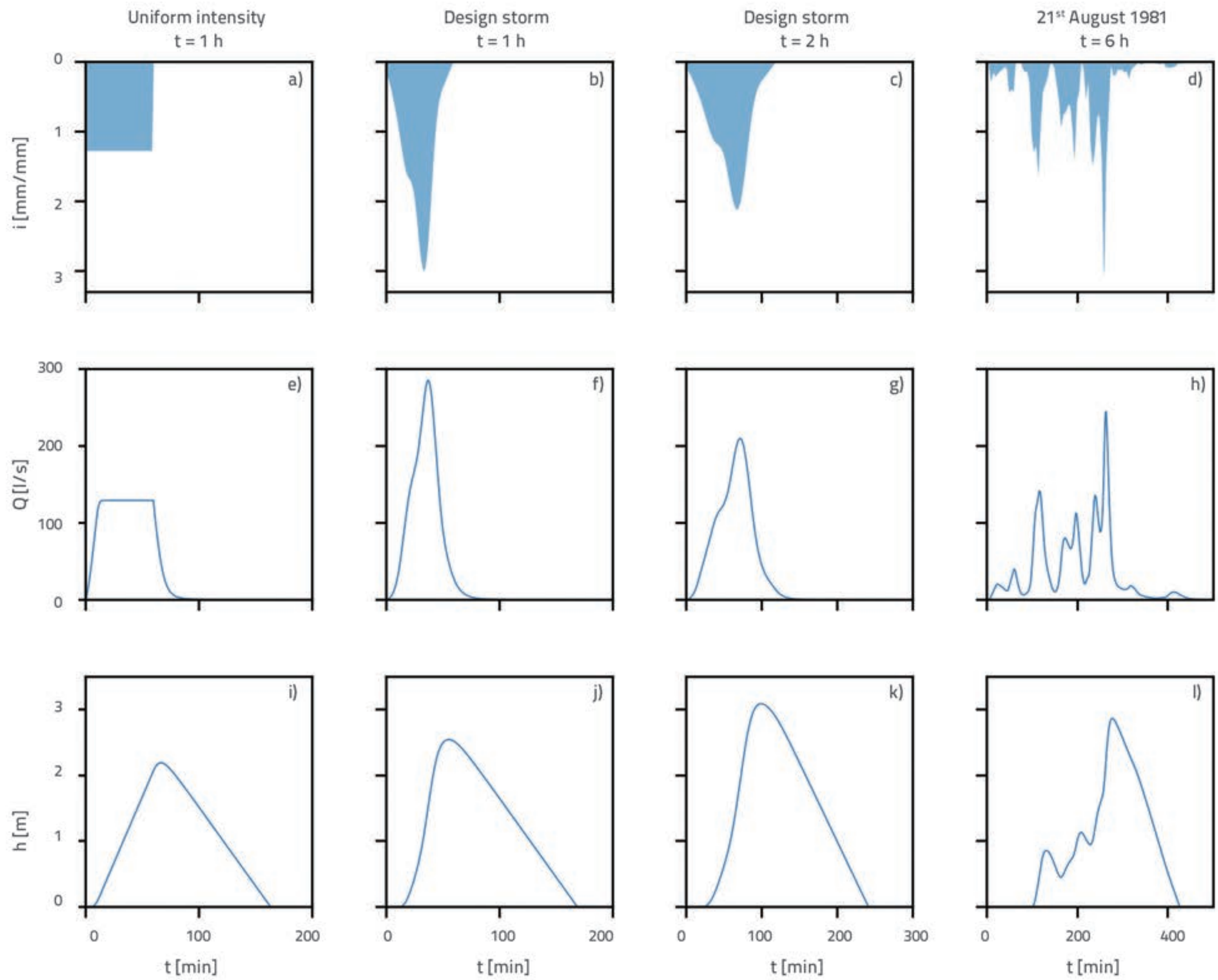

Figure 9. Design of infiltration tank storage dimensions according to three different approaches: a) 1-hour uniform rainfall intensity, d) 1-hour design storm, c) design storm with a critical duration of 2 hours, and d) 7.5-hour long rainfall observed in August 1981, with corresponding runoff hydrograph (e-h) and water level changes in infiltration tank (i-l)

3.4 Example of sizing infiltration tank and evaluating extreme rainfall event from 1981

Water level changes in the infiltration tank were also analysed for a heavy storm event that occurred in August 1981, when 173 mm of rain fell in seven and a half hours, which corresponds to a 30-year return period event for Rijeka. Furthermore, statistical indicators for time intervals shorter than 2 hours suggest a 10-year to 20-year return period. Therefore, this storm event should not cause any overflows of an infiltration tank that is designed for a 20-year return period, although this was one of the most extreme events in the 1961 – 1990 period.

The parameters similar to those used in previous examples were used for computing the surface runoff hydrograph. Since in Croatian engineering practice, the infiltration systems - among which the most common ones are infiltration tanks - are regularly designed for a 1-hour rainfall, the design of infiltration tank dimensions is conducted in this example according to three different approaches: a) uniform 1-hour rainfall intensity, b) 1-hour design storm, and c) design storm of critical duration selected from a series of storm durations ranging from 1 to 6 hours. Note that only the depth of the infiltration tank storage area, i.e. the overflow height, was optimized, whereas the tank bottom area accounted for constant 2% of the watershed area A_b = 120 m^2, and the infiltration velocity was q_{inf} = 0.4 mm/s.

The surface runoff hydrograph, and the water level changes in the infiltration tank, are shown in Figure 9 for the four analysed scenarios. The maximum computed total water depths amount to h = 2.18 m for the uniform 1-hour rainfall intensity (Figure 9.i), h = 2.54 m for the 1-hour design storm (Figure 9.j), h = 3.08 m for the design storm with a critical duration of 2 hours (Figure 9.k), and finally h = 2.96 m for the 7.5-hour long rainfall observed in August 1981 (Figure 9.l). Note that although the maximum runoff discharge was the result of a 1-hour rainfall (Figure 9.f), the maximum water level depth was registered for the critical duration of 2 hours (Figure 9.k).

The functionality of different-size infiltration tanks under the inflow generated by the extreme 1981 rainfall event was

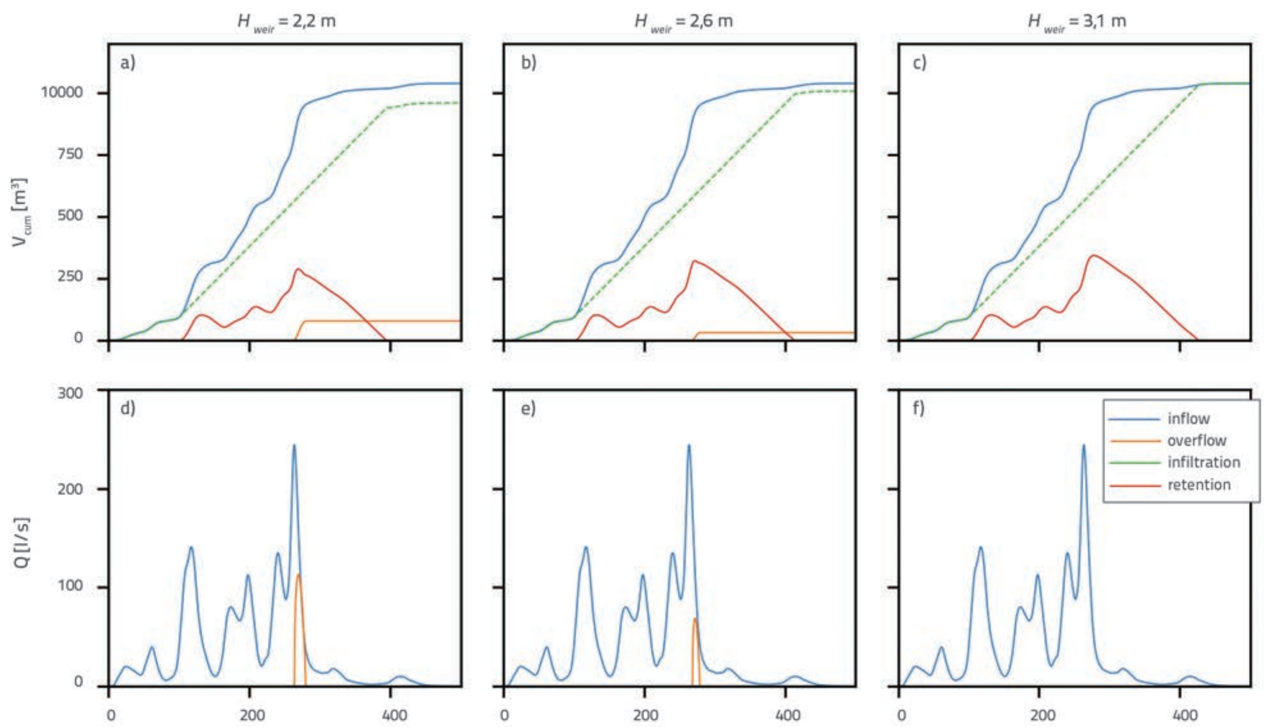

Figure 10. Water level changes in infiltration tank during an extreme event from August 1981, when tank storage dimensions were designed for: a) 1-hour uniform rainfall intensity ($H_{overflow}$ = 2.2 m), b) 1-hour design storm ($H_{overflow}$ = 2.6 m), and c) design storm of critical duration ($H_{overflow}$ = 3.1 m), and resulting inflow and outflow changes (d - f)

analysed in the final step. The overflow heights were designed according to the previously computed maximum water level depths in the infiltration tank, i.e., the overflow heights were $H_{overflow}$ = 2.2, 2.6, and 3.1 m, with three overflow 200 mm long pipes to keep the overflow head H_p under 0.3 m. The infiltration tank storage volume was determined based on the selected overflow heights, and an additional 0.5 m of safety height, which resulted in the tank volumes of V = 324, 372 and 432 m^3. The stormwater volume changes and the infiltration tank inflow and outflow rates are shown in Figure 10. Results show that when the tank is designed to a uniform 1-hour rainfall, a total 78.7 m^3 of stormwater is overflown with the maximum discharge of 113.0 l/s (Figures 10.a - 10.d), which is more than 50% over the peak inflow discharge. The tank also overflows when the 1-hour design storm is used, albeit to a smaller extent, i.e. the total of 31.1 m^3 of stormwater is overflown with the maximum discharge of 68.8 l/s (Figures 10.b - 10.e). Finally, when a critical 2-hour design storm is used, there is no overflow of the infiltration tank (Figure 10.c - 10.f).

4. Conclusion

This paper emphasizes the importance of an optimum design of infiltration systems as key elements in the LID approach to the stormwater management and storm drainage design. Although the LID approach is widely used in various areas around the world, including Croatia, there is still no common and unified methodology for the design of infiltration systems and selection of an appropriate meteorological data to be used for this purpose. Previous conceptual approaches to the storm drainage design, based on an immediate evacuation of the stormwater through sewer systems, proved to be of limited value and ineffective. Meteorological data used in these outdated approaches (HDF and IDF curves) were shown to be unsuitable for the design of infiltration systems because, in the modern approaches (LID), it is important to understand and quantify the entire dynamics of the runoff process and stormwater volume, in addition to the peak surface runoff.

Therefore, a contribution was made in this paper to overcome the current weaknesses by analysing and discussing several significant elements in hydrological computations according to the modern stormwater-management approach, including the shape of the design storm, the critical storm duration, and the runoff process, as well as the infiltration system geometry and terrain characteristics defined by infiltration velocity. A temporal distribution of the design storm for Rijeka was also analysed, based on a 10-year sample from the 1961-1990 period. For this purpose, an appropriate methodology was proposed for defining the design storm based on the average variability method, with additional smoothing by means of Bezier curves.

The results revealed that a critical storm duration can not be estimated in advance for sizing the infiltration tank storage volume. Instead, a series of different storm durations defined for a particular site must first be analysed. From this series of computations, a critical scenario should be chosen, which is defined by a critical value or, in this case, by the largest required infiltration storage volume.

The effects of infiltration velocity on the design of infiltration systems are also examined in the paper. This is usually the least reliable parameter, especially in heterogenic karst terrains, where scarce field tests and measurements are available. However, this parameter may significantly affect the infiltration tank dimensions. The analyses conducted in this paper suggest that lower infiltration velocities correspond to longer storm durations.

The analyses and the corresponding conclusions presented in this paper do not intend to be complete and final guidelines, neither for defining the appropriate design storm curve (especially not for the Rijeka area, for which the analysis was based on 10 years of data only), nor for the design of infiltration systems. With further development of modern LID approaches to stormwater management, it is imperative to improve not only practical implementation of modern drainage system elements, but also their optimum design, for which the present paper may constitute an appropriate methodological basis.

Acknowledgement

This paper has been fully supported by the University of Rijeka under the project number 17.06.2.1.02 (River-sea interaction in the context of climate changes).

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