

RAINFALL–RUNOFF SIMULATION FOR DESIGN FLOOD ESTIMATION IN SMALL RIVER CATCHMENTS

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Abstract. *This paper presents development and application of the model aimed at simulating peak flood runoff from the small river basin Obnica in Serbia (having an area of 185 km²) with an aim to estimate design floods using different approaches. The model is developed using the HEC-HMS software (The United States Army Corps of Engineers (USACE) Hydrologic Centre's Hydrologic Modelling System). The model is calibrated against eight events with observed hydrographs and corresponding rainfall, and verified with a separate set of events. Flood hydrographs are simulated with the constant intensity design storms of various durations and with the 24-hour design storm with design hyetograph determined using the alternating block method. All design floods obtained from the simulated hydrograph peaks are compared with the design floods estimated by statistical analysis of annual maximum flows. The results have shown that the temporally distributed 24-hour storms yield the design floods that are the closest to the statistically derived design flows, while the constant intensity storms cannot reproduce the statistically derived design flows.*

Key words: *design floods, design storms, rainfall-runoff, simulation, HEC-HMS, flood hydrograph*

1. INTRODUCTION

Flood runoff assessment procedure depends on whether hydrological measurements are available on a given watercourse and if they have required scope and quality. Availability of hydrological data of satisfactory quality usually leads to the flood assessment procedure based on the statistical analysis that includes choice of the probability distribution for the observed floods, estimation of the distribution parameters and calculation of the design flood hydrograph characteristics (usually the peak flow) of a certain return period. If the flood

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measurements are not available, the flood assessment relies on the methods which are based on the rainfall-runoff transformation for the ungauged catchments.

For the catchments where rainfall and runoff data are available, a rainfall-runoff model may be developed. Once calibrated, such a model can be used to estimate design floods from either design or observed rainfall storms. There are several software packages that allow modelling of the rainfall-runoff process, and one of them is the HEC-HMS (Hydrologic Centre's Hydrologic Modelling System) by US Army Corps of Engineers [14]. The HEC-HMS model can be used for either event based or continuous modelling of the rainfall-runoff process and consists of a variety of methods for assessing various processes in a catchment (interception, surface depression storage, infiltration losses, runoff transformation, baseflow and hydrograph propagation). The flood hydrograph assessment is usually based on the event modelling, when the processes such as interception and evapotranspiration can be neglected in comparison to flood runoff during a relatively short simulation period covering rainfall and runoff duration. Moreover, the baseflow characterization is usually not important for the purpose of defining the design flood flows and therefore there is no need to employ the model in a continuous mode in order to compute baseflow.

Developing a rainfall-runoff model for the ungauged catchments is impossible and the usual procedure for determining the design relies on the regional relationships between the flood flows and physiographic catchment characteristics (e.g. catchment size, stream length slope, etc). Typically, synthetic unit hydrographs are developed for the ungauged catchments based on their physiographic characteristics, which are used in conjunction with the synthetic design storms to obtain the design flood hydrographs. Since the ungauged catchments are usually small, a question of the most critical design storm duration is usually resolved by trial and the storm duration producing the greatest peak flood flow is then adopted for further analysis. However, such a customary engineering practice sometimes overlooks some basic assumptions underlying the engineering methods applied in this procedure.

In this paper, the goal is to evaluate the typical procedures for design flood assessment at ungauged catchments in Serbia by comparing the traditional design flood estimates with alternative ones. To this end, in this paper a gauged small catchment (the Obnica River basin) in Serbia is selected, which is treated as an ungauged catchment and for which the design floods are estimated based on the design storms and synthetic unit hydrographs. Instead of developing a synthetic unit hydrograph based on the catchment's physiographic characteristics, a rainfall-runoff model is developed for the selected catchment in HEC-HMS based on the available observed events that served for model calibration and verification. This model was applied with the same design storms to obtain the design floods under different assumptions about the most critical design storms. Also, the design flood flows at the catchment outlet that are estimated using statistical analysis of the observed annual maximum flows provide a ground for verification of the other methods of flood flow estimation.

2. MODELLING FLOOD HYDROGRAPHS IN HEC-HMS

2.1. Model description

The way of simulating the runoff process with a mathematical model depends on the problem we need to solve. HEC-HMS schematizes the hydrological cycle as shown in Fig. 1 in a way that is suitable for most problems [8]. Three main components of the model should

be defined first: basin model, meteorological model and control specifications. The basin model represents the catchment and the methods of computing different catchment processes, meteorological model describes the precipitation input and the methods for calculating snow melt and evapotranspiration, and the control specifications define the modelling time frame and time step.

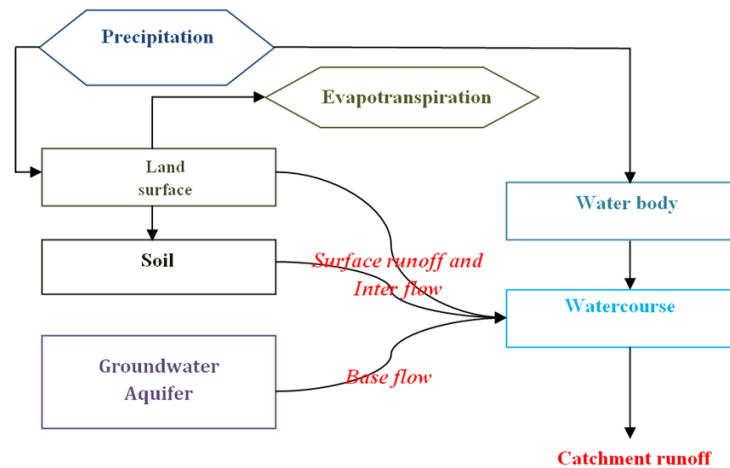


Fig. 1 The hydrologic cycle in HEC-HMS software

In HEC-HMS, a basin model is comprised of a number of elements such as sub-basins, river reaches, junctions, retention, springs, estuaries, reservoirs and diversions (relief) [13] [14] [17]. A sub-basin is the element where the methods for the runoff modelling are defined. For the event modelling, the following methods are important: the loss method, the runoff transform method and the baseflow method. Other methods are related to interception and surface depression storage. River reaches are the elements in which hydrograph routing is performed, and is necessary when a basin needs to be decomposed into sub-basins to route hydrographs from the sub-basins to the basin outlet.

There are seven methods for modelling losses in HEC-HMS: deficit and constant, initial and constant, Green-Ampt, SCS-CN, soil moisture accounting (SMA) method, exponential losses, Smith-Parlange method.

The transformation of excess rainfall into runoff can be obtained by using one of the seven methods: Clark's unit hydrograph, kinematic wave, modified Clark's method, SCS unit hydrograph, Snyder's unit hydrograph, user specified S-graph and a user specified unit hydrograph.

The base flow can be modelled by the following methods: bounded recession, constant monthly, linear reservoir, the Boussinesq equation (non-linear) and the recession method.

Several methods are available for the hydrograph routing, including the simple lag method, the modified pulse method, the Muskingum method, but also the kinematic wave and the Muskingum-Cunge methods for detailed hydraulic computations.

The meteorological model defines type of the rainfall input, for which a range of options are available. The simplest method is the user specified hyetograph. The SCS daily storm distribution is also available for a specified total rainfall depth. Other two

methods (frequency storm and the standard design storm) are also available as the design storm methods. There are three methods for calculating rainfall over the catchment from multiple gauges: gage weights, inverse distance, and gridded rainfall.

2.2. Direct runoff modelling: Snyder's synthetic unit hydrograph

In order to estimate direct runoff from small ungauged catchments or urban areas, the synthetic unit hydrograph (SUH) concept is commonly used. In this study, direct runoff simulation in HEC-HMS is performed with Snyder's SUH.

In 1938 Snyder, according to [1], was the first who developed method for SUH determination. Construction of such a UH involves estimating a number of parameters including basin lag time, hydrograph time base, storm duration, peak flow, hydrograph widths at discharges equal to 50% and 75% of the peak discharge.

The basin lag time t_p is defined as time from the centroid of excess rainfall hyetograph to the maximum ordinate of direct runoff hydrograph [10]. Snyder introduced the standard unit hydrograph on the basis of data from catchments in the north-eastern USA with areas ranging from 30 to 30000 km². The standard synthetic unit hydrograph has a ratio 5.5 between the basin lag time and "standard" excess rainfall duration t_k [7]:

$$t_p = 5.5 \cdot t_k \quad (1)$$

Based on the basin geomorphologic characteristics, Snyder came to the following expression for the basin lag in hours:

$$t_p = 0.75 \cdot C_t \cdot (L \cdot L_c)^{0.3} \quad (2)$$

where C_t is the coefficient obtained from regional analysis of the selected unit hydrographs and usually ranges from 1.8 to 2.2, L is the main stream length from the catchment divide to the outlet (in km), and L_c is the distance from the outlet to the point on the main stream which is closest to the catchment centroid (in km). The "standard" excess rainfall duration t_k is therefore a duration that satisfies equation (1).

The peak flow of the standard unit hydrograph is computed using the following relation:

$$Q_p = \frac{2.75 C_p A}{t_p} \quad (3)$$

where C_p is the peak flow coefficient (higher C_p values are associated with lower values of C_t), and A is the catchment area in km².

Once the elements of the standard unit hydrograph are computed, the unit hydrograph for another required rainfall duration t_k' can be derived from it. The basin lag time t_p' for the required duration t_k' is modified according to:

$$t_p' = t_p + 0.25(t_k' - t_k) \quad (4)$$

where all lags and durations are in hours. The peak flow of the required unit hydrograph is then computed from the relationship:

$$Q'_p = \frac{Q_p t_p}{t'_p} \quad (5)$$

Since the area under the unit hydrograph is equal to the runoff volume from the unit excess rainfall, the hydrograph time base T_b is obtained by assuming the triangular shape of the unit hydrograph.

In HEC-HMS, application of the Snyder's SUH method requires specifying two parameters: the basin lag t_p for the standard unit hydrograph and the coefficient C_p in equation (3). HEC-HMS then estimates the required unit hydrograph by setting rainfall duration equal to the computational time step, and computing the basin lag and peak unit flow from equations (4) and (5). However, HEC-HMS then estimates the hydrograph time base and the ordinates by finding a Clark's SUH with the equivalent peak flow and time to peak [14].

2.3. Base flow modelling

In this study, the recession method in HEC-HMS is used to model base flow. The recession method assumes the exponential decay of base flow Q_b from an initial value InQ_b [14]:

$$Q_b = InQ_b \cdot k^t \quad (6)$$

where t is time elapsed from occurrence of InQ_b , and k is the recession constant. The initial condition can be specified as initial flow value or as initial flow value per unit catchment area.

The recession model (6) is applied at the start of runoff simulation to simulate recession limb of previous flood event, and after the direct runoff peak time when the direct runoff decreases to the threshold value specified by the user. At the time of occurrence of the threshold value, total flow is defined by (6). The threshold value can be specified as the flow rate or the ratio of the flow to the computed peak flow (R).

2.4. Excess rainfall

The SCS method for modelling excess rainfall is widespread, simple and often used for assessment of direct runoff at ungauged catchments. The Curve Number (CN) parameter of this method varies in a wide range for different soil types and land use and can be found in comprehensive tables developed by former Soil Conservation Service (SCS), now National Resources Conservation Service of the US Department of Agriculture [16].

The excess rainfall is obtained from the following relationship:

$$Q = \frac{(P - I_a)^2}{P - I_a + S} \quad \text{for } P > I_a \quad (7)$$

$$Q = 0 \quad \text{for } P \leq I_a$$

where P is cumulative rainfall depth (mm) at given time step, I_a is initial abstraction (mm), Q is cumulative excess rainfall at given time step (mm), and S is potential maximum soil retention (mm). Based on the assumption that the initial abstraction is a fraction of the potential maximum retention:

$$I_a = \lambda S \quad (8)$$

equation (7) becomes:

$$Q = \frac{(P - \lambda S)^2}{P + (1 - \lambda)S} \quad (9)$$

The potential retention S in mm is expressed in terms of the curve number CN :

$$S = \left(\frac{1000}{CN} - 10 \right) \cdot 25.4 \quad (10)$$

The curve number CN ranges from 0 when $S \rightarrow \infty$, to 100 when $S = 0$.

When rainfall-runoff data are available, they can be directly used to estimate the potential retention S [16]:

$$S = \frac{P}{\lambda} + \frac{(1 - \lambda)Q - \sqrt{(1 - \lambda)^2 Q^2 + 4\lambda PQ}}{2\lambda^2} \quad (11)$$

According to equations (10) and (11), the event value of CN can be obtained from:

$$CN = \frac{25400}{\frac{P}{\lambda} + \frac{(1 - \lambda)Q - \sqrt{(1 - \lambda)^2 Q^2 + 4\lambda PQ}}{2\lambda^2} + 254} \quad (12)$$

2.5. Model calibration and verification

Adjusting model parameters so that the simulated hydrograph matches the observed hydrograph as closely as possible is called model calibration. This process is an optimization process with an objective function that measures the degree of agreement between the computed and the observed hydrograph. The process effectively searches for the minimum value of the objective function. Usually, model parameters are estimated through calibration if they cannot be obtained by observation and measurement, or they do not have physical meaning.

Calibration can be performed manually or automatically. Manual calibration employs the knowledge on the physical characteristics of the catchment and modeller's experience, while in the automatic iterative estimation procedure the parameters are adjusted until the minimum of the selected objective function is reached. HEC-HMS version 4.0 features an optimization manager that enables automatic calibration. There are five possible objective functions [4]: peak-weighted root mean square error (PWRMSE), sum of squared residuals (SSR), sum of absolute residuals (SAR), percent error in peak flow (PEPF) and percent error in volume (PEV).

In the flood runoff hydrograph modelling, PWRMSE is an appropriate objective function, with a weighting factor that gives greater weight to the errors around the value of the hydrograph peak:

$$PWRMSE = \sqrt{\frac{\sum_{t=1}^N (O(t) - M(t))^2 \frac{O(t) - \bar{O}}{2\bar{O}}}{N}}; \quad (13)$$

$$\bar{O} = \frac{1}{N} \sum_{t=1}^N O(t)$$

where $O(t)$ and $M(t)$ are the observed and the modelled flow at time t , and \bar{O} is the average observed flow.

To minimize the objective function, HEC-HMS uses two search methods [4]:

- the univariate gradient (UG) method, which optimizes only one parameter while the others remain constant;
- the Nelder and Mead (NM) method, which is based on the simultaneous optimization of all parameters (SIMPLEX method).

The initial parameter values are set at the beginning of the optimization procedure. HEC-HMS has built-in parameter constraints that have physical meaning. Fine bounds can be defined by user in order to achieve greater efficiency.

Model performance can be evaluated by different model efficiency measures. In addition to PWRMSE as an objective function given by equation (13), some of these measures are described below.

The percent error in peak flow (PEPF) considers only the peak value and disregards the hydrograph volume and time to peak:

$$PEPF = 100 \left| \frac{O(peak) - M(peak)}{O(peak)} \right| \quad (14)$$

where $O(peak)$ and $M(peak)$ are the observed and the modelled peak values, respectively.

The correlation coefficient between modelled and measured flows (CORR) is obtained using the expression:

$$CORR = \frac{\sum_{t=1}^N (O_t - \bar{O}) \times (M_t - \bar{M})}{\sqrt{\left[\sum_{t=1}^N (O_t - \bar{O})^2 \times \sum_{t=1}^N (M_t - \bar{M})^2 \right]}} \quad (15)$$

where O_t and M_t are the observed and the modelled flow at time t , \bar{O} and \bar{M} are corresponding means over the calibration period, and N is the number of data.

The root mean standard error is obtained as:

$$RRMSE = 100 \times \sqrt{\frac{1}{N} \sum_{t=1}^N \left(\frac{M_t - O_t}{O_t} \right)^2} \quad (16)$$

The Nash-Sutcliff efficiency E is defined by [5]:

$$E = 1 - \frac{\sum_{t=1}^N (O_t - M_t)^2}{\sum_{t=1}^N (O_t - \bar{O})^2} \quad (17)$$

Model verification is the process of testing the model on the basis of data that were not used for calibration. During the verification process, the parameter values are not changed. Model performance is evaluated again in the verification process to check the adopted parameter values. If the model is well calibrated, its performance with the verification data should be similar to that with the calibration data.

3. DESIGN STORMS

Design storm that produces maximum runoff on a small ungauged rural or urban basin, where time of concentration ranges from several minutes to several hours, is usually selected from the depth-duration-frequency (DDF) relationships. The greatest problem in estimating rainfall of duration shorter than one day in Serbia is lack of representative DDF relationships for a given catchment due to sparse recording raingauge network. The hydrologists are therefore forced to transpose the design storms from the nearest recording gauges to the location of the study by using data on daily rainfall from the non-recording gauges within the basin or close to it. The transposition is made by assuming that the ratio of the short duration rainfall to the daily rainfall at the gauge location is also valid for the studied basin. The short duration rainfall for the basin is therefore calculated as:

$$P_{BASIN}(t_k, T) = \frac{P_{GAUGE}(t_k, T)}{P_{GAUGE,daily}(T)} P_{BASIN,daily}(T) \quad (18)$$

where t_k denotes storm duration and T denotes return period.

Daily design rainfall for the basin is obtained by finding the areal average from the design daily rainfall at the non-recording gauges. In this study we applied the Thiessen polygon method as a frequently applied method for averaging rainfall. The maximum daily rainfall depth for the return period T is then [9]:

$$P_{BASIN,daily}(T) = \sum_{i=1}^n \frac{a_i \cdot P_i(T)}{A} \quad (19)$$

where a_i is the Thiessen polygon area for i^{th} gauge (km^2), $P_i(T)$ is the maximum daily rainfall for return period T at i^{th} gauge (mm), A is the catchment area (km^2), and n is the number of considered non-recording gauges.

The last step in defining design storms is to develop the design hyetograph. The short duration rainfall is generally not distributed uniformly in time and the shape of the runoff hydrograph is strongly affected by the hyetograph shape. In typical engineering applications in Serbia, temporal distribution of design storms is not considered and constant intensity hyetograph shape is usually adopted (so called block storm method). Temporal distribution

of design storms can be developed either statistically or synthetically. In this study, two types of design storms are used as the input to the rainfall-runoff model:

- design storms of constant intensity with different durations (block storms), and
- 24-hour design storms with temporally varying intensity.

The synthetic design rainfall hyetographs for the 24-hour storms are obtained by the alternating block method [3], following the steps below:

- a. For the design storm of return period T and duration t_k , design rainfall depths for all durations in Δt increments are taken from the DDF curve for the same return period;
- b. Incremental rainfall depths are calculated from the design rainfall depths for all durations in Δt increments;
- c. The highest incremental value is placed in the middle of hyetograph. The second highest block is placed to the right of the maximum block and the third highest to the left. The fourth highest rainfall increment is placed to the right of the maximum block after second block and so on until the last one.

4. DATA AND RESULTS

4.1. Basin model

The rainfall-runoff model for the Obnica River basin, shown in Fig. 2, is developed in HEC-HMS software as the lumped model. The basin is a typical mountainous basin with an area of 185 km². The flow is measured at hydrological station Belo Polje at the catchment outlet. As described in section 2, Snyder's SUH is selected for direct runoff computation and rainfall excess is assessed applying the SCS-CN method. Base flow is modelled by the recession method.



Fig. 2 The Obnica River basin with hydrological station (blue triangle), rain gauges (violet dots) and Thiessen polygons for the rain gauges

Geomorphologic characteristics of the catchment needed for the basin lag time estimation are shown in Table 1, and were determined using GIS software on the basis of digitized 1: 25,000 topographic maps.

For the rainfall excess assessment, initial value of CN is estimated using Corine Land Cover database [2], taking into account runoff conditions (land use and soil hydrogeological characteristics) and literature [9].

Table 1 Geomorphologic characteristics of Obnica River Basin

River	Obnica
Station	Belo Polje
Catchment area (km ²)	185
Catchment length (km)	28.1
Weighted channel slope (%)	1.11
Absolute channel slope (%)	2.61
Average catchment slope (%)	18.26
Distance to centre of gravity (km)	14.5
Average catchment elevation (m.a.s.l.)	401

4.2. Model calibration and verification

The model calibration was performed using eight recorded hydrographs and the corresponding rainfall events in the Obnica River basin [9] using the PWRMSE objective function as the criterion. Computational time step was 30 minutes in accordance with the available rainfall and flow data. Table 2 provides an overview of the parameters of the selected modelling methods and explains how they are calibrated. Calibration was performed with both search methods described in section 2.5, namely one parameter optimization (UG) and the simultaneous optimization of all parameters (NM).

Table 2 Parameters subject to calibration

Modelling method	Parameter	Comment
Loss method:	Curve number CN	Initial value based on Corine Land Cover [2]
SCS-CN	Initial abstraction I_a	
Transform method:	Standard basin lag t_p	Calibration of parameter C_t in Eq. (2)
Snyder's UH	Peaking coefficient C_p	Calibration of parameter C_p in Eq. (3)
Baseflow method:	Initial baseflow InQ_b	Manually calibrated, initial values are adopted
Recession	Inflection point to peak ratio R	as described in section 2.3
	Recession constant k	

The calibration results with two optimization methods showed similar correlation coefficients between the observed and simulated hydrograph characteristics. The parameter values obtained by the UG optimization method were adopted because the values of CN and initial abstraction vary in a narrow range across the calibration events (Table 3). The correlation coefficient between the observed and modelled hydrograph volumes is $CORR = 0.987$, while the correlation coefficient between the observed and modelled flow peaks is $CORR = 0.947$. Base flow parameters were calibrated manually according to minimum difference between runoff volumes of the measured and modelled

hydrograph. Calibration results are given in Table 3. Adopted values of the parameters are shown in the last row of Table 3.

Table 3 Values of calibrated model parameters and calibration efficiencies

Event No.	Parameter							Efficiency		
	SCS CN		Snyder's UH		Recession			E	RRMSE %	PEPF %
	I_a mm	CN	t_p h	C_p	ImQ_b m ³ /s	R	k			
103	7.16	89.3	9.22	0.75	4.97	0.484	0.485	0.985	1.0	0.8
95	7.14	93.2	12.56	0.99	20.01	0.928	0.073	0.745	10.3	6.6
79	7.69	77.9	4.78	0.48	0.23	0.025	1.000	0.845	1.3	0.7
71	7.63	87.0	10.40	0.73	1.50	0.161	0.791	0.927	2.0	1.2
67a	11.42	98.9	10.63	0.71	1.02	0.188	0.861	0.935	1.8	1.1
63	7.70	83.7	10.96	0.47	0.98	0.161	1.000	0.912	1.2	0.9
60	7.64	83.2	10.74	1.00	1.57	0.168	0.785	0.921	4.1	2.8
59	7.01	88.4	12.31	0.83	2.27	0.336	0.747	0.785	2.0	1.0
Adop.	7.9	87.7	10.2	0.70	1.88	0.246	0.770			

The model was verified with a separate set of events. The results of the verification are given in Table 4 by comparing the observed and simulated peak flows, times of peak and total runoff volumes. The model performance indicators shown in Table 4 justify the parameter values adopted in the calibration process.

Table 4 Model verification results (peak flow Q , time of peak t , runoff volume V ; subscript “o” denotes observed values and “s” simulated values) and corresponding efficiencies

Event No.	Q_o m ³ /s	Q_s m ³ /s	t_o h	t_s h	V_o mm	V_s mm	E	RRMSE %	PEPF %
70	29.10	15.00	06/06/1980 00:00	05/06/1980 23:30	9.64	6.31	0.447	-3.3	3.4
75	18.10	19.50	18/06/1981 05:30	18/06/1981 06:00	9.31	12.98	0.201	3.3	1.6
86	113.90	67.70	18/04/1985 02:00	18/04/1985 02:30	79.09	57.49	0.763	15.2	8.7
94a	61.10	41.10	21/05/1987 14:30	21/05/1987 14:30	44.88	35.32	0.791	5.7	3.0
100	34.70	38.80	17/06/1989 06:00	17/06/1989 23:30	30.35	35.53	0.758	4.4	3.0

4.3. Development of design storms

Maximum daily rainfall for the Obnica River basin was determined by the Thiessen polygons method based on the statistical analysis of maximum daily rainfall at five rain gages: Majinović, Počuta, Pecka, Osečina and Valjevo. Processing period was 1946-2006 [5] [11].

Short duration rainfall depths and intensities (DDF and IDF curves) were available for the Valjevo rain gage station. The study included the annual maximum intensities for the following durations: 10, 20, 30, 60, 120, 180, 360, 720 and 1440 minutes. Design rainfall depths of different durations for the Obnica basin were calculated using the DDF curves for the Valjevo station multiplied by the ratio of maximum daily rainfall depths for the basin and at Valjevo [11]. The resulting DDF curves ordinates for exceedance probabilities 1%, 2% and 5% are shown in Table 5. Last two rows in Table 5 show maximum daily rainfall for the basin and at the Valjevo station that were used for calculations.

Five meteorological models were made in HEC-HMS depending on design rainfall duration. For the durations of 3, 6, 9 and 12 hours constant intensity rainfall was assumed (block storms), while a synthetic hyetograph was developed by the alternating blocks method for the 24-hour rainfall duration.

Table 5 Design rainfall P of different exceedance probabilities p for durations t_k for the Obnica River Basin at h.s. Belo Polje

t_k (min)	P (mm)		
	p=1%	p=2%	p=5%
10	23.11	21.68	19.49
20	31.57	29.57	26.53
30	39.07	36.57	32.72
60	55.00	51.38	45.75
120	72.67	67.82	60.20
180	81.52	76.08	67.49
360	89.18	83.27	74.02
720	90.51	84.65	75.58
1440	90.69	84.99	76.38
max Pdaily Obnica	86.80	80.60	71.80
max Pdaily Valjevo	97.90	88.20	75.80

4.4. Results of model simulations with design storms

The comparative results of the runoff hydrograph peak flows simulated by the rainfall-runoff model using HEC-HMS and the design flows obtained by statistical analysis of annual maximum flows of the Obnica River at Belo Polje hydrologic station for 1954-2006 [5] are shown in Fig. 3. Table 7 also shows the values of simulated peak flows with exceedance probability of 1%, 2% and 5%, and the corresponding characteristic times of the synthetic unit hydrographs.

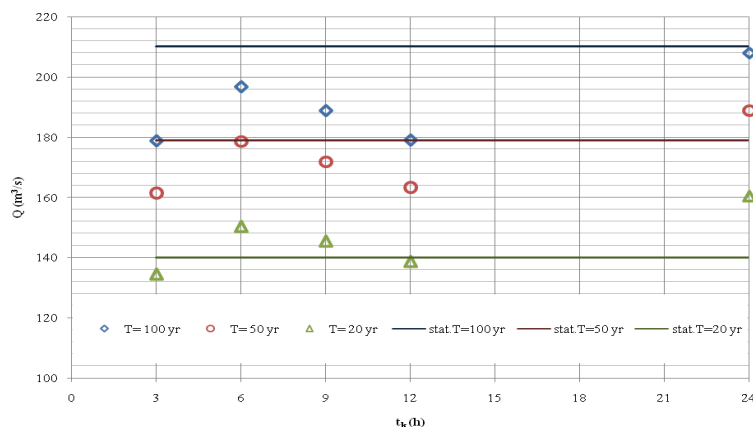


Fig. 3 Design floods of the Obnica River at Belo Polje based on design storms (markers) and obtained by statistical analysis (lines). The x-axis denotes design storm duration t_k .

Table 7 Design floods for different design storms and flood hydrograph parameters (basin lag t_p , time to peak T_p , recession time T_r , hydrograph base T_b)

Design storm type	Design floods			Synthetic unit hydrograph elements			
	$Q_{1\%}$ m ³ /s	$Q_{2\%}$ m ³ /s	$Q_{5\%}$ m ³ /s	t_p h	T_p h	T_r h	T_b h
$tk=3h, i=const.$	178.7	161.5	134.5	10.2	11	38.5	49.5
$tk=6h, i=const.$	196.6	178.6	150.3	10.2	12.5	39.5	52
$tk=9h, i=const.$	188.8	171.9	145.4	10.2	14.5	40.5	55
$tk=12h, i=const.$	179	163.3	138.7	10.2	16	41.5	57.5
$tk=24h, AB$	208	188.9	160.4	10.2	11	40	51
Stat. analysis [3]	210	179	140				

Compared to the flood peaks estimated by statistical methods (Log-Pearson type 3 probability distribution [5]), it can be seen that the best agreement in the 1% design floods is obtained with synthetic design storm of 24 hours duration. Among the block storms of various durations, the 2% design flood closest to the statistical ones was obtained with the 6 hours block storm, while the 12 hour block storm is the closest to the 5% flood. In general, block storms of all durations performed well for higher (5%) probability of exceedance, while the 24-hour alternating block storm outperforms block storms for lower probabilities of exceedance (2% and 1%).

5. CONCLUDING REMARKS

By calibrating a flood hydrograph model based on synthetic unit hydrographs for a gauged basin (the Obnica River) using the HEC-HMS software and then applying the calibrated model to the design storms – a procedure typical for ungauged basins, it was possible to analyse the uncertainties in estimating design floods at the ungauged basins. The results have shown that the following conclusions can be made.

The highest values of design floods with return periods of 20, 50 and 100 years were obtained with the 24 hour temporally distributed synthetic design storm. Constant intensity storms of shorter durations fail to reproduce design floods of low frequency such as the 100-year flood. Among the block storms, the 6-hour design storm provides the highest design floods; this indicates that 6 hours is the critical storm duration for the Obnica River basin, what is in accordance with other studies [9] [18]. At the same time, it is obvious that the assumption on the constant rainfall intensity is not justified for the given catchment size and critical storm duration.

For application of the commonly used SCS curve number (CN) procedure the model calibration has shown that it would be of great importance to develop a unique method of estimating CN in Serbia based on the observed rainfall and flows. Curve number CN is usually estimated from the tables provided by SCS (NRCS) [16] based on different types of land cover and use. Such estimation needs to be compared to the neighbouring catchments (similar) where there are data on observed flows and rainfall.

Design flood hydrograph estimation using the design rainfall concept is affected by the selected hyetograph form, especially in terms of peak flow and time to peak. Also, realistic flood hydrographs specific for considered area can be determined only through

calibrating a rainfall-runoff model considering soil structure, moisture conditions and processes in the catchments. However, since this procedure is impossible at ungauged catchments, it is necessary to carefully select not only the elements of the synthetic unit hydrographs, but also to provide a set of design storms that can produce realistic design floods.

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REFERENCES

1. Bhunya P. K, Panda S. N., and Goel M. K. (2011) "Synthetic Unit Hydrograph Methods: A Critical Review", *The Open Hydrology Journal*, 5, 1-8
2. Bossard M., Franec J., Othel J. (2000) *Corine Land Cover Technical Guide – Addendum 2000*, Technical report No 40, Copenhagen (EEA)
3. Chow V., T., Maidment D., R., Mays L., W. (1988) *Applied Hydrology*, McGraw-Hill
4. Cunderlik J., Simonović S. (2004) Calibration, verification and sensitivity analysis of the HEC-HMS hydrologic model, CFCAS project: Assessment of Water Resources Risk and Vulnerability to Changing Climate conditions, Project report IV, University of Western Ontario, University of Waterloo, Upper Thames River Conservation Authority
5. Institut za vodoprivredu „Jaroslav Černi” (2009) *Vodoprivredna Osnova Republike Srbije – Hidrometeorološke podloge*
6. Kranse P., Boyle D. P., Base F. (2005) "Comparison of different efficiency criteria for hydrological model assessment, *Advances in Geosciences*", 5, pp 89-97
7. McCuen R. H. (2005) *Hydrologic analysis and design*, Pearson Education, Inc. Upper Saddle River, New Jersey
8. Mijalić M., Volf G., Ožanić N. (2009) "Određivanje Hidrograma otecanja korišćenjem HEC – HMS programa", *Zbornik radova, Knjiga XII, Građevinski fakultet Sveučilišta u Rijeci*, pp 55 - 86
9. Petrović J. (1996) *Usporedna analiza maksimalnih proticaja na malim rekama metodama parametarske i statističke hidrologije*, Magistarski rad, Građevinski fakultet, Beograd
10. Prohaska S. (2003) *Hidrologija I deo*, Rudarsko-geološki fakultet, Institut za vodoprivredu "Jaroslav Černi" i RHMZ Srbije, Beograd
11. Prohaska S., Bartoš Divac V. sa saradnicima (2014) *Intenziteti jakih kiša u Srbiji*, Monografija, Izdavač: Institut za vodoprivredu „Jaroslav Černi”, Beograd, ISBN 978-86-82565-40-6, str. 1-481
12. Šraj M., Dirbek L., Brilly M. (2010) "The Influence of Effective Rainfall on Modelled Runoff Hydrograph", *J. Hydrol. Hydromech*, 58, 1, pp 3 – 14
13. US Army Corps of Engineers, Hydrologic Engineering Center (2008) *HEC – HMS Applications Guide*
14. US Army Corps of Engineers, Hydrologic Engineering Center (2000) *HEC – HMS Technical Reference Manual*
15. United States Department of Agriculture (1986). *Urban hydrology for small watersheds*, TR-55
16. United States Department of Agriculture, National resources conservation Service (2004) *NEH 630 Chapter 9, Hydrologic Soil – Cover Complexes*, 210 – VI – NEH
17. Yusop Z., Chan C. H., Katimon A. (2007) "Runoff Characteristics and Application of HEC – HMS for Modelling Stormflow Hydrograph in an Oil Palm Catchment", *Water Science and Technology*, Vol 56, No 8, pp 41 – 48
18. Zlatanović N. (2016) *Metod distribuiranih brzina za određivanje vremena kašnjenja sintetičkog jediničnog hidrograma*, *Zbornik 17. savetovanja SDHI i SDH*, pp. 458-469.

ODREĐIVANJE RAČUNSKIH PROTOKA MODELOM PADAVINE-OTICAJ NA MALIM SLIVOVIMA

U ovom radu je predstavljen model za simulaciju pika poplavnog talasa na malom slivu reke Obnice u Srbiji (površine 185 km²) sa ciljem određivanja računskih velikih voda koristeći različite pristupe. Primjenjen simulacioni model je napravljen u HEC-HMS programu (The United States Army Corps of Engineers (USACE) Hydrologic Center's Hydrologic Modeling System). Model je kalibrisan na osnovu osam snimljenih hidrograma i odgovarajućih kišnih epizoda i verifikovan za hidrograme i kišne epizode koji nisu učestvovali u procesu kalibracije. Za simulaciju su korišćene računске kiše konstantnog inteziteta različitog trajanja kao i kiše trajanja 24 časa čiji je hijetogram formiran metodom alternativnih blokova. Svi simulirani protoci upoređeni su sa protocima dobijenim statističkom analizom maksimalnih godišnjih protoka. Rezultati pokazuju da se uključivanjem vremenske neravnomernosti u računске kiše trajanja 24 časa dobijaju vrednosti protoka koje su najbliže rezultatima statističke analize, dok to korišćenjem kiša konstantnog inteziteta nije bilo postignuto.

Ključne reči: računске velike vode, računске kiše, padavine-oticaaj, simulacija, HEC-HMS, hidrogram velikih voda