JEE Journal of Ecological Engineering

Journal of Ecological Engineering 2022, 23(7), 246–256
https://doi.org/10.12911/22998993/149721
ISSN 2299–8993, License CC-BY 4.0

Received: 2022.04.12
Accepted: 2022.05.16
Published: 2022.06.01

Trend Analyses of River Dragacina Runoff for Identification of the Water Availability and Accounting for Water Needs

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ABSTRACT

This study aimed to analyze the available amount of water in the Dragaçina River to meet the different water needs in the Municipality of Suhareka. The water problems in this city are more pronounced, especially in the vegetation period of July–September, where the area is significantly affected by drought. The Dragacina River carries about 10 hm^3 of water per year, and affected neither by urbanism nor massive deforestation of the basin. However, there are no multi-year measurements of inflows for this river, whether they are average, maximum or minimum ones. Therefore, the study is based on several multi-annual monthly rainfall measurements and some characteristics of the Dragaçina River Basin. Knowing the average annual flow coefficient $\eta = P_{eff} / P_{bruto}$ it is possible to convert these precipitations to P_{eff} [mm] flow and then to monthly flow. The inputs for other years from 1983/84 onwards are obtained by simulating time series. Then, for such inflows, the probability distribution functions of small waters are assigned and the usable volume balance is carried out. Assuming an average annual withdrawal from the reservoir $QA_{min}^{mes.} = 0.63 \times Q_{mes.}$ which should be constant throughout the years, then the length of the critical period will be 0.13 years or approximately 48 days, for $P_H = 95\%$. Starting from the initial acquired volume of 1 hm^3 it is possible to achieve $95\% < P_H < 99\%$. Therefore, it follows from this analysis that this river can provide a significant amount of water for the needs of the Municipality of Suhareka.

Keywords: time series, flow simulation, accumulation, statistical parameters, probability.

INTRODUCTION

The municipality of Suhareka is located in the southern part of the Republic of Kosovo, has an area of 361.78 km^2 and a population of about 88126 inhabitants (Municipal Development Plan Suhareka 2020–2028). To the northeast of Suhareka lies the Dragaçina River, the basin of which has an area of 39.6 km^2 and mainly mountain vegetation cover that protectes it from the erosion process. The main tributaries of Dragacina are the tributary that originates in the village of Budakovë at an altitude of 1120 m and the tributary that comes from the village of Greiçec, as well as a series of other smaller streams that feed this river. For this reason, the Dragacina port is more suitable for accumulation, which would enable a more rational use of water in this area.

Analysis and simulation of feeds

From Kosovo Hydro Meteorological Institute, it was only possible to obtain monthly rainfall data for a period of about 80 years for the city of Suhareka, but only a time series of 30 years is marked, while in other years there are disconnections of measurements for various reasons. Since the Dragacina River flow measurements are unavailable and taking into account that the average annual flow coefficient is $\eta = 0.362$ (Yaraslov Černi Institute, 1983), then the rainfall must be converted to the flow according to the expression $P_{eff.} = 0.362 \cdot P_{bruto}$. Table 1 gives the monthly rainfall for 30 full years at the Suhareka hydrometric station.

The average annual flow coefficient $P_{eff} / P_{bruto} = \eta = 0.362$ shows that on average 36.2% of the total water is falling into the pond flows. The regional analysis of the average water flow provides

Table 1. Monthly rainfall for 30 years at [mm] -st. Suhareka hydrometric Station, Raingauge RG-34-02

Hydrological year												
Year	O	N	D	J	F	M	A	M	J	J	A	S
1954/55	112	120	66	48	136	37	57	16	56	94	68	121
1955/56	93	143	50	66	121	49	70	95	50	10	18	13
1956/57	20	103	28	7	25	16	49	139	43	59	88	71
1957/58	134	22	44	97	23	222	92	58	22	9	38	25
1958/59	41	50	40	112	18	18	24	69	44	134	58	111
1959/60	37	78	77	42	72	69	54	131	27	73	2	83
1960/61	122	116	64	24	25	91	45	161	32	52	21	28
1961/62	14	125	68	44	56	127	103	30	70	70	5	48
1962/63	86	104	137	214	151	48	65	52	36	18	30	28
1963/64	44	67	160	7	46	47	126	94	128	78	60	76
1964/65	65	93	67	27	58	84	78	113	41	22	57	11
1965/66	14	107	144	157	44	78	72	52	66	50	7	27
1966/67	49	109	110	68	11	91	103	59	42	86	9	36
1967/68	23	19	94	92	50	23	22	66	98	18	96	68
1968/69	18	73	88	34	62	96	81	90	60	53	25	71
1969/70	3	54	83	116	125	108	63	79	36	76	26	13
1970/71	132	72	68	82	37	93	20	60	58	75	21	113
1971/72	10	46	22	49	15	7	33	32	41	115	60	186
1972/73	72	100	1	54	63	40	76	58	77	268	45	106
1973/74	80	97	133	52	41	29	51	159	61	24	24	31
1974/75	155	59	60	37	10	27	56	64	162	47	12	25
1975/76	41	60	40	76	17	35	57	72	114	79	56	62
1976/77	29	129	142	71	93	29	53	53	65	49	91	45
1977/78	37	115	71	81	73	116	72	134	51	11	8	230
1978/79	53	15	130	136	47	27	74	47	122	48	127	50
1979/80	78	180	58	97	33	48	21	118	69	40	36	26
1980/81	104	112	115	80	56	73	58	58	24	50	92	72
1981/82	94	120	90	24	22	121	52	26	22	60	62	19
1982/83	39	34	79	20	98	19	32	49	199	65	31	55
1983/84	44	79	71	96	82	97	58	37	60	46	96	57

an opportunity to approximately determine the average flow in the places where no measurements have been made (Husno Hrelja, Inženjerska Hidrologija, 2007). Therefore, the average inflows for the years given in Table 1 will be determined according to the following expression:

$$P_{eff.} = \frac{Qm. \times t}{F} \quad (1)$$

where: $Q_{m.}$ – average flow in [m^3/s],
F – basin area in [km^2] and t – time in [s].

Separating $Q_{m.}$ from eq. (1) as well as assuming that P_{eff} = H * P_{bruto} we obtain:

$$Qm. = \frac{\eta \times P_{br.} \times F}{t} \ [\mathrm{m}^3/\mathrm{s}] \quad (1')$$

The year 1988 was historically quite a dry year and this is thought to have been the result of two events: the explosion of the Chernobyl nuclear power plant (1986) and the explosion of Supernova A (February 23, 1987). Therefore, an assessment of time series homogeneity is considered necessary. Thus, the year 1988 was used as the year in which a change in the flow regime could have occurred and for this reason it is necessary to simulate the inflows from 1983/84 until today.

Time series models are often based on first-order Markov models:

$$x_i = \mu_x + \rho_1 \times (x_{i-1} - \mu_x) + \\ + t_i \times \sigma_x \times (1 - \rho_1^2)^{1/2} \quad (2)$$

Or for the given time series:

$$x_i = \mu_x + b_i \times (x_{i-1} - \mu_{i-1}) + + t_i \times s_x \times (1 - r_i^2)^{1/2} \quad (2')$$

where: b_i represents the regression coefficient expressed as:

$$b_i \cong r_i \times \left({}^{s_i}/_{s_{i-1}}\right) \quad (3)$$

The generation of synthetic time series is taken according to Fiering, who gives the following formula for a normally distributed series:

$$q_{i,j} = \mu_j + \frac{r_j \times s_j}{s_{j-1}} \times \left(q_{i,j-1} - \mu_{j-1}\right) + + t_i \times s_j \times (1 - r_j^2)^{1/2} \quad (4)$$

where: $q_{i,j}$ – i[th] generated flow in time interval t (j = 1,…, 12);
μ_j – average flow in the month j ($j \leq 12$);
t_i – random numbers normally distributed N (0,1).

The statistical parameters for equation (4) are defined as:

- average flow:

$$\mu_j = \frac{1}{N}\sum_{k=1}^{N} x_{k,j} \quad (5)$$

- variance:

$$s_j^2 = \frac{1}{N}\sum_{k=1}^{N} x_{k,j} - \frac{1}{N(N-1)}\left(\sum_{k=1}^{N} x_{k,j}\right)^2 \quad (6)$$

- autocorrelation coefficient:

$$r_j = \frac{\sum_{k=1}^{N} x_{k,j} \times x_{k,j-1} - N \times \mu_j \times \mu_{j-1}}{s_j \times s_{j-1} \times (N-1)} \quad (7)$$

If the monthly feeds are not normally distributed, then the values of (t_i) need to be transformed. Considering the coefficient of the historical time series asymmetry (C_S), the following transformation of the size (t) normally distributed to the size (t_g) of the Gamma distribution is recommended:

$$t_g = \frac{2}{C_{t,j}} \times (1 + \frac{C_{t,j} \times t_i}{6} + \frac{C_{t,j}^2}{36})^3 - \frac{2}{C_{t,j}} \quad (8)$$

where:

$$C_{t,j} = \frac{C_{sj} - r_{j-1}^3 \times C_{sj-1}}{(1 - r_j^2)^{3/2}} \quad (9)$$

t_g – random Gamma distribution numbers (0.1, C_g),
C_{sj} – coefficient of asymmetry of the month j.

On the basis of what was said above, for the series of 30-year monthly feeds, the following statistical parameters written in Table 2 were obtained.

Taking the initial inflow of September as the average, i.e. μ_0 = 0.3515 and applying equation (4) the inflow values for the respective months are obtained.

For these 30 years, the time series and the simulated one have these results as in the following graphs.

The dependence between them has also been tested through the statistical test of Durbin-Watson (Başkent Universitesi, Istatistiksel formüller ve tablolar, 2005), where the null hypothesis is laid as:

Table 2. Statistical parameters of the series of average monthly inflows in [m^3 / s], N = 30 years

Month	μ_j	s_j	Cs	r_j	b_j
O	0.3288	0.2233	0.6155	-0.1902	-0.1520
N	0.4795	0.2169	-0.0059	0.0062	0.0059
D	0.4281	0.2109	0.2911	0.1313	0.1277
J	0.3764	0.2463	1.1600	-0.1754	-0.2048
F	0.3377	0.2288	0.9378	0.1177	0.1093
M	0.3505	0.2466	1.3801	-0.2472	-0.2665
A	0.3349	0.1404	0.4533	-0.082	-0.0467
M	0.4051	0.2113	0.7686	-0.0487	-0.0733
J	0.3642	0.2320	1.6716	-0.0224	-0.0246
J	0.3352	0.2628	2.6357	-0.0126	-0.0142
A	0.2442	0.1779	0.6582	-0.2130	-0.1442
S	0.3515	0.2794	1.7415	0.0125	0.0197

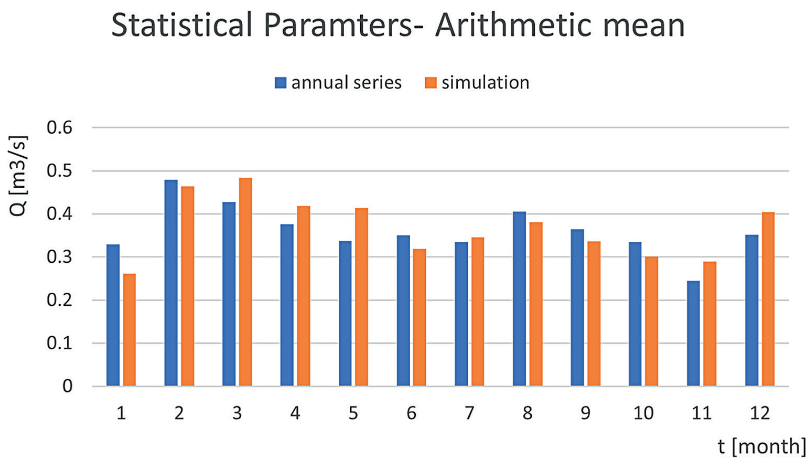

Figure 1. Comparison of time series with simulated one (1954 / 55-1983 / 84) (Maniak, 2010)

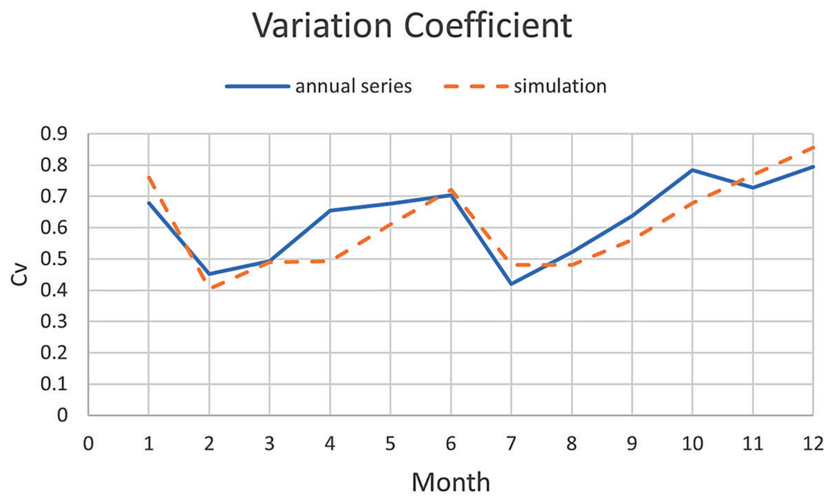

Figure 2. Comparison of time series and that simulated series via Cv (Maniak, 2010) (data source: KHMI)

$$H_0 : \rho = 0$$
$$H_a : \rho \neq 0$$

$$DW = d = \frac{\sum_{i=2}^{n}(e_i - e_{i-1})^2}{\sum_{i=1}^{n} e_i^2} \tag{10}$$

where: $e_i = y_i$ - $\acute{y}_i$ are the residuals between the observed and predicted values. According to the test, d = 2.623 was obtained, while from the table for N = 30, k = 1 and the significance level α = 0.05 the limit values were d_L = 1.35 and d_U = 1.49 (DurbinWatsonTest.dvi (nsysu.edu.tw). Due to the symmetry, the values (4-1.49) < d < (4-1.35) were obtained, which means the rejection of the null hypothesis and the alternative hypothesis Ha: $\rho \neq 0$ was accepted, indicating autocorrelation between the series.

To test the homogeneity of the long time series, it was divided into two subcategories: (N_1) before the show and (N_2) after the change. Thus, the homogeneity of the available series was checked by testing the statistical significance between the means (z-test) or between the standard deviations (variances) (F-test or Fischer test). In the following, the first N_1 = 34 (1954 / 55-1987 / 88) and the second subseries N_2 = 33 (1988 / 89-2020 / 21) were taken. The authors hypothesized the testing of statistical significance between averages:

$$H_0 : \mu_1 - \mu_2 = 0$$
$$H_a : \mu_1 - \mu_2 \neq 0$$

$$z = \frac{X_{m_1} - X_{m_2} - \delta}{\sqrt{\frac{s_1^2}{N_1} + \frac{s_2^2}{N_2}}} ; (n \geq 30) \tag{11}$$

From which:

$$z = \frac{0.374 - 0.3674}{\sqrt{\frac{0.0512^2}{34} + \frac{0.0596^2}{33}}} =$$

$$= 0.4856 \ \in [-2.81 - 2.81]$$

Since $z \leq z_{cr}$, then the null hypothesis was accepted, i.e., below the 5% probability level there is no statistically significant difference. Similarly, the possible differences between the variances can be tested; the null hypothesis is as follows:

$$H_0 : \sigma_1^2 = \sigma_2^2$$
$$H_a : \sigma_1^2 \neq \sigma_2^2$$

$$F = \frac{s_1^2}{s_2^2} \ ; \ df_1 = n_1 - 1 \ ; \ df_2 = n_2 - 1 \quad (12)$$

From which:

$$F = \frac{0.0596^2}{0.0512^2} = 1.355 \ ; \ df_1 = 33 \ ; \ df_2 = 32$$

Since $F = 1.355 \leq F_{cr} = 2.019$, then the null hypothesis below the significance level $\alpha = 0.05$ is accepted which means that there is no statistically significant difference between the variances.

Statistical analysis of low waters

The period in which the precipitation deficit occurs (in relation to any expected value) is known as drought. Since the lack of precipitation in the observed basin affects the reduction of river inflows and the decrease of the groundwater level, hydrological drought occurs, which means a longer period of time with low waters, with the river flowing significantly more smaller than the average flow (Husno Hrelja, Inženjerska Hidrologija, 2007). In other words, a meteorological drought causes hydrological drought or small waters. Discharges below (0.15–0.5) Q_{avg} are estimated approximately for the low water area (Ulrich Maniak, Hydrologie und Wasserwirtschaft, 2010). In our climatic zone there are periods with little water lasting from several weeks to 3 months, mainly from July to September.

Statistical analysis of low waters is usually based on the analysis of a series of minimum annual inflows with a certain duration (Δt = 7, 10, 14, 20, 30,… days). These series should be statistically sufficient in length and of satisfactory quality. If the statistical hypotheses are satisfied, then a satisfactory theoretical distribution is defined, where in small waters the Pearson III distribution, the log-Pearson III distribution, the extreme III-Weibull distribution and the Galton distribution are satisfactorily applied.

Annual extremes method

In the statistical analysis of the extreme values of the hydrological series, and consequently of the minimum annual inflows, the method of annual extremes was most often used. This method is based on the analysis of annual values (one data per year) over a multi-year period. The purpose of the analysis is to determine the probability of occurrence, namely the function of distributing the probability of minimum annual inflows. The probability distribution function $F \ p \ F(Q) = P[Q \leq Q^m] \ or \ \phi(Q) = P[Q > Q^m]$ is a complete distribution characteristic. This means that all results of a random variable (Q) can be obtained from the probability distribution function $F(Q)$ respectively $\phi(Q)$.

The values of the minimum annual inflows of a given return period (T) are determined by the equation:

$$Q_T = \frac{1}{F(Q)} \quad (13)$$

For this purpose, the series of minimum annual inflows (an extreme value per year) is taken, simulated from 1988/89 to 2020/21 (N = 33 years) and with duration Δt = 30 days. The statistical parameters of this group are calculated and an empirical probability distribution for the previously adjusted sample of the random variable is determined according to Weibull.

$$\phi_e = P[Q \leq Q_{min.}] = \frac{m}{N+1} \quad (14)$$

Some theoretical probability distribution functions, which serve to calculate the largest absolute differences between the empirical and theoretical probability distributions were selected. These include: Gaussian, Galton, Gumbel, Pearson, and log-Pearson distributions. All these theoretical probability distribution functions fit satisfactorily with the empirical distribution according to Kolmogorov's test, satisfying the condition

$d_{max} < c$ below the 5% probability level, respectively $P(d < c) = 1 - \alpha$. However, the function that has a lower value of:

Table 3. Statistical parameters of the series of minimum annual inflows (HRELJA, 2007) (data source: KHMI)

N = 33	$Q_{m.}$	s_Q	*Cv*	*Cs*
Q	0.08486	0.06251	0.73663	0.4922
logQ	-1.25835	0.48689	-0.38692	-0.90077

$$F(\lambda) = \lim_{n\to\infty} P\left[D_n \le \frac{\lambda}{\sqrt{n}}\right] \quad (15)$$

or higher value of [1 - $F(\lambda)$], has a better fit of the empirical probability distribution function with the theoretical distribution. Therefore, based on the latter, the theoretical probability distribution functions of Pearson III and log-Pearson III have lower values of $F(\lambda)$, but since the theoretical log-Pearson distribution is defined for any $x > 0$ ($y = \log(x)$), it was also chosen as the final theoretical distribution of probability.

The probability density function for the Gamma 3-parametric distribution (Pearson III distribution) is as follows:

$$f(x) = \frac{1}{\beta^\alpha \times \Gamma(\alpha)} \times (x - x_0)^{\alpha-1} \times e^{-\frac{x-x_0}{\beta}}; \quad x_0 < x < \infty \quad (16)$$

where, the three parameters of the function are:
α – form parameter;
β – scale parameter;
x_0 – position parameter.

The corresponding cumulative probability distribution function is:

$$F(x) = \int_{x_0}^{x} \frac{1}{\beta^\alpha \times \Gamma(\alpha)} \times (x - x_0)^{\alpha-1} \times e^{-\frac{x-x_0}{\beta}} dx; \quad x_0 < x < \infty \quad (17)$$

The characteristic statistical moments of the distribution are:

- Average value:

$$E[X] = \mu = x_0 + \alpha \times \beta \quad (18)$$

- Variance:

$$Var[X] = \sigma^2 = \alpha \times \beta^2 \quad (19)$$

- Coefficient of asymmetry:

$$Cs = \frac{M_3}{\sigma^3} = \frac{2}{\sqrt{\alpha}} \quad (20)$$

Using the frequency factor for this distribution $K_p = f(Cs; T)$ the flow for the respective return period was calculated:

$$x_T = \mu + K_p \times s_x = \mu \times \left[1 + K_p \times C_v\right] \quad (21)$$

If the logarithms of the variable X have a Pearson III distribution, then the variable X is said to follow the log-Pearson III distribution. Therefore, in equations (16), (17) and (20) the transformations were applied $y = \ln(x)$ or $y = \log(x)$, $x > 0$, so that the flow is:

$$y(T) = \mu_y + K_p \times s_y ;\ K_p = f\left(C_{sy}; T\right) \quad (21')$$

The value of the random variable (x) for the corresponding return period will be:

$$x_T = e^{y(T)} \ ose\ x_T = 10^{y(T)} \quad (22)$$

There is great practical interest in defining the boundaries within which, with a certain probability, a population distribution function can be found, provided that the type of this function is known. In other words, the confidence intervals of the probability distribution function, the parameters of which are estimated on the basis of the group, must be determined. For this purpose, empirical methods will be used, assuming a normal x_T event distribution function, a confidence interval with a security factor of 1 - τ.

$$P\left[x_T^d < x_T \le x_T^g\right] = 1 - \tau \quad (23)$$

where: x_T^d and x_T^g are the lower and upper limits of the confidence interval, respectively, which are defined using the following expressions:

- Lower limit of confidence interval:

$$x_T^d = x_T - z_{1-\tau/2} \times SG_T \quad (24)$$

- Upper limit of confidence interval:

$$x_T^g = x_T + z_{1-\tau/2} \times SG_T \quad (25)$$

where: $z_{1-\tau/2}$ – is the value of the standardized normal variable for the selected security factor (confidence level) of 1-$\tau/2$;
SG_T – represents the standard estimation error, which according to this empirical method, for a random variable of a given period of return T, is calculated using the following expression:

$$SG_T = \delta_T \times \frac{S}{\sqrt{n}} \quad (26)$$

where: S – is the standard deviation based on the series (group),
δ_T – is a function of the frequency factor (K) and a certain number of statistical moments, depending on the applied theoretical distribution function.

The standard estimation error for the log-Pearson III $SG_T(y)$ distribution ($y = \log x$), can be calculated according to the same equations as for the Pearson III distribution (Husno Hrelja, Inženjerska Hidrologija, 2007), but including the logarithmic group values (Sy, Csy). The standard estimation error thus defined for the logarithmic group $SG_T(y)$ is converted to the standard error of the group (without calculation) $SG_T(x)$, through the equation:

$$SG_T(x) = \frac{x_T \times [\exp(SG_T(y) - \exp(SG_T(y))]}{2} \quad (27)$$

Finally, we can written the confidence intervals for log-Pearson III distribution function:

$$x_T^{d,g} = x_T \pm z_{1-\tau} \times SG_T(x) \quad (28)$$

The selected theoretical probability distribution function (log-Pearson III) together with the corresponding empirical function as well as the 95% confidence interval limits are graphically presented in Figure 3.

Aspect of low water management

From the point of view of water use and protection, the following two management regimes are especially important:
a) Minimum water management (Q_{vm}) and
b) Guaranteed ecological minimum (Q_{em}).

Guaranteed ecological flow should always be provided in the water flow for the survival and normal development of the flora and fauna in it. It enters management tasks as a constraint, as opposed to minimum water management as a management variable.

Dorđević and Dašić propose to determine Q_{em}:
1. In the extra-vegetative period (October-March), Q_{em} is selected based on the relation:

$$Q_{em} = 0.1 \times \bar{Q}_g \; for \; Q_{95\%} \leq 0.1 \times \bar{Q}_g \quad (29)$$

$$Q_{em} = Q_{95\%} \; for \; 0.1 \times \bar{Q}_g \leq Q_{95\%} \quad (30)$$

$$Q_{em} = 0.15 \times \bar{Q}_g \; for \; Q_{95\%} \geq 0.15 \times \bar{Q}_g \quad (31)$$

where: $\bar{Q}_g$ represents the average multi-year watercourse flow, while
$Q_{95\%}$ – is the average minimum monthly flow with 95% certainty.

In the vegetation period (April–September), Q_{em} is selected by the conditions:

$$Q_{em} = 0.15 \times \bar{Q}_g \; for \; Q_{80\%} \leq 0.15 \times \bar{Q}_g \quad (32)$$

$$Q_{em} = Q_{80\%} \; for \; 0.15 \times \bar{Q}_g \leq Q_{80\%} \quad (33)$$

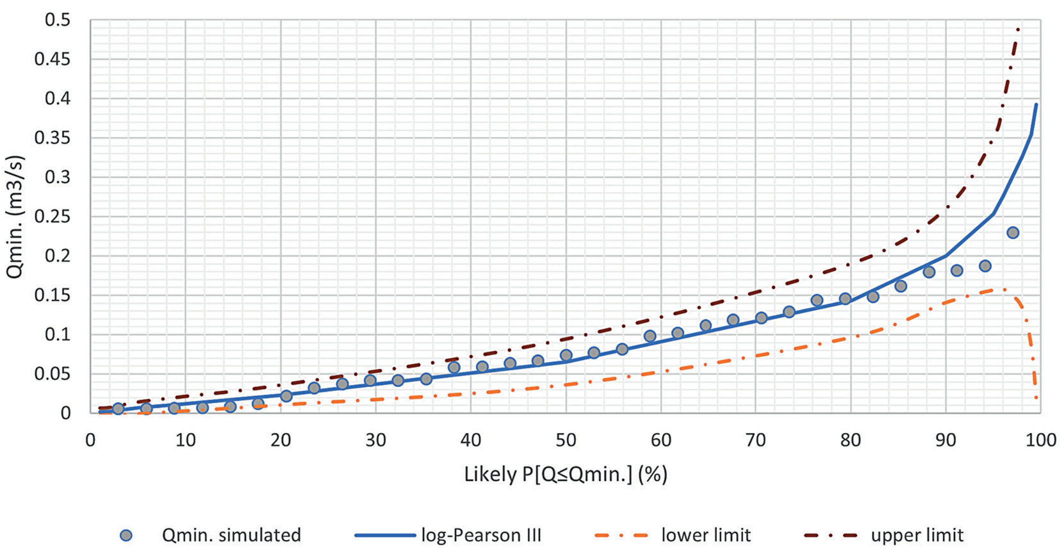

Figure 3. Graph of the log-Pearson III probability distribution function and corresponding 95% confidence intervals for the minimum annual Dragaçina River inflows

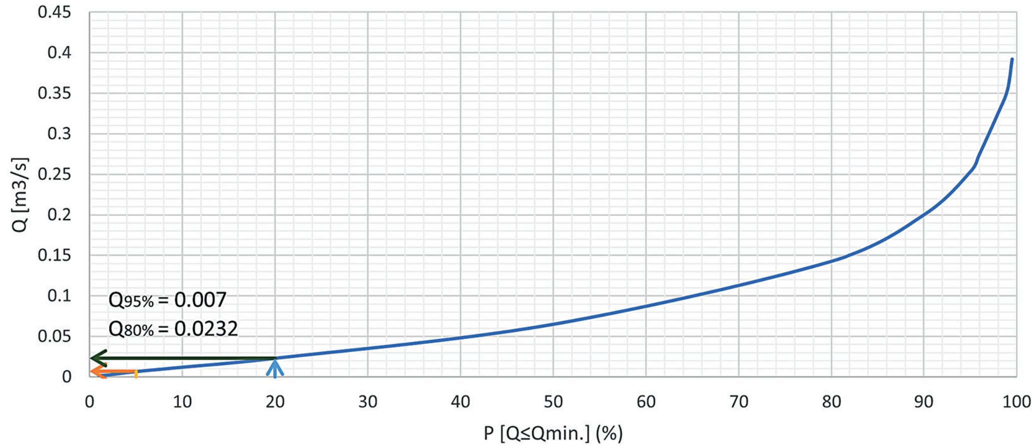

Figure 4. Distribution probability function of the average monthly average inflows of the Dragaçina River

$$Q_{em} = 0.25 \times \bar{Q}_g \; for \; Q_{80\%} \geq 0.25 \times \bar{Q}_g \quad (34)$$

where: $Q_{80\%}$ – is the average minimum monthly flow with 80% certainty.

For $\bar{Q}_g$ = 0.366 m³ / s and for $Q_{80\%}$ and $Q_{95\%}$ read in Figure 4, analogous to the graph in Figure 3, the following can be obtained:

- For the extra-vegetative period Q_{em} = 37 l / s,
- For the vegetation period Q_{em} = 55 l / s.

The feed defined by the relations above is also consistent with the expression used in France and Austria:

$$(0.15 - 0.20) \times \bar{Q}_g \quad (35)$$

Water management plan based on the cumulative curve

The water management plan is designed to assess the long-term compensatory effect of a dam. The usable space is determined based on the storage equation, where the left side of the equation contains the mean values over the increase of the calculation of Δt:

$$QZ - QA + N - V = \frac{\Delta S}{\Delta t} \quad (36)$$

where: QZ – average flow in the dam in the time interval Δt in [m³ / s],
QA – average discharge (including overflow) in the time interval Δt in [m³ / s],
N – precipitation on the surface of the reservoir during Δt in [m³ / s],
V – evaporation from the reservoir surface during Δt at [m³ / s],
ΔS – change of reservoir volume in Δt to [m³] ($\Delta S = S_{t+\Delta t} - S_t$).

If the inflow amount is above the discharge amount, a surplus is displayed, while a deficit exists if at any time the total discharge exceeds the

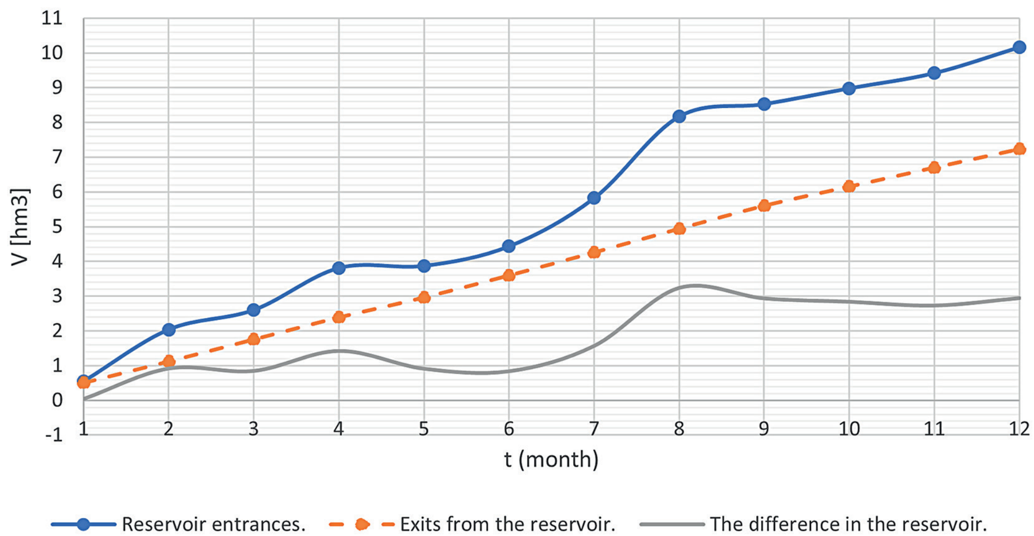

Figure 5. Cumulative inlet, outlet and change reservoir curves for a characteristic year

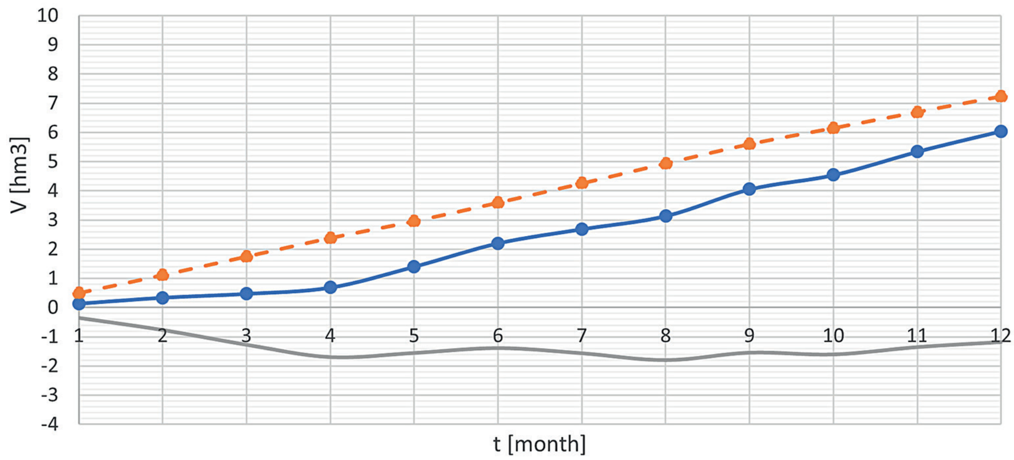

Figure 6. Cumulative inlet, outlet and change reservoir curves for a dry year

total inflow. In addition to the deficit, its duration also plays a role, for which there is no longer a complete planned supply. Deficits in individual years are taken accordingly as the maximum difference between the cumulative input and output curves. The following graphs show the cumulative curves (Jasna Plavšić & Zoran Radić, Inženjerska hidrologija-rešeni zadaci, 2015) for a characteristic year and another extremely dry year (1967/68), whereas discharges in 8 months are taken from 200 l / s, while in other 4 months from 150 l / s without counting here Q_{em} in the respective periods.

As it can be seen from the graphs in Figures 5 and 6, the cumulative curve is presented in the orthogonal coordinate system as an ascending curve. It is more convenient to choose the average flow as the reference value, which corresponds to the curve connecting the start and end point of the cumulative curve. The sum of successive surpluses and deficits gives the size of the usable volume of the dam for the respective successive wet or dry periods. The required volume size for the entire period under review is the largest value of the amount of surplus and subsequent deficit. At the end of each filling phase, the tank volume indicates the highest filling level during this period.

On the basis of the above, inflows of three consecutive typical years are taken, the average of which corresponds to about 0.34 m^3 / s, and the usable volume of the reservoir is examined.

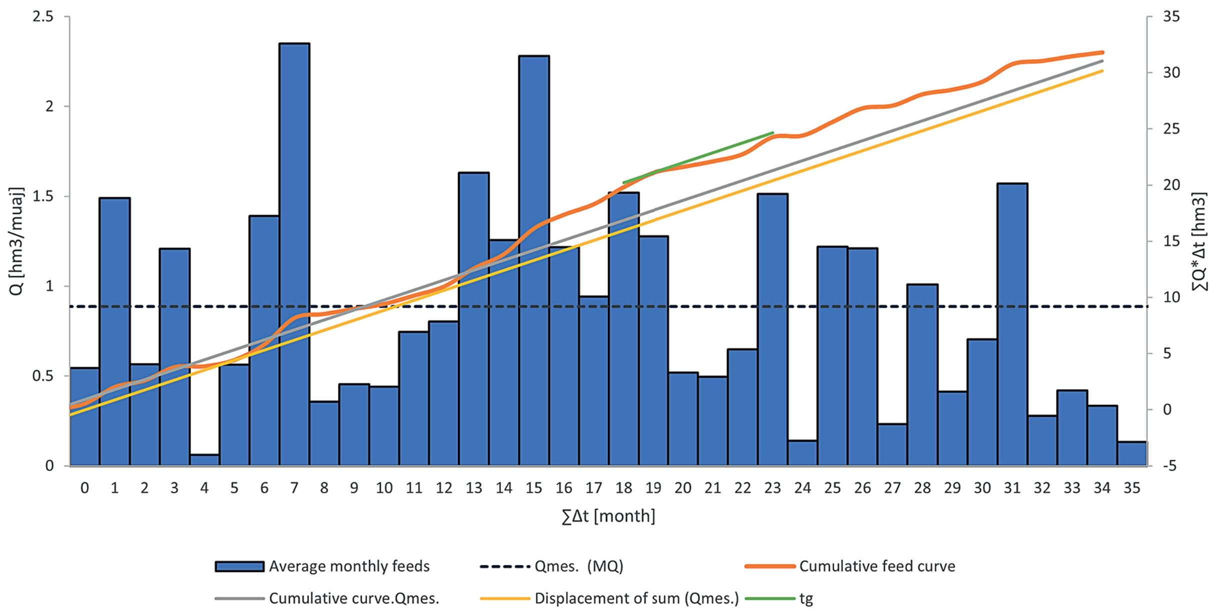

Figure 7. Usable volume of the reservoir for three consecutive ordinary years (Maniak, 2010)

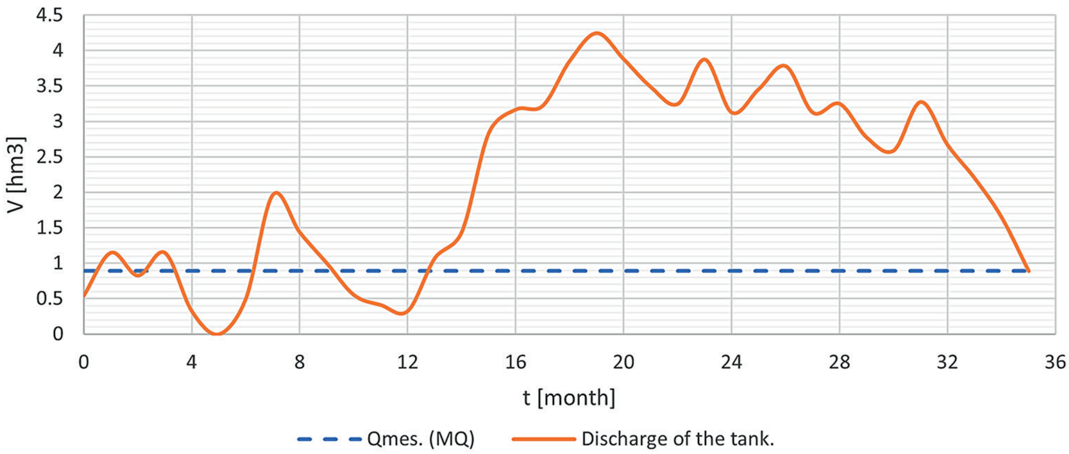

Figure 8. Usable volume Vsh. optimal reservoir for the relevant period (Maniak, 2010)

Here, the average is set as $\sum(Q \times \Delta t) / (3 \times 12)$, respectively 31.93725 / 36 = 0.8871 hm^3 / month and represents a continuous discharge for each month. The parallel displacement of the cumulative discharge curve is done because an initial volume is needed to cover the maximum deficit, which in this case is D = 0.89 hm^3. This volume may be required only in extremely dry periods.

The usable volume for this 3-year period will be:

$$V_{sh.} = |3.354 - (-0.89)| = 4.248\ hm^3$$

Acquiring an initial volume of 1 hm^3 and that from eq. (36) $V \leq N$ we obtain the usable volume of the reservoir $V_{sh.} = 4.5$ hm^3.

The following graph gives the relevant downloads along with Q_{em} for a dry hydrological year like that of 1967/68 (the Jaroslav Černi Institute in this year had organized measurements of daily river inflows for 9 consecutive months, respectively October–August).

Volume based on the theory of probability of inflows and reservoir fillings

If a degree of compensation α is to be guaranteed, the volume required for the critical period nkr. taken as:

$$S_{n,p} = \alpha \times n \times x - Q_{n,p} \tag{37}$$

where: $Q_{n,p}$ – is the sum of the n-year input with a probability of stagnation P_u, which is defined as:

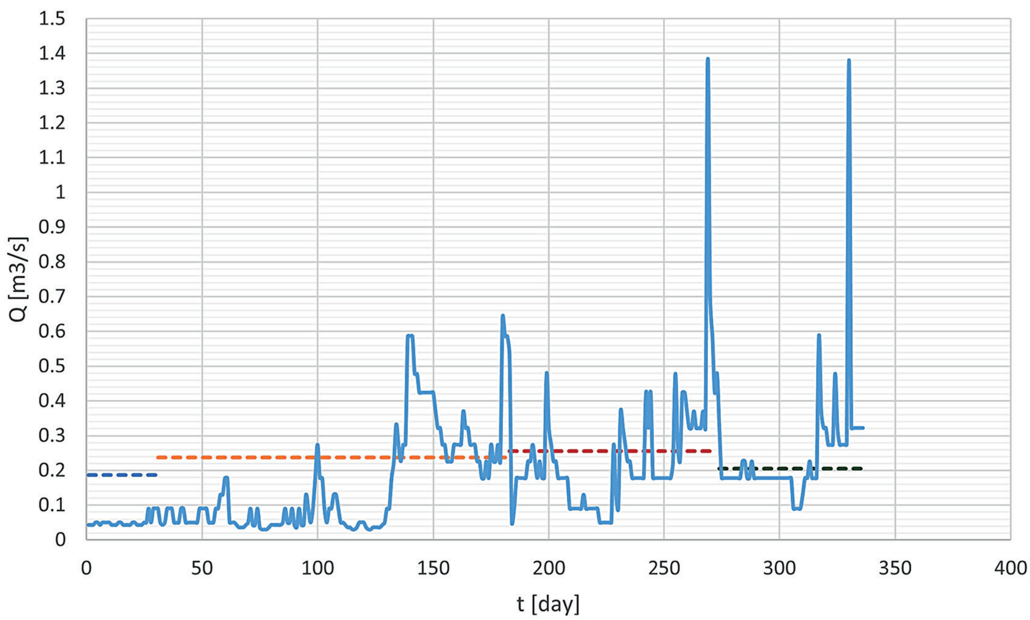

Figure 9. Surplus and deficit during a dry hydrological year (HMI Jaroslav Černi, 1983)

$$Q_{n,p} = n \times x - z_p \times \sqrt{n} \times s \quad (38)$$

whereas: z_p – the standard normal distribution variable for P_u (C_s = 0),
$n \times x = x^n_{mes.}$ the n-year average of the independent series of annual feeds.

Assuming an average annual withdrawal from the reservoir $QA^{\text{mes}}_{\text{min}} = 0.63 \times Q_{\text{mes.}}$ which should be constant throughout the years, then the length of the critical period will be:

$$n_{kr.} = \frac{z_p^2}{4 \times (1-\alpha)^2} \times C_v^2 \quad (39)$$

respectively, for P_H = 95%, derives $n_{kr} \cong 0.13$ years ~ 48 days.

For the same security, the degree of expansion β will be:

$$\beta = \frac{z_p^2}{4 \times (1-\alpha)} \times C_v^2 \quad (40)$$

hence: $\beta = 0.048 \times 1.2 = 0.0576$, while the volume usable for the critical period will be $S = \beta \times Q_{\text{mes.}}$ 0.665 hm³.

Similar is also achieved through eq. (36) and (37), where for the critical period of 2 months (60 days) we have $S_{n,p} = -0.653$ hm³.

With the initial acquired volume of 1 hm³ given in point 5. 95% confidence was obtained, i.e. 95% < P_H < 99%.

CONCLUSIONS

The performed study aimed at analyzing the available amount of water in the Dragaçina River, for which the river does not have multi-year measurements of inflows, whether they are average, maximum or minimum. Therefore, the study is based on a series of multi-year monthly rainfall measurements, for the period of 30 years (1954 / 55-1983 / 84) and some characteristics of the Dragaçina river basin, such as the average annual flow coefficient $\eta = P_{eff} / P_{bruto}$ that enabled the conversion of these precipitations to P_{eff} flow [mm] and hence to monthly flow. Given that there is a lack of data on feeds for other years after 1983/84, these feeds were obtained by simulating time series. For such inflows the probability distribution functions of small water were assigned and then the usable volume balance was made.

Assuming an average annual withdrawal from the reservoir $QA^{\text{mes}}_{\text{min}} = 0.63 \times Q_{\text{mes.}}$ which should be constant throughout the years, then the length of the critical period will be 0.13 years or approximately 48 days, for P_H = 95%. For the same certainty, the degree of expansion β will be $\beta = 0.048 \times 1.2 = 0.0576$, while the usable volume for the critical period will be 0.665 hm³. The same is achieved for the critical period of 2 months (60 days) when we have $S_{n,p} = -0.653$ hm³. To increase the safety, it is recommended that this volume be 1 hm³, which also plays the role of the initial filling volume.

Starting from the initial acquired volume of 1 hm³, 95% confidence was obtained, i.e. 95% < P_H < 99%. The amount of accumulated water will be able to provide in 4 months of the year from 150 l/s and in the other eight months from 200 l/s without including the ecological flow.

Finally, it follows from this analysis that this river can provide a significant amount of water for the needs of the Municipality of Suhareka.

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