

Article

Hydrological Analysis of a Dyke Pumping Station for the Purpose of Improving Its Functioning Conditions

Jerzy Machajski and Stanisław Kostecki * 

Faculty of Civil and Hydro Engineering, Wrocław University of Science and Technology, 50-370 Wrocław, Poland; jerzy.machajski@pwr.edu.pl

* Correspondence: stanislaw.kostecki@pwr.edu.pl; Tel.: +48-71-320-2340

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Abstract: This paper presents a methodology of calculating the water transfer capacity of a dyke pumping station in flood wave conditions in order to improve its functioning, especially with regards to the safety of the areas being drained. The exemplary analysis was carried out for a pumping station situated on a small right-bank tributary of the Odra River in the southwest part of Poland, which, due to the inadequate capacity of its pumps, extensively flooded the surrounding areas in May and June 2010. Hydrological analyses were conducted in order to determine the rate of the designed and control flows using a spatial regression equation, and as a comparison, the rainfall-runoff method was also used. The corresponding flood-wave hydrographs were also determined, which included total precipitation using the German Association For Water Resources and Land Improvement (DVWK) method, effective precipitation using the Natural Resource Conservation Service curve number (NRCS-CN) method, as well as hypothetical waves using the instantaneous unit hydrograph (IUH) method. Flood-wave routing was carried out and alternative solutions for both the output of the required pumps and the retarding reservoir capacity were highlighted on this basis. The paper presents the possibility of a correct pump capacity selection, and in turn, the size of the pumping station retarding reservoir that results from this selection. This will enable pumping station exploitation costs or maintenance costs of the retarding reservoir to be considerably reduced.

Keywords: hydrological model; dyke-pump station; design hydrograph; peak discharge; uncontrolled basin; small watershed; NRCS-CN method; rainfall-runoff model; spatial regression equation; retarding reservoir

1. Introduction

Dyke pumping stations on the tributaries of main rivers are hydraulic structures, the operation of which is vital for the flood protection of areas over dykes when the main river is flooded. Unfortunately, not enough attention is paid to the necessary dimensioning of such structures, which results in design errors such as the insufficient capacity of installed pumps, an inadequate capacity of a retarding reservoir, or even the lack of such a reservoir. The design of dyke pumping stations involves various difficulties when the catchment area is uncontrolled. In this case, the first step is to calculate flows using the probability of exceedance, known in Poland as designed and control flows. The genetic rainfall formula, the spatial regression equation, rainfall-runoff models, and other regional methods could be applied in Polish conditions to determine such flows, as well as the shape and volume of designated flood waves. Szolgay et al. [1] analyzed the relationship between flood peaks and the corresponding flood event volumes, which were modeled by empirical and theoretical copulas in a regional context. They tested the reliability of different formulas of synoptic floods, flash floods, or snowmelt floods and found out that different flood types should be treated separately. Grimaldi and Petroselli [2] proposed an approach that uses input information similar to those required by the Rational Formula

for application in fully ungauged basins. The core of their proposed procedure is the use of a rainfall excess estimation method and the use of an appropriately adapted geomorphological unit hydrograph that does not require runoff observations. Wan and Konyha [3] developed a consolidated conceptual model for the prediction of runoff from uncontrolled river basins. They proposed using three storage zones depending on the type and saturation of the soil. Outflows from these storage zones represent different flow components of catchment runoff, i.e., shallow subsurface flow, saturation limited overland flow, and direct runoff from impervious urban. Castiglioni et al. [4] proposed a method of rainfall-runoff model calibration in uncontrolled conditions using an approximate probable regional maximum approach. It consists, in practice, of a multiobjective calibration procedure to fit between the mean, standard deviation, and lag-one autocorrelation coefficient of river flows that are simulated by the rainfall-runoff model and the regional estimates of the same statistics. The other regional model, mostly used in southern Africa, is the Pitman model [5–7]. It is a conceptual, semidistributed, rainfall-runoff model that also includes functions to represent anthropogenic impacts on the availability of water resources as well as direct river abstractions. Jiang [8] used the Hydro-Informatics Modeling System (HIMS) for semiarid river basins. HIMS is based on the rainfall infiltration model (LCM), which contains infiltration excess and saturation excess runoff mechanisms and can thus reflect nonlinear runoff. He proposed the optimization method based on master–slave swarms in order to derive the model's parameters.

Among the many rainfall-runoff simulation models, the Natural Resource Conservation Service curve number (NRCS-CN) is one of the most enduring methods for estimating the volume of direct surface runoff in uncontrolled catchments [9]. The NRCS-CN method was initially developed by the Natural Resources Conservation Service of the United States Department of Agriculture. Initially, the method was developed for computation of direct surface runoff from storm rainfall events in small agricultural watersheds in the United States, but was very soon adopted for several regions, land uses, and climatic conditions. In this model, the curve number (CN) is a key variable that varies with cumulative rainfall, soil type, land use, and antecedent moisture conditions (AMC). Due to its simplicity, well documented environmental inputs, and also the incorporation of many other factors in a single CN parameter, the model has become the most popular for engineers and practitioners of small catchment hydrology [9–14]. The advantages of the NRCS-CN method are: its simplicity, predictability, stability, applicability for uncontrolled watercourses; its reliance on only two parameters (the CN that specifies the soil storage capacity (S) and also initial abstraction (Ia) that is expressed as a percentage of S); its convenience of use; and also its capability of incorporating easily accessible watercourse characteristics, such as soil type, land use/treatment, hydrological conditions, and AMC. Due to these many advantages, the method is still commonly used today among water resource practitioners and enjoys many applications across the globe [15].

It is also noted that several complex models, such as the Soil and Water Assessment Tool (SWAT), the Hydrologic Modeling System (HEC-HMS), the Erosion Productivity Impact Calculator (EPIC) and the Agricultural Non-Point Source Pollution Model (AGNPS), have been developed based on the NRCS-CN method [16–18].

The NRCS-CN method has a number of limitations and misinterpretations due to its basic empirical structure [15]. Firstly, this model was derived from approximately 10 years of rainfall-runoff data collected exclusively from agricultural and rangeland and therefore works well on agricultural sites, fairly well on range sites, and poorly on humid, semiarid, and forested regions. Secondly, this model is based on estimating the daily runoff that could result from daily rain storms, without considering the effect of the antecedent moisture in its basic formulation. Thirdly, this model does not contain any expression for time and, as a result, ignores the impact of rain intensity and time duration. In addition, there is no explicit provision for the spatial variability of rainfall. Therefore, in principle, it may not be appropriate for subdaily time resolution. Consequently, the use of this procedure in small catchments may be erroneous and prone to relatively large errors, as it requires hourly or subhourly temporal resolution of the net rainstorm hyetograph. The most criticized assumption in the CN method

is that the ratio of actual retention to potential retention is the same as the ratio of actual runoff to potential runoff.

A simple formula for calculating wave hydrographs, based on precipitation, was first proposed by Nash [19,20] and was based on the linear reservoir model to develop an instantaneous unit hydrograph (IUH) within a river basin. The linear reservoirs assumed in Nash's model are imaginary reservoirs in which storage is directly proportional to the outflow from them [21]. This model has been used by many authors (e.g., [21–23]).

This paper presents a methodology that was proposed by the authors, which is adequate for dimensioning dyke pumping stations for an uncontrolled river basin. In the presented paper, the river catchment is sufficiently described with all the required physical parameters and also has average annual hourly rainfall observations. This fulfills the known limitations of the NRCS-CN model. The spatial regression equation was used in order to make a comparison with the results calculated using the rainfall-runoff model. This allowed maximum flows with a definite probability of exceedance to be estimated for an uncontrolled river catchment. As an illustration, the proposed methodology was applied to the case of the Ciechowice land dyke pumping station, which is situated on an outlet of the Łęgoń watercourse to the Odra River. During a flood in 2010, extensive areas were flooded including the pumping station building and other technical facilities, and heavy losses were incurred as a consequence.

2. Study Area

The Łęgoń watercourse is a right-bank tributary of the Odra River (Figure 1). The sources of this watercourse are situated at the edge of the Moravian Gate Arboretum at an elevation of 184.25 m a.s.l. The entire watercourse flows through Racibórz County in the Silesian Province. The watercourse catchment is characterized by a fairly dense drainage network with numerous drainage ditches. The Łęgoń watercourse catchment is a lowland used for agriculture and is characterized by small gradients.

The catchment is mainly built from river alluvia. Fluvial soils occupy its eastern part, while in the rest of its area, there are soils formed from old accumulation terrace (weakly argillaceous and argillaceous) sands, clay overlying sands and lightweight boulder clays, and soils formed from loess deposits.

The average annual precipitation for the multiannual period 2003–2012 in the catchment amounted to 614.3 mm. The average annual evaporation for this period was 564 mm. The annual climatic water balance (CWB) amounted to 50.3 mm. The average annual temperature on the basis of the climate atlas of Poland is 8.5 °C. The conditions prevailing in the Łęgoń river catchment, especially the small river bed slope, tend to favour the formation of high overbank flows.

When flume stage conditions occur in the Odra river, waters of the Łęgoń river gravitationally flow through two 1.20 m × 1.40 m dyke culverts. At high water in the Odra river dyke, culverts are closed by manoeuvrable flaps and water is then pumped with the Ciechowice dyke pumping station from the retarding reservoir to the Odra River. According to Polish technical regulations, the Ciechowice dyke pumping station should be built with regards to the designed and control flood flows, which should be estimated with the probability of exceedance of 1% and 0.3%, respectively. The reinforced concrete main building consists of underground intake wells and an aboveground pump house with four two-stage vertical-axis vortex impeller pumps, each of which has a capacity of $0.9 \text{ m}^3 \cdot \text{s}^{-1}$ ($3200 \text{ m}^3 \cdot \text{h}^{-1}$). Upstream of the pumping station, there is a retarding reservoir in the form of a widened (for a length of about 100 m) Łęgoń watercourse channel. The retarding reservoir is about 15 m wide at its bottom.

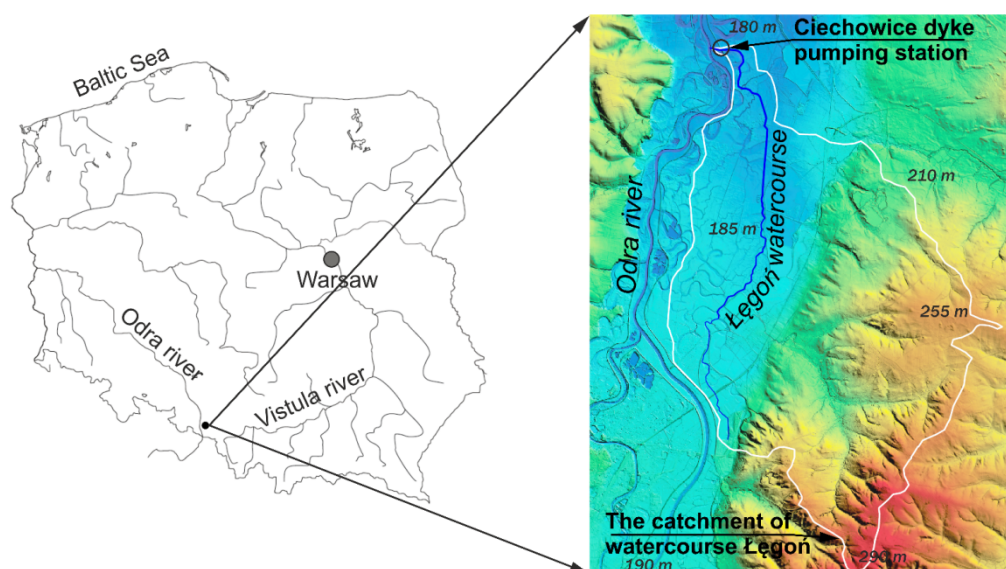


Figure 1. Location of the study area (Łęgoń watershed) and the digital elevation model.

3. Methodologies of Flows Estimation

As the Łęgoń watercourse is an uncontrolled river and there are no water stages and flow measurements, a rainfall-runoff model and a spatial regression equation were used to determine the probable maximum flows. Regional interdependences between unit flows and maximum flows, with a probability of exceedance $p = 1\%$, were developed to verify the calculation results. The values yielded by the rainfall-runoff model are given as the ultimate results of the design and control flood flow calculations due to the fact that the NRCS-CN method is the most commonly used method for estimating peak discharge in a river catchment.

Before estimating the peak discharge in the special watershed, it is better to test the NRCS-CN method with some accurate models to precisely estimate the impact of individual parameters. The sensitivity of the models increases with an increase in the return period. Therefore, calibration of this method for high return periods is necessary [15]. The spatial regression equation and regional interdependences between unit flows and maximum flows were used for such a calibration.

For maximum flows, wave hydrographs must be defined, i.e., shape and volume with the use of a mathematical model of precipitation and drainage in order to analyze the flood wave routing through the retarding reservoir for both the assumed number and efficiency of pumps and also the volume of the reservoir. An optimum solution involves the determination of the maximum retarding reservoir capacity associated with land topography with regards to lower reservoir maintenance costs when compared to a high capacity of a pump station.

3.1. Design Flows—The Spatial Regression Equation

The spatial regression equation allows the maximum flow to be calculated with a definite probability of exceedance in hydrological uncontrolled river catchments with an area of $50 \div 2000 \text{ km}^2$. This equation, which is recommended by the Polish Hydrologists Association, is written in the following form [24]:

$$Q_{\max p\%} = \lambda_p Q_{\max 1\%} \quad (1)$$

where $Q_{\max p\%}$ is the annual maximum flow with a probability of exceedance of $p = 1\%$ in $\text{m}^3 \cdot \text{s}^{-1}$ and λ_p is the quantile for the nondimensional regional curves of the maximal flows, which equals $\lambda_p = 1.224$ for $p = 0.3\%$.

The maximal flow with a probability of exceedance $p = 1\%$ can be calculated using the following equation [23]:

$$Q_{\max p=1\%} = \alpha_{area} A^{0.92} H_1^{1.11} \phi^{1.07} I_r^{0.10} \psi^{0.35} (1 + lake)^{-2.11} (1 + B)^{-0.47} \quad (2)$$

where: $Q_{\max p=1\%}$ is the maximum flow with a probability of exceedance equal to 1% in $\text{m}^3 \cdot \text{s}^{-1}$; α_{area} is the regional parameter of the formula, which is taken on the basis of the investigated area location in Poland and is equal to $\alpha_{area} = 2.992 \times 10^{-3}$; A is the river catchment area that amounts to $A = 49.40 \text{ km}^2$; H_1 is the maximum daily rainfall for a probability of exceedance of $p = 1\%$, which is taken from the meteorological station in Raciborz and is equal to $H_1 = 87.3 \text{ mm}$; and ϕ is the runoff coefficient for peak flows, which is equal to $\phi = 0.55$ and qualified on the basis of "The map of Polish soils". For a catchment with several soil groups with different values of runoff coefficient ϕ , a medium weight should be calculated for the whole catchment.

$$\phi = \frac{1}{A} \sum_{i=1}^{i=n} \phi_i A_i \quad (3)$$

where A_i is an area covered with soil of a given group in km^2 and I_r is the longitudinal river bed slope that is equal to $I_r = 0.73\text{‰}$.

$$I_r = \frac{W_g - W_p}{L + l} \quad (4)$$

where: W_g is the elevation of the watershed in the point crossing the axis of a dry valley of the longest watercourse in m a.s.l.; W_p is the elevation of the calculated cross-section closing the catchment in m a.s.l.; L is the length of the longest watercourse in the catchment in km; l is the length of a dry valley in the lengthened part of the longest watercourse of the catchment in km; and ψ is the average slope of the river catchment, which is equal to $\psi = 17.0\text{‰}$.

$$\psi = \frac{W_{\max} - W_p}{\sqrt{A}} \quad (5)$$

where: $W_{\max}$ is the maximal catchment elevation in m a.s.l.; A is the catchment area in km^2 ; and $lake$ is the lake index of the river catchment.

$$lake = \frac{1}{A} \sum_{i=1}^m A_{lake\ i} \quad (6)$$

where $A_{lake\ i}$ is an area of the $lake\ i$ catchment that is equal to $A_{lake} = 2.5 \text{ km}^2$ and m is the number of lake catchments, which is equal to $m = 1$.

B is the swamp index of the river catchment, which is equal to zero.

$$B = \frac{1}{A} \sum_{i=1}^k A_{B\ i} \quad (7)$$

where $A_{B\ i}$ is an area of i -succeeding swamp or peat land region in km^2 and k is the number of swamp regions.

Details on how to determine the particular variables in Formula (2) can be found in the extensive literature on the subject (e.g., in [24]).

The following physical-geographic parameters of the Łęgoń catchment were assumed in the conducted calculations: a catchment area of 49.40 km^2 , the maximum catchment length of 11.57 km , an average catchment gradient of 1.7‰ , and also a forestation ratio of 15% . Therefore, the maximum

flows to the Ciechowice pumping station, with the specified probability of exceedance calculated using the spatial regression Formula (2), amounted to:

$$Q_{\max 1\%} = 2.992 \times 10^{-3} \cdot 49.4^{0.92} \cdot 87.3^{1.11} \cdot 0.73^{0.10} \cdot 17^{0.35} (1 + 0.0506)^{2.11} \cdot 1 = 19.20 \text{ m}^3 \text{ s}^{-1}$$

$$Q_{\max 0.3\%} = Q_{\max 1\%} \lambda_p = 19.20 \cdot 1.224 = 23.50 \text{ m}^3 \text{ s}^{-1}$$

3.2. The NRCS-CN Method

Precipitation with a probability of occurrence of $p = 1\%$ and 0.3% during $T = 24$ h was assumed in the calculations. The precipitation distribution over time for the Łęgoń catchment was determined using the method proposed by German Association For Water Resources and Land Improvement DVWK [25]. According to the recommendations of this method, rain with a maximum intensity in the middle of its duration should be assumed as the reliable precipitation intensity distribution. The following precipitation distribution was assumed in the calculations and expressed as a percentage of the total precipitation:

Fraction of precipitation time [h]	$(0-0.3)T$	$(0.3-0.5)T$	$(0.5-1.0)T$
Percentage of total precipitation	20	50	30

The total precipitation hyetograph can also be expressed by a beta distribution, the density function of which is described by Formula (8):

$$P(t) = \frac{t^{\alpha-1}(1-t)^{\beta-1}}{B(\alpha, \beta)}, \quad 0 \leq t \leq T, \alpha > 0, \beta > 0 \quad (8)$$

where: $P(t)$ is the total precipitation in mm; t is precipitation duration in hours; α and β are distribution parameters of beta function $B(\alpha, \beta)$; and T is the total time of precipitation in hours.

When creating a model for the division of the daily precipitation total into 1 h calculation intervals, such beta distribution parameters were selected in order to maintain agreement with the DVWK method assumptions [25], i.e., $\alpha = 4.5$, $\beta = 6.1$. The values of the maximum daily precipitation totals with occurrence probability of $p = 1\%$ and $p = 0.3\%$ are equal to 87.3 and 101.6 mm accordingly and were obtained from the Racibórz rainfall gauging station.

Daily rainfalls with a probability of exceedance of 0.3% and 1% , which were estimated on the basis of measured values from the period of 1998–2013, were set in a series and their homogeneity was verified with the use of the Mann–Kendall test. For these series, the parameters of log-norm distribution were estimated using the maximal reliability method and their correctness was verified with the Kolmogorow test. Using this distribution, the values of the required quantiles were obtained. Uncertainty was calculated as the upper limit of one-sided confidence interval $\beta = 84\%$ of $P_{\max, p}^{\beta}$. In the performed calculations, the value of uncertainty for probability of 1% and 0.3% were equal to 96.5 mm and 116.1 mm, which is 10.5% and 14.3% , respectively. The calculations were performed by the Polish Institute of Meteorology and Water Management.

The effective precipitation was determined using the NRCS-CN method [9–11]. According to this method, the following factors have a bearing on effective precipitation: the land development, the kind of soils, the kind of vegetation cover, and the catchment wetness (wetness degree II was assumed). The totality of the factors was expressed by a dimensionless parameter CN , assuming values that ranged from 0 to 100. The NRCS-CN method is based on the hypothesis that the ratio of cumulative effective precipitation (direct runoff) P_e to cumulative total precipitation P minus initial losses I_{α} is equal to the ratio of current cumulative infiltration F to maximum potential catchment storage S :

$$\frac{P_e}{P - I_{\alpha}} = \frac{F}{S} \quad (9)$$

Water balance is defined by:

$$P = I_{\alpha} + F + P_e \quad (10)$$

Substituting F from Equation (9) into Equation (10) results in the following cumulative effective precipitation equation:

$$P_e = \frac{(P - I_\alpha)^2}{P - I_\alpha + S} \quad (11)$$

where: P_e is the effective precipitation in mm; P is the total precipitation in mm; I_α is the initial abstraction in mm; S is the potential catchment storage in mm; and F is the current infiltration in mm after runoff begins.

The effective precipitation amount totalized over time from 0 (the beginning of precipitation) to t (the current instant) is

$$P_e(t) = \begin{cases} 0 & \text{for } P(t) - \lambda \cdot S \leq 0 \\ \frac{(P(t) - \lambda \cdot S)^2}{P(t) + (1 - \lambda) \cdot S} & \text{for } P(t) - \lambda \cdot S > 0 \end{cases} \quad (12)$$

where: P_e and P are the effective and the total precipitation amounts totalized over time from 0 to T [mm], λ is an empirical coefficient contingent on the CN (curve number) parameter, which is normally assumed as a constant value of 0.2 in order for S to be the only parameter of the method; and S is the maximum catchment storage [mm].

From the Verma investigations [15], the coefficient λ can have several values that range from 0.05 to 0.3, although it can theoretically be greater than 1.0. This coefficient is of regional character and, as mentioned earlier, depends on many parameters: the region's characteristics and soil, land use, and also climate conditions. If rainfall and runoff investigations are available, the value of λ can be estimated on their basis and then used in the design process. The original value $\lambda = 0.2$ was specified for small agriculture catchments. However, later investigations [26] showed that smaller values of λ , e.g., $\lambda = 0.05$, give better results for the river catchment for which the investigations were conducted. Despite this, a value $\lambda = 0.2$ is still applied in many practical applications [27].

Chung et al. in paper [28] showed the possibility of calculating the λ parameter from the relation $\lambda = 0.2/\varepsilon$. In this relation, ε only depends on the two parameters α and β , which can be calculated if the following, are given: t_p is the ponding time, K_s is the saturated permeability coefficient of soil, t is the rainfall duration, and p is the rainfall intensity. Since the last three variables can be measured easily and t_p is determined by any adequate formula, the uncertainty of the adopted infiltration models is reduced to a minimum.

The maximum catchment storage is given by relation (13):

$$S = 25.4 \cdot \left(\frac{1000}{CN} - 10 \right) \quad (13)$$

According to the assumptions of the NRCS-CN method, a flood begins when the amount of precipitation exceeds the height of the layer of water retained by the processes of interception, surface storage, and infiltration before a surface runoff starts. The amount of precipitation which is involved in the above processes is referred to as initial abstraction I_α . Parameter CN is determined as a weighted average for the whole catchment area on the basis of the adopted soil group, the catchment use, and the hydrologic conditions (14):

$$CN = \frac{1}{A} \sum_{i=1}^n A_i \cdot CN_i \quad (14)$$

where: A is the catchment area in km²; CN_i is the characteristic values of the particular areas A_i ; and n is the number of homogenous areas.

The types of soils in the Łęgoń watercourse catchment area were identified on the basis of a soil-agricultural map with a scale of 1:50,000 and also the division of soils included in [29]. This enabled the NRCS-CN method to be directly applied for Polish conditions. The catchment soils were assigned to the types of soils included in the map and aggregated. They were then assigned to one of the four soil groups (A, B, C, D) depending on the potential surface runoff formation conditions

stemming from the soils' permeability. Land use classes were determined using the CORINE Land Cover database and a land cover map, which is presented in Figure 2. Three soil groups (Table 1) and seven land uses (Table 2) of the catchment area were distinguished. The CN parameter for the whole catchment amounted to 71. This value was adopted in order to create effective precipitation hyetographs using Equation (12), which are presented in Figure 3.

Table 1. Information used for the selection of CN values.

Soil	% Area	Hydrologic Soil Group
Sediments	12.8	A
Peat	13.58	C
Sand	31.39	B
Sandy loam	28.9	C
Loess and silt	8.23	B
Water	5.1	-

Table 2. Land use information of the Łęgoń watershed.

Type	A (km ²)	% Area
Agricultural areas	29.37	59.45
Grasslands	1.71	3.47
Mixed forest	8.22	16.65
Coniferous forest	1.44	2.92
Settlement	6.15	12.44
Water	2.50	5.07
Total catchment area	49.4	100

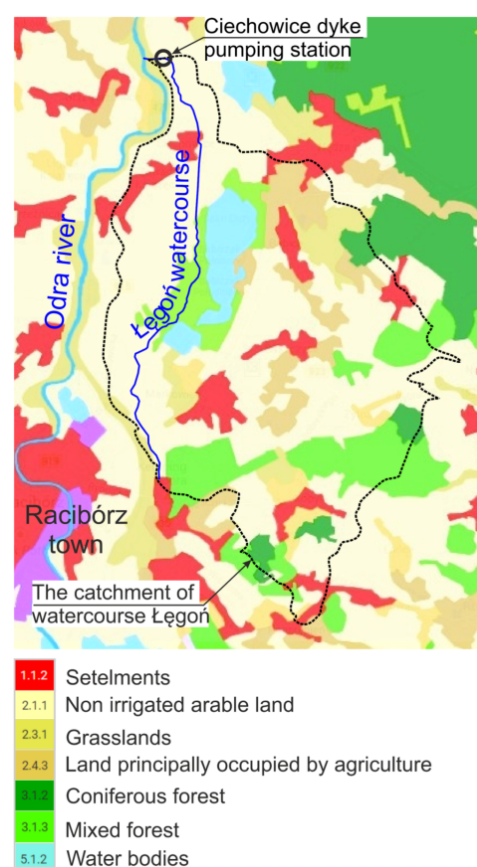


Figure 2. Land use classes of the Łęgoń catchment according to the CORINE Land Cover Map.

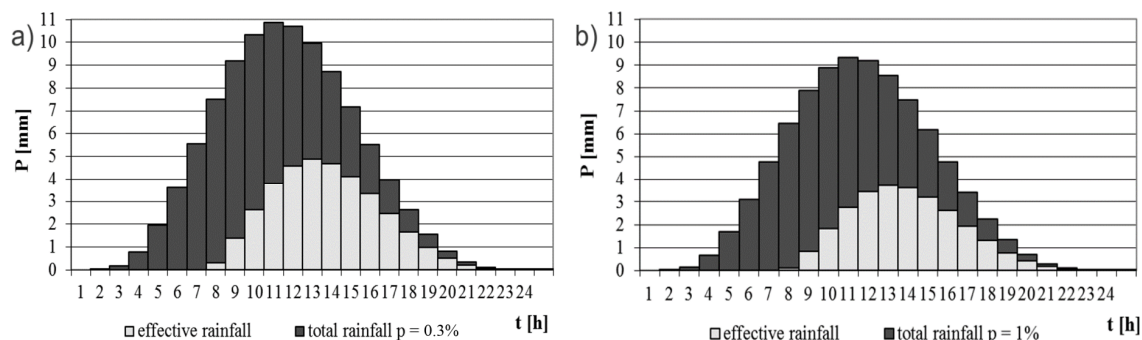


Figure 3. Hyetographs of the total and effective precipitation in the Łęgoń watercourse catchment for (a) $p = 0.3\%$, (b) $p = 1\%$.

3.3. Determination of Wave Hydrographs

Hydrographs of the designed and control flood waves for the Łęgoń watercourse were developed using a rainfall-runoff model, which enabled instantaneous unit hydrographs (IUHs) to be determined [30]. The unit hydrograph shows the course of a flood wave (the catchment response) caused by effective precipitation that amounts to one unit and lasts for Δt . The parameters of the flood models were determined using Nash's conceptual model [19,21]. The Nash model shows the transforming action of the catchment, i.e., the transformation of the effective precipitation unit into an instantaneous unit hydrograph of the immediate runoff, which is in the form of a reservoir cascade with linear characteristics. The instantaneous unit hydrograph is the two-parameter gamma distribution probability density function:

$$u(t) = \frac{1}{k\Gamma(N)} \cdot \left(\frac{t}{k}\right)^{N-1} e^{-t/k} \quad (15)$$

where: $u(t)$ is the instantaneous unit hydrograph ordinates in h^{-1} ; t is the time from the coordinate system origin in h; k is the reservoir storage parameter in h determined using the empirical Formula (16); N is the number of reservoirs in the Nash model $N = LAG/k$, where runoff lag time (delay between the culmination of precipitation and the peak of the flood wave) is determined using Formula (17); and $\Gamma(N)$ is the gamma function that is equal to $(N - 1)!$.

$$k = 0.56A^{0.39}(1 + U)^{-0.62}P_e^{-0.11}D^{0.22} \quad (16)$$

$$LAG = 1.28A^{0.46}(1 + U)^{-1.66}P_e^{-0.27}D^{0.37} \quad (17)$$

where: A is the catchment area in km^2 ; U is the fraction of impermeable surfaces in the whole catchment; P_e is the amount of effective precipitation in mm; and D is the duration of effective precipitation in h. The time of growth amounts to $t_p = (N - 1)k$ and the maximum of the unit hydrograph amounts to

$$u_p = \frac{1}{k\Gamma(N)} \cdot \frac{(N - 1)^{N-1}}{e^{(N-1)}} \quad (18)$$

The instantaneous unit hydrographs are the basis for determining flood-wave hydrographs. The ordinates of the surface runoff hydrographs can be calculated using Equation (19):

$$Q_i = \sum_{j=1}^{\min(i,n)} h_k \cdot \Delta P_{e,j}, \quad k = i - 1 + j, \quad i = 1, 2, \dots, m + n - 1 \quad (19)$$

where: ΔP_{ej} is the partial effective precipitation in time interval j in mm; m is the number of the unit hydrograph ordinates; n is the number of effective precipitation time intervals; and h_k is the unit hydrograph ordinates in $\text{m}^3 \cdot \text{s}^{-1} \cdot \text{mm}^{-1}$ calculated using Equation (20), or if $t_p > 3\Delta t$ using Equation (21):

$$h_k = \frac{A}{3.6\Delta t} \int_{t-\Delta t}^t u(\tau) d\tau, \quad t = k\Delta t, \quad k = 1, 2, \dots, m \quad (20)$$

$$h_k = \frac{A}{3.6} \cdot \frac{1}{2} [u(t) + u(t - \Delta t)], \quad t = k\Delta t, \quad k = 1, \dots, m \quad (21)$$

The designed and control flood-wave hydrographs for the Łęgoń watercourse calculated by the IUH method are shown in Figure 4. The peak flows amount accordingly to $Q_{1\%} = 18.75 \text{ m}^3 \cdot \text{s}^{-1}$ and $Q_{0.3\%} = 24.8 \text{ m}^3 \cdot \text{s}^{-1}$.

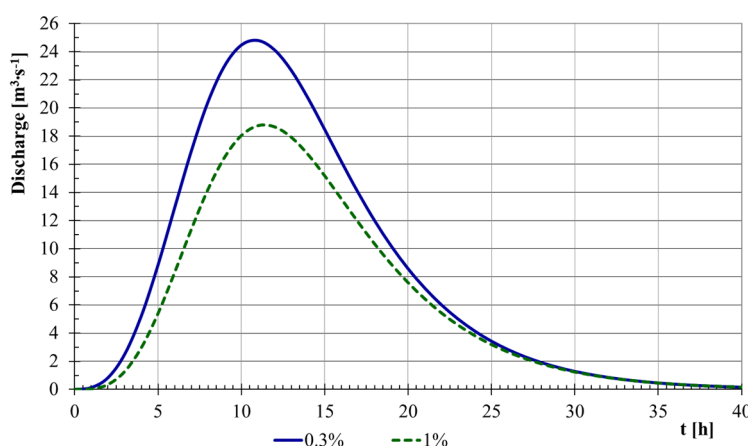


Figure 4. Hydrographs of the designed and control flood waves for $p = 1\%$ and 0.3% .

The volume of the hypothetical designed and control flood waves calculated on the basis of Equation (19) amounts to $V_m = 0.923 \text{ M m}^3$ and $V_k = 1.182 \text{ M m}^3$, respectively.

3.4. Flood Waves Routing

The flood waves routing was carried out in order to calculate the required volume of the retarding reservoir and to determine its effect on the capacity of the pumping station. Another aim of the calculations was to determine the maximum water storage levels in the reservoir and the reduced water runoffs. A method that uses the continuity equation [31,32] in a differential form (22) was selected from many methods for reservoir flood wave routing:

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = 0 \quad (22)$$

where: Q is the water flow in $\text{m}^3 \text{ s}^{-1}$; x is a coordinate consistent with the direction of the water flow in m; A is the surface area of the flow in m^2 ; and t is the time in s.

Having been integrated from x_1 to x_2 and appropriately transformed, Equation (22) assumes the form of (23) in which the value of the integral expresses the volume of the water stored in the reservoir or that discharged from the reservoir:

$$Q(x_1) - Q(x_2) + \frac{d}{dt} \int_{x_1}^{x_2} A dx = 0 \quad \Rightarrow \quad Q(x_1) = Q(x_2) + \frac{\Delta V}{\Delta t} \quad (23)$$

where: x_1 coordinates at the beginning of the reservoir; x_2 coordinates at the end of the reservoir; $Q(x_1)$, $Q(x_2)$ represent the flow of water into and out of the reservoir in $\text{m}^3 \text{s}^{-1}$, respectively; and $\Delta V/\Delta t$ is the change in water volume in the reservoir over time.

3.5. Methodology Summary

Below, the authors present a methodology of proceeding when making a correct selection of the capacity of a dyke pumping station, which collaborates with a retarding reservoir with an adequately selected capacity.

1. Selecting the methodology for estimating calculated flows with a specified probability of exceedance, which depends on whether the catchment is being controlled or not.
2. Conducting hydrological calculations for designing flows estimation.
3. Generating hydrographs to calculate flows.
4. Evaluating terrain conditions in the localization of a dyke pumping station, which enables a retarding reservoir with a determined capacity and parameters to be constructed.
5. Selecting the capacity of pumps in the dyke pumping station.
6. Selecting computational scenarios that consider the variable capacity of pumps and the variable volume of a retarding reservoir.
7. Estimating the functioning conditions of a dyke pumping station and a retarding reservoir during the occurrence of a flood wave.
8. Estimating the costs of construction, exploitation, and maintenance for a dyke pumping station and a retarding reservoir with regards to different computational scenarios.

4. Pump Station Capacity Assumptions

The specified flood wave hydrographs, the retarding reservoir capacity curves, and the pumping station capacity characteristics were used to calculate several computational cases of flood waves passing through the reservoir. The calculations were performed with regards to the following assumptions:

- in each of the calculation cases, the retarding reservoir bottom is situated at 174.60 m a.s.l.,
- the retarding reservoir side slope is 1:2,
- the normal drainage level is 176.30 m a.s.l.,
- the maximum allowable water level in the upper dyke areas is 178.00 m a.s.l.,
- the dyke culvert flaps are closed (there is a flood on the main river),
- the initial water level in the retarding reservoir is situated at 176.00 m a.s.l.,
- the pump switching on/off levels: I-176.30/175.80; II-176.70/176.20; III-177.10/177.60; IV-177.50/177.00.

The calculation cases were as follows:

- Case 1: current (existing) state, retarding reservoir bottom dimensions of 15×100 m, pumps' output of $4 \times 0.9 = 3.60 \text{ m}^3 \text{s}^{-1}$, design flood wave routing;
- Case 2: current state, retarding reservoir bottom dimensions of 15×100 m, pumps' output after alteration of $4 \times 1.05 = 4.20 \text{ m}^3 \text{s}^{-1}$, design flood wave routing;
- Case 3: a total pumping station capacity of $18.75 \text{ m}^3 \text{s}^{-1}$, four pumps as is the case for the current state, four additional pumps with a capacity of $15.15 \text{ m}^3 \text{s}^{-1}$, retarding reservoir bottom dimensions of 45×240 m, design flood wave routing;
- Case 4: a total pumping station capacity of $18.75 \text{ m}^3 \text{s}^{-1}$, four pumps as is the case for the current state, four additional pumps with a total capacity of $15.15 \text{ m}^3 \text{s}^{-1}$, retarding reservoir bottom dimensions of 45×240 m, control flood wave routing;

- Case 5: a total pumping station capacity of $18.75 \text{ m}^3 \text{ s}^{-1}$, four pumps after alteration $4 \times 1.05 = 4.20 \text{ m}^3 \text{ s}^{-1}$, four additional pumps with a capacity of $14.55 \text{ m}^3 \text{ s}^{-1}$, retarding reservoir bottom dimensions of $45 \times 240 \text{ m}$, design flood wave routing;
- Case 6: a total pumping station capacity of $15.0 \text{ m}^3 \text{ s}^{-1}$, four pumps, retarding reservoir bottom dimensions of $45 \times 240 \text{ m}$, design flood wave routing;
- Case 7: a total pumping station capacity of $15.0 \text{ m}^3 \text{ s}^{-1}$, four pumps, retarding reservoir bottom dimensions of $55 \times 910 \text{ m}$, design flood wave routing;
- Case 8: a total pumping station capacity of $8.0 \text{ m}^3 \text{ s}^{-1}$, four pumps, retarding reservoir bottom dimensions of $100 \times 1700 \text{ m}$, design flood wave routing;
- Case 9: a total pumping station capacity of $6.0 \text{ m}^3 \text{ s}^{-1}$, four pumps, retarding reservoir bottom dimensions of $100 \times 2150 \text{ m}$, design flood wave routing.

5. Calculation Results and Discussion

The results of numerical simulations of the designed flood wave routing through the retarding reservoir for the particular cases, including the capacity of the reservoir and its bottom dimensions, the maximum water level elevation, the retarding reservoir filling level, and the capacity of the pumps, are presented in Table 3. Selected cases are also graphically presented in Figure 5a–f.

Table 3. Results of wave transformation calculations with regards to the assumed design conditions.

Case	Bottom Dimensions [m × m]	Reservoir Capacity [m^3]	Reservoir Depth [m]	Max. Water Elevation in the Reservoir [m a.s.l.]	Capacity of the Pumping Station [$\text{m}^3 \text{ s}^{-1}$]
1	15 × 100	7412	-	above 178.00	$4 \times 0.9 = 3.60$
2	15 × 100	7412	-	above 178.00	$4 \times 1.05 = 4.20$
3	45 × 240	31,724	2.63	177.23	$4 \times 0.9 + 15.16 = 18.76$
4	45 × 240	-	-	above 178.00	$4 \times 0.9 + 15.16 = 18.76$
5	45 × 240	31,591	2.62	177.22	$4 \times 1.05 + 14.46 = 18.76$
6	45 × 240	-	-	above 178.00	$4 \times 3.75 = 15.00$
7	55 × 910	191,209	3.40	178.00	$4 \times 3.75 = 15.00$
8	100 × 1700	617,304	3.40	178.00	$4 \times 2.0 = 8.00$
9	100 × 2150	778,266	3.39	177.99	$4 \times 1.5 = 6.00$

An analysis of the results, which aimed to determine the required capacity of the retarding reservoir and the required capacity of the pumping station, showed that it is not possible to safely route the designed flood wave through the pumping station in its current state, i.e., the existing retarding reservoir and the pumps' total output amounting to $3.60 \text{ m}^3 \text{ s}^{-1}$ (Case 1). The capacity of the retarding reservoir is insufficient to reduce the designed flood wave—the water level significantly exceeds the maximum permissible level of 178.00 m by 0.50 m. (Figure 5a). Similar results were obtained in the calculations of flood waves passing through a retarding reservoir while taking into account the change in the pumps' capacity to the total capacity of $4.20 \text{ m}^3 \text{ s}^{-1}$ (Case 2).

The designed flood wave can be routed through the retarding reservoir if the latter's capacity is increased to about $32,000 \text{ m}^3$ and the pumping station capacity is increased to the capacity determined by the hydrological calculations in both Case 3 (Figure 5b) and Case 5 (Figure 5d).

During the control flood wave routing (Case 4), the maximum permissible water level can be exceeded up to the level of 178.50. This is due to the fact that the control flow is treated as a verifying flow, and it is actually the designed flow that is used to dimension hydraulic structures. The verifying calculations carried out to determine the effect of the reduction in the total capacity of the pumps and the corresponding readjustment of the retarding reservoir capacity (Cases 6–9) showed that each reduction in the capacity of the pumping station results in an increase in the capacity of this reservoir and its dimensions (Figure 5f–h).

The results of the calculations clearly confirmed the assumptions made in design Case 5, i.e., the best solutions are: the total capacity of the pumps— $18.75 \text{ m}^3 \text{ s}^{-1}$ with the replacement of the

pumps in the existing pumping station in order to obtain a total capacity of $4 \times 1.05 = 4.20 \text{ m}^3 \text{ s}^{-1}$, and the building of another pumping station in order to obtain a capacity of $14.55 \text{ m}^3 \text{ s}^{-1}$. The required retarding reservoir capacity is $31,590 \text{ m}^3$ and its bottom dimensions are $45 \times 240 \text{ m}$. Moreover, Case 3, which can be executed at a lower cost, can also be considered as it does not require the reconstruction of the existing pumping station, but only requires the replacement of the pumps due to their condition.

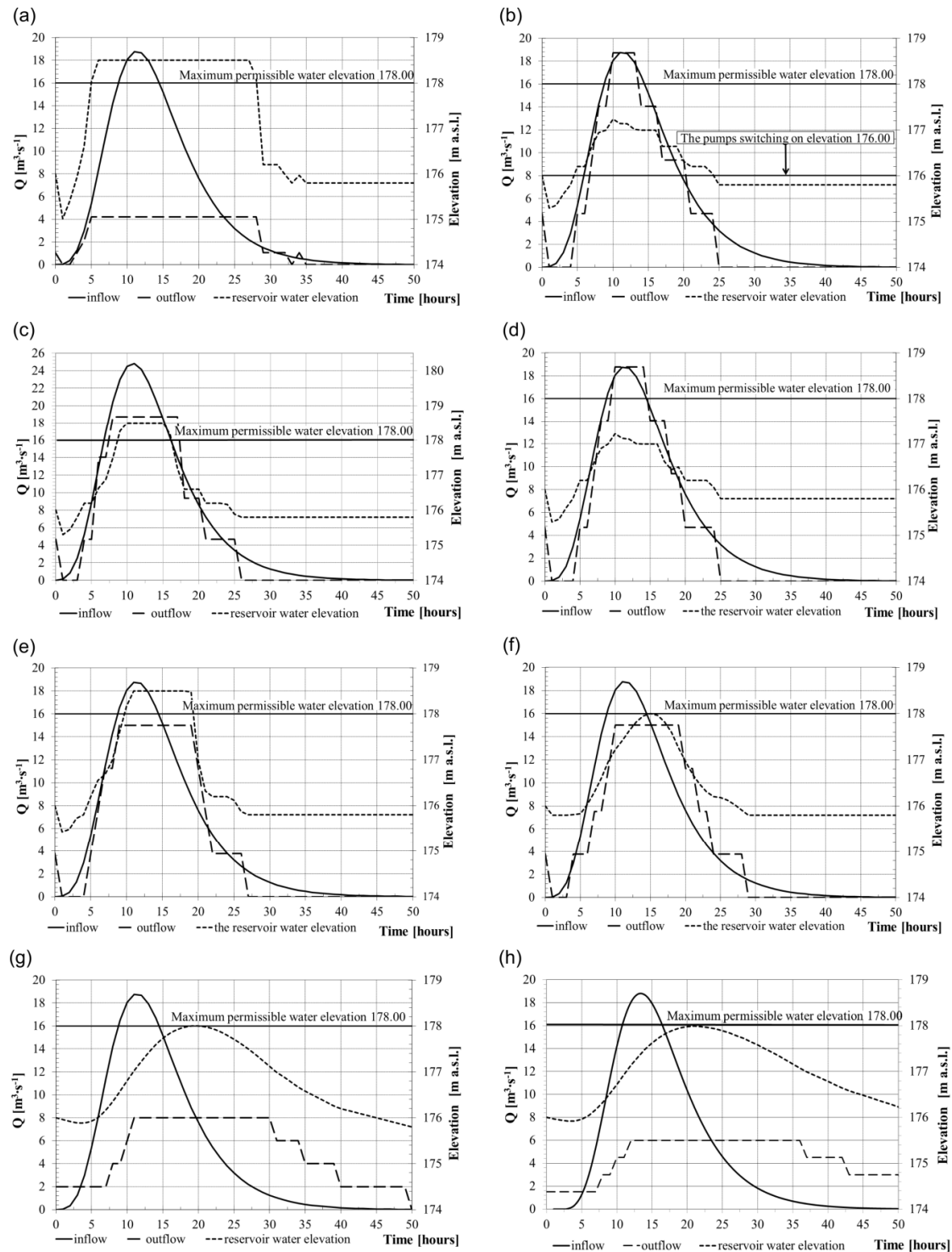


Figure 5. Flood wave transformation in (a) Case 1; (b) Case 3; (c) Case 4; (d) Case 5; (e) Case 6; (f) Case 7; (g) Case 8; and (h) Case 9.

6. Conclusions

This paper presents the authors' methodology that was proposed in order to select a correct solution for a dyke pumping station, the functioning conditions of which were strictly connected with the choice of the capacity of the installed pumps or retarding reservoir. The correct solution should consider exploitation and maintenance costs. The proposed methodology can be used in the design procedure of the other hydroengineering structures. It can also be used in order to estimate flood wave routing through dry and wet storage reservoirs. The presented methodology of proceeding when making a correct selection of a pumping station in order to secure areas that are located above it can be successfully used in the design of wet or dry flood protection reservoirs to protect the areas located below.

The Łęgoń watercourse is an uncontrolled river, and therefore it was not possible to validate the flows that were estimated using the rainfall-runoff method. For validation, calculations of the flows were performed using the spatial regression method that is recommended by the Polish Hydrologists Association. The authors obtained small differences that were within the permissible limits, which were equal to -2.4% and $+5.2\%$ for the probability of 1% and 0.3%, respectively.

The purpose of every dyke pumping station is to transfer the water of watercourses, the outlets of which are cut off from the main river by the river embankment during high river flows. The transfer capacity of a pumping station should be determined on the basis of hydrological calculations so that no hazard of flooding the areas over the dyke is created. A methodology for calculating the capacity of a pumping station, illustrated using the case of an existing pumping station on the Łęgoń watercourse, was presented. The proposed methodology requires the calculation of the rate of the designed (dimensioning) and control flows and then the determination of the hydrographs of hypothetical waves using the mathematical rainfall-runoff model for each hydraulic structure. For this purpose, the authors used the IUH model and then calculated: the total precipitation, the effective precipitation, the instantaneous unit hydrograph, and the direct (surface and subsurface) runoff hydrograph.

The hydrological analysis and the numerical simulations carried out for the Łęgoń river catchment showed that:

1. The designed flood wave of $p = 1\%$ cannot be safely passed through the pumping station for the existing specifications—the total pumping station capacity of $3.60 \text{ m}^3 \text{ s}^{-1}$ and the retarding reservoir capacity of 7412 m^3 .
2. On the basis of computer simulations, it should be indicated that the solution that involves reducing the required pumping station capacity leads to an increase in the required retarding reservoir capacity. The approach to ensure safe operation of the pumping station should not only take into consideration the capacity of the pumping station and volume of the retarding reservoir, but also the retention capacity of the riverbed. This will reduce the cost of rebuilding an existing facility.
3. On the basis of the performed analysis, it was decided that Case 5 is the most beneficial for reconstructing the dyke pumping station, and in this case, the designed flood wave of $p = 1\%$ can be safely passed through the Ciechowice pumping station at the total pumping station capacity of $18.75 \text{ m}^3 \text{ s}^{-1}$ and the retarding reservoir capacity of $31,590 \text{ m}^3$.

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References

1. Szolgay, J.; Gaál, L.; Bacigál, T.; Kohnová, S.; Hlavčová, K.; Výleta, R.; Parajka, J.; Blöschl, G. A regional comparative analysis of empirical and theoretical flood peak-volume relationships. *J. Hydrol. Hydromech.* **2016**, *64*, 367–381. [\[CrossRef\]](#)
2. Grimaldi, S.; Petroselli, A. Do we still need the Rational Formula? An alternative empirical procedure for peak discharge estimation in small and ungauged basins. *Hydrol. Sci. J. J. Sci. Hydrol.* **2015**, *60*, 67–77. [\[CrossRef\]](#)
3. Wan, Y.; Konyha, K. A simple hydrologic model for rapid prediction of runoff from ungauged coastal catchments. *J. Hydrol.* **2015**, *528*, 571–583. [\[CrossRef\]](#)
4. Castiglioni, S.; Lombardi, L.; Toth, E.; Castellarin, A.; Montanari, A. Calibration of rainfall-runoff models in ungauged basins: A regional maximum likelihood approach. *Adv. Water Resour.* **2010**, *33*, 1235–1242. [\[CrossRef\]](#)
5. Hughes, D.A.; Slaughter, A. Daily disaggregation of simulated monthly flows using different rainfall datasets in Southern Africa. *J. Hydrol. Reg. Stud.* **2015**, *4*, 153–171. [\[CrossRef\]](#)
6. Hughes, D.A. Simulating temporal variability in catchment response using a monthly rainfall-runoff model. *Hydrol. Sci. J. J. Sci. Hydrol.* **2015**, *60*, 1286–1298. [\[CrossRef\]](#)
7. Hughes, D.A.; Slaughter, A. Disaggregating the components of a monthly water resources system model to daily values for use with a water quality model. *Environ. Model Softw.* **2016**, *80*, 122–131. [\[CrossRef\]](#)
8. Jiang, Y.; Liu, C.; Li, X.; Liu, L.; Wang, H. Rainfall-runoff modelling, parameter estimation and sensitivity analysis in a semiarid catchment. *Environ. Model Softw.* **2015**, *67*, 72–88. [\[CrossRef\]](#)
9. Soulis, K.X.; Valiantzas, J.D.; Dercas, N.; Londra, P.A. Investigation of the direct runoff generation mechanism for the analysis of the SCS-CN method applicability to a partial area experimental watershed. *Hydrol. Earth Syst. Sci.* **2009**, *13*, 605–615. [\[CrossRef\]](#)
10. Ajmal, M.; Waseem, M.; Ahn, J.-H.; Kim, T.-W. Improved Runoff Estimation Using Event-Based Rainfall-Runoff Models. *Water Resour. Manag.* **2015**, *29*, 1995–2010. [\[CrossRef\]](#)
11. Soulis, K.X.; Valiantzas, J.D. SCS-CN parameter determination using rainfall-runoff data in heterogeneous watersheds—The Two-CN system approach. *Hydrol. Earth Syst. Sci.* **2012**, *16*, 1001–1015. [\[CrossRef\]](#)
12. Jiao, P.; Xu, D.; Wang, S.; Yu, S.; Han, S. Improved SCS-CN Method Based on Storage and Depletion of Antecedent Daily Precipitation. *Water Resour. Manag.* **2015**, *29*, 4753–4765. [\[CrossRef\]](#)
13. Leopardi, M.; Scorzini, A.R. Effects of wildfires on peak discharges in Watersheds. *iForest-Biogeosci. For.* **2015**, *8*, 302–307. [\[CrossRef\]](#)
14. Taher, T.M. Integration of GIS Database and SCS-CN Method to Estimate Runoff Volume of Wadis of Intermittent Flow. *Arab. J. Sci. Eng.* **2015**, *40*, 685–692. [\[CrossRef\]](#)
15. Verma, S.; Verma, R.K.; Mishra, S.K.; Singh, A.; Jayaraj, G.K. A revisit of NRCS-CN inspired models coupled with RS and GIS for runoff estimation. *Hydrol. Sci. J.* **2017**, *62*, 1891–1930. [\[CrossRef\]](#)
16. Kousari, M.R.; Malekinezhad, H.; Ahani, H.; Zarch, M.A.A. Sensitivity analysis and impact quantification of the main factors affecting peak discharge in the SCS curve number method: An analysis of Iranian watersheds. *Quat. Int.* **2010**, *226*, 66–74. [\[CrossRef\]](#)
17. Fan, F.; Deng, Y.; Hu, X.; Weng, Q. Estimating Composite Curve Number Using an Improved SCS-CN Method with Remotely Sensed Variables in Guangzhou, China. *Remote Sens.* **2013**, *5*, 1425–1438. [\[CrossRef\]](#)
18. Tramblay, Y.; Bouvier, C.; Martin, C.; Didon-Lescot, J.-F.; Todorovik, D.; Domergue, J.-M. Assessment of initial soil moisture conditions for event-based rainfall-runoff modeling. *J. Hydrol.* **2010**, *387*, 176–187. [\[CrossRef\]](#)
19. Nash, J.E. A unit hydrograph study with particular reference to British catchments. *Proc. Inst. Civ. Eng.* **1960**, *17*, 249–282. [\[CrossRef\]](#)
20. Nash, J.E.; Sutcliffe, J.V. River Flow Forecasting through Conceptual Models. Part I—A Discussion of Principles. *J. Hydrol.* **1970**, *10*, 282–290. [\[CrossRef\]](#)
21. Jaiswal, R.K.; Thomas, T.; Galkate, R.V.; Ghosh, N.C.; Lohani, A.K.; Kumar, R. Development of Geomorphology Based Regional Nash Model for Data Scarce Central India Region. *Water Resour. Manag.* **2014**, *28*, 351–371. [\[CrossRef\]](#)
22. Nourani, V. A Comparative Study on Calibration Methods of Nash's Rainfall-Runoff Model to Ammameh Watershed, IRAN. *J. Urban Environ. Eng.* **2008**, *2*, 14–20. [\[CrossRef\]](#)

23. Szilagyi, J. State-Space Discretization of the Kalinin-Milyukov-Nash-Cascade in a Sample-Data System Framework for Streamflow Forecasting. *J. Hydrol. Eng.* **2003**, *8*, 339–347. [[CrossRef](#)]
24. Stachy, J.; Fal, B. The rules of calculation of maximal probable flows. The spatial regression equation. *Gospodarka Wodna* **1987**, *1*, 8–13. (In Polish)
25. DVWK. *Ermittlung der Verdunstung von Land- und Wasserflächen*; Merkblätter zur Wasserwirtschaft: Bonn, Germany, 1996; p. 238.
26. Hawkins, R.H.; Jiang, R.; Woodward, D.E.; Hjelmfelt, A.T.; Van Mullem, J.A.; Quan, Q.D. Runoff Curve Number Method: Examination of the Initial Abstraction Ratio, In Proceedings of the Second Federal Interagency Hydrologic Modeling Conference, Las Vegas, NV, USA, 28 July–1 August 2002; ASCE: Lakewood, CO, USA, 2002.
27. Nigam, G.K.; Sahu, R.K.; Sinha, M.K.; Deng, X.; Singh, R.B.; Kumar, P. Field assessment of surface runoff, sediment yield and soil erosion in the opencast mines in Chirimiri area, Chhattisgarh, India. *Phys. Chem. Earth* **2017**, *101*, 137–148. [[CrossRef](#)]
28. Chung, W.H.; Wang, I.T.; Wang, R.Y. Theory-Based SCS-CN Method and Its Applications. *J. Hydrol. Eng.* **2010**, *15*, 1045–1058. [[CrossRef](#)]
29. Ponce, V.M. *Engineering Hydrology: Principles and Practices*; Prentice Hall: Upper Saddle River, NJ, USA, 1989.
30. Zakizadeh, F.; Malekinezhad, H. Comparison of Methods for Estimation of Flood Hydrograph Characteristics. *Russ. Meteorol. Hydrol.* **2015**, *40*, 12. [[CrossRef](#)]
31. Jain, S.C. *Open-Channel Flow*; John Wiley & Sons Inc.: New York, NY, USA, 2001.
32. Bačová Mitková, V.; Pekárová, P.; Miklánek, P.; Pekár, J. Hydrological simulation of flood transformations in the upper Danube River: Case study of large flood events. *J. Hydrol. Hydromech.* **2016**, *64*, 337–348. [[CrossRef](#)]



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