

Discharge capacity of shaft spillway with a polygonal section: a case study of Djedra dam (East Algeria)

Kahlerras Djillali, Brakeni Abderrezak, Gouryev Alim Petrovic and Beglarova Evelina Sourenevna

ABSTRACT

This paper proposes a new design for the shaft spillway of the Djedra dam (East Algeria). The procedure consists of replacing the circular section with a polygonal configuration of twelve (12) sections. The study of the model was divided into five different versions according to the variants of technical modifications of the water intake funnel; this configuration was tested on a hydraulic model in the laboratory of hydroelectric power plants of the Moscow State University of Environmental Engineering. The use of the polygonal section weir can greatly simplify the formwork of the structure, also the free entry funnel increases the discharge coefficient by 20%, without exposing the head of the weir to the risk of cavitation. In the event of submersion, the adopted design can ensure the crossing of an estimated relative flow with a probability of $P = 0.01\%$, which means reducing the height of the dam by 0.68 m, and thus making the hydraulic structure more efficient economically and more reliable. The experimental model was produced on a 1:60 scale of the prototype, which guarantees self-simulation of the hydraulic phenomena of the Djedra dam, and its final design is judged to be hydraulically satisfactory and recommended for construction.

Key words | discharge capacity, Djedra dam, flow coefficient, polygonal section, shaft spillway

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HIGHLIGHTS

- A new design of the weir of the dam is a polygonal configuration.
- The polygonal section weir simplifies its structure.
- The free entry weir funnel increases the discharge coefficient by 20%.
- In the event of submersion, the weir provides an estimated flow rate of $P = 0.01\%$.
- The experimental model produced on a scale of 1:60 of the prototype guarantees self-simulation of the hydraulic phenomena of the dam.

INTRODUCTION

In order to specify the parameters of the shaft spillway with a polygonal configuration, hydraulic researches were carried out in two stages, a theoretical analysis, and hydraulic tests on the physical model in the laboratory of hydroelectric

power station of Moscow State University of Environmental Engineering.

The first stage consists of the theoretical analysis to specify the technical solutions and the parameters of the spillway and shows that the hydraulic calculation of the weir in a well is determined according to a given flow rate of the contours of the structural elements which ensure the normal system operation. It is necessary to design the

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structure in such a way that under normal conditions of their operation, a stable flow regime was provided (pressure or non-pressure) within the water supply part (tunnel) (Sun *et al.* 2009). Transient, partial-pressure hydraulic modes are allowed only when ensuring the transition from a pressureless regime to a pressure regime and conversely, without the occurrence of significant ripple effects, which should be justified by special calculations and laboratory data hydraulic studies (Slissky 1986; Rozanov *et al.* 1995).

Flood evacuation is often the most important hydraulic problem posed by the construction of a large dam. It is necessary to ensure the passage of abnormally high water bodies with a difference in height equal to the height of the dam, that is to say frequently greater than one hundred meters. For the first time, a theoretical solution for the discharge surface of the mine was obtained (Akhutin 1935). The use of these evacuators is hampered by the development of cavitation phenomena, which are dangerous for the flow path in wells. Eliminating the danger of cavitation phenomena requires a more complicated design or even an increase in cost (Binnie & Wright 1941; Bukhartsev 2016). To reduce construction costs, spillways that require a minimal amount of high-flow construction materials are used. These conditions are satisfied with shaft spillway (Moise *et al.* 1970; Novak *et al.* 2003). To cope with the complexity of the internal drainage surface of the latter, a shaft spillway has been designed for the Djedra dam to facilitate and improve reliability.

The greatest contribution to the development of the theory of constructing the drainage surface of the mine was given by Wagner (Wagner 1956), who experimentally studied the outer and inner surfaces of the jet on a ring spillway with a sharp edge.

To reduce the cost of the shaft spillway of the new dams and reduce the problem of cavitation, as well as to overcome the difficulties during construction, a special design has been developed. It consists of replacing the circular cross-section of the shaft with a polygonal cross-section that ensures the formation of a continuous flow over the entire length of the culvert and simplifies the formwork and reinforcement work under the following circumstances (Guryev 2008, Savic *et al.* 2014). (i) The replacement of the well plan configuration facilitates concreting and reinforcement work and at the same time improves the hydraulic operation of the well discharge surface due to the absence of transverse joints (Novak & Cabelka 1981;

Savic *et al.* 2014). (ii) The realization of the well discharge surface with an elliptical profile in the longitudinal direction made it possible to connect the discharge head of the well water receiving funnel with the lateral surfaces of the connecting elbow thus avoiding the possible cavitation of the well drainage system (Lempérière & Ouamane 2003; Liu *et al.* 2018).

The second stage is intended for hydraulic tests in the laboratory on the basis of a model produced on a scale of 1: 60 of the natural size of the shaft spillway designed for the Djedra dam in Algeria, ensuring self-stimulation hydraulic phenomena. The results obtained satisfied the expected objective of this research, namely, the precision of the value of the discharge flows of probability flood $P = 0.01\%$, and that the shaft spillway operates in the free flow for the whole range of possible operating loads, therefore, the final design of the shaft spillway is considered hydraulically satisfactory for the construction of the dam studied.

MATERIALS AND METHODS

Study area

The Djedra dam is located in the East of Algeria and is intended for long-term regulation of the river flow. The coordinates of its location are $7^{\circ}54'17.305''\text{E}$ $36^{\circ}16'44.064''\text{N}$. Its geographical location is presented in Figure 1.

Site rainfall data

Average annual precipitation for the watershed of wadi Djedra is adopted from the Souk-Ahras meteorological station. The distribution within the year is irregular; in the wet season from September to April it falls to on average 87–90% of the annual sum and in the period from November to March – almost 80%. During certain years, in dry periods, the precipitation is less than 1% of the annual sum.

The average annual precipitation in the study area for the period 1920–2007 is presented in Table 1.

The structure of the dam includes a reinforced concrete dam about 60.0 m high with a crest level of 560.0 m, a weir in a well with a head in polygonal section, a tunnel and a water channel. The detail is presented in Figure 2.

The main hydraulic parameters of the Djedra dam are summarized in Table 2.

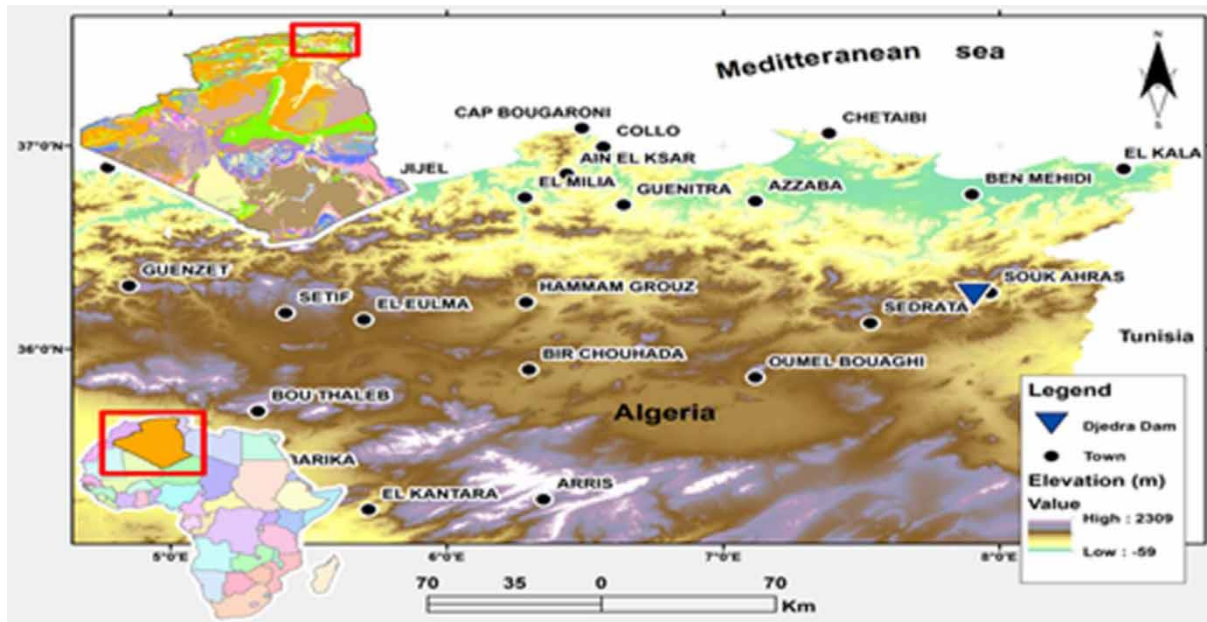


Figure 1 | Localization of the study area.

Table 1 | Average annual precipitation in the study area (period: 1920–2007)

Average annual precipitation (mm)	780					
Coefficient of variation (CV)	0.25					
Coefficient of skewness (CS)	1.50					
Probability (%)	5	25	50	75	90	95
Average annual precipitation (mm) for probability	1,120	905	768	628	530	435

Experimental setup

The test installation was placed in the organic glass pane in the laboratory of the laboratory of hydroelectric power stations of the Moscow State University of Environmental Engineering (Russia); this channel has a bottom with zero slope, a width of 100 cm and a length of 950 cm and is joined to a feeding tank whose dimensions in area are 1.64×2.0 m. The structure of this channel is shown in Figure 3.

From a supply line 1, the water flows into the reception tank 3 through a grid 2. A mirror tank 4 is fixed on the reception tank, whose end is connected with a blind flap 5, which makes it possible to adjust the water level in the tank 4. The water then flows into the tank 6, at the end of which is a

triangular measurement weir 7 with a sharp edge, an angle of 90° cutout and a threshold height $P = 280$ mm. The tank 6 is installed in the tank 8 containing recycled water, located under the laboratory floor. In the initial section of the mirror chute 4, adjacent to the receiving tank 3, was placed the modeled zone of the water zone of the upper dimension 9 adjacent to the weir well. In the simulated section of the upper dam 9, the model comprising a water inlet funnel, the vertical well spillway 10 with the connection elbow, the horizontal tunnel 11, and the discharge channel 12, which are connected by a section of the bed of the channel 13. A piezometer 14 is placed to control the installation of the upper level 3. The height of the measurement weir 7 is determined using a piezometer 15.

Description of the studied models of the selected funnel

The hydraulic research process for the design of this weir was based on 5 alternative versions depending on the technical modifications of the variants of a water intake funnel. A schematic representation of the investigated options is shown in Figure 4.

First variant: free entry; Second variant: entry with guide piers in upstream; Third variant: entry with guide piers in downstream; Fourth variant: guide piers in upstream

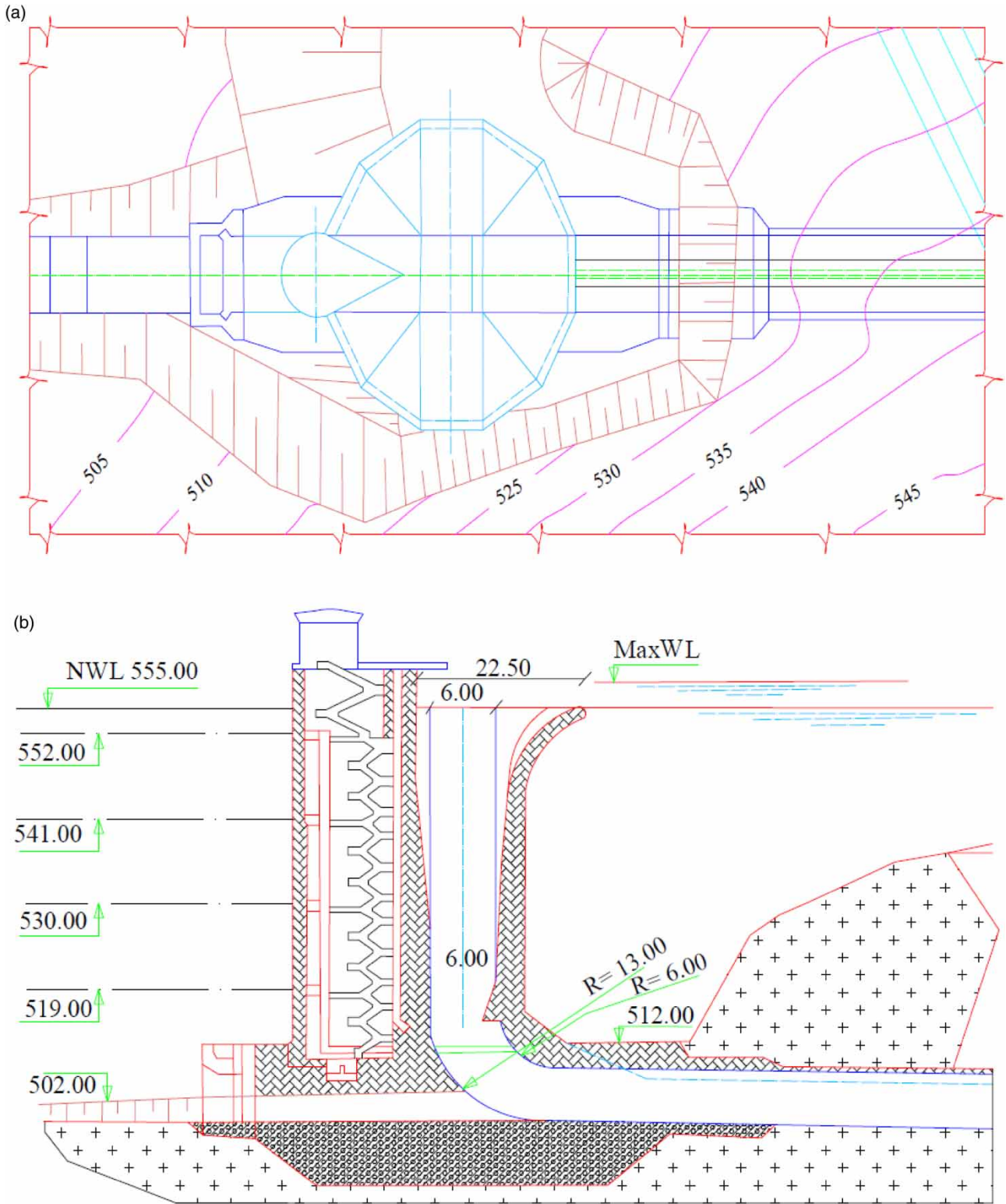


Figure 2 | Site plan (a), and vertical longitudinal section (b) of the spillway of the Djedra dam.

Table 2 | The main features of Djedra dam

Characteristic	Value
Average annual contribution	15 Hm ³ /year
Normal level of restraint	555.00 m
Maximum restraint level rating	558.28 m
Total capacity of restraint	35 Hm ³
Ridge coast	555.00 m
Dam height	60 m
Crest length	425 m
Width in crest	22.50 m

+ shortened chamber; Fifth variant: guide piers in upstream
+ thin wall in downstream.

The shape of the funnel for receiving the water from the spillway into a well is designed in polygonal section, and its dimensions in area are shown in Figure 5.

Instrumentation and measurement techniques

The parameters of the hydraulic regimes of the spillway measured on the model are polyparametric indices depending on the accuracy of the direct measurement of the fixed quantities, as well as on the accuracy of the equipment used.

Measuring needles are installed at piezometers to make the necessary measurements of the levels of the water level upstream and in the weir channel.

The indications of the piezometer of the static level of the Pitot tube with a diameter of 3.0 mm and a diameter of the inlet orifices of 0.3 mm which are used to measure the velocities of the flow, including the charge dynamic range was determined by an inclined micromanometer with nonius measurement to the nearest 0.1 mm.

The model flow was calculated by the formula of the Chipoletti weir flow (Akan 2006).

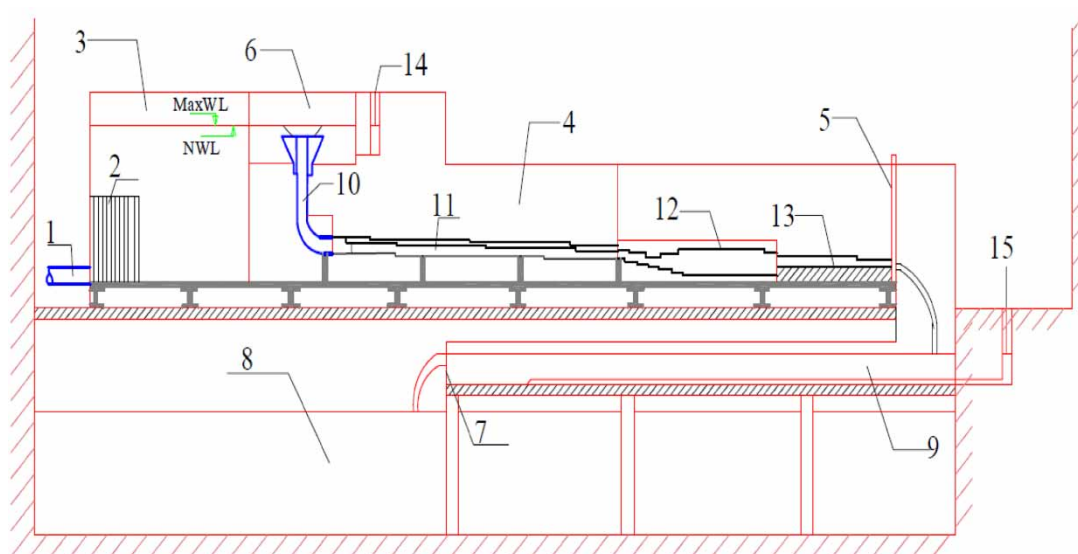
The distribution of flow pressures on the overflow surface was established using piezometers, installed on the surface sectors, in the converging connection section, and on the concave surface of the connection elbow.

The total quantity of piezometers installed on the model of the well spillway is 118, the diameter of the receiving orifices of which is equal to 1 mm, and are connected to piezometric tubes with a diameter of 4 mm.

The depths of flow in the gallery were measured by taking photographs with a digital camera with subsequent recording on a computer, and the visual and instrumental processing of the prints.

The amplitude of the fluctuations in the levels of the free surface of the flow in the gallery was established by direct measurements on the model using the steel ruler with a division value of 0.5 mm.

Unit and total flows were calculated in accordance with the depths and velocities measured.

**Figure 3** | Experimental setup of the polygonal section model.

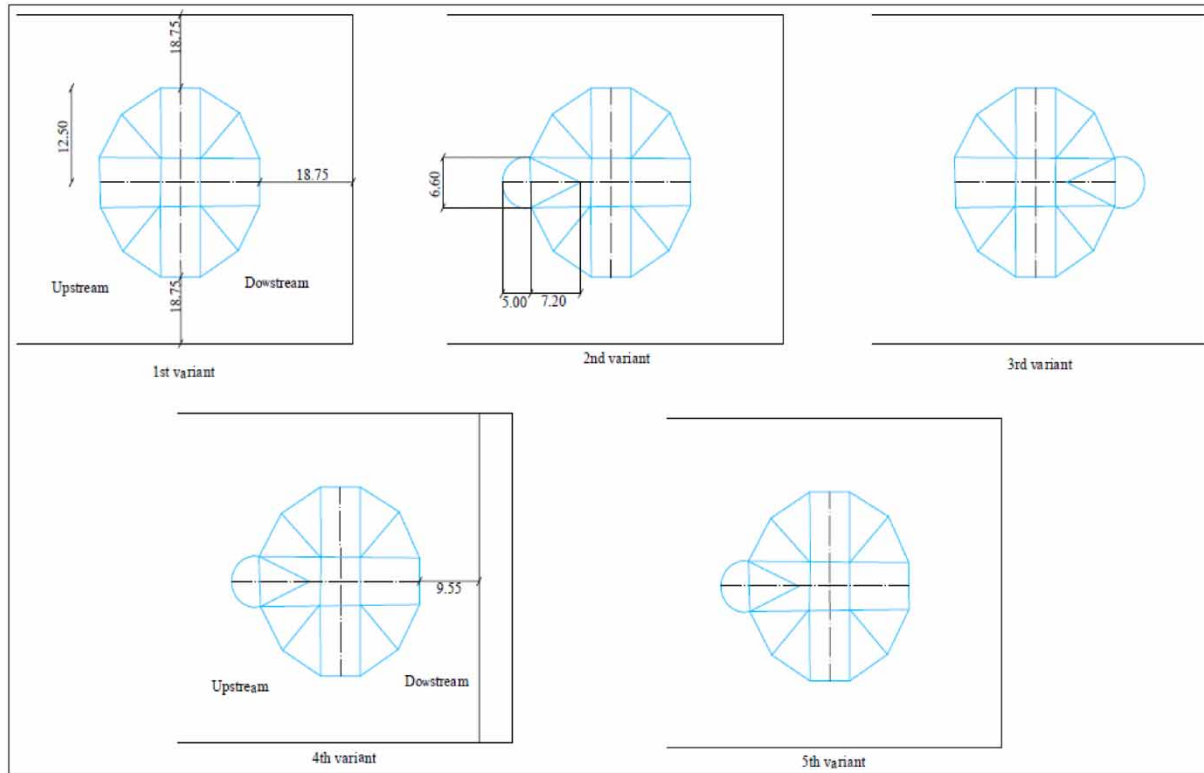


Figure 4 | Schematic representation of the investigated variants of a water intake funnel.

During the manufacture of the model and its installation, the accuracy of the dimensions is ensured with tolerances of ± 0.1 mm, this accuracy was ensured by the use of the measuring needle having an accuracy of reading at 0.05 mm nonius scale.

To ensure the possibility of visual observation and to photograph the processes that take place in the weir, the model of the weir in the well, the bottom and the pressure wall of the leak channel connecting the well to the upper basin are made of transparent Plexiglas (Figure 6).

Test model methodology

Laboratory studies on a reduced model of hydro technical structures make it possible to predict the behavior of the future full-scale structure and to find, during design, the optimal solutions satisfying the conditions of safety and economy (Kashkaki et al. 2019).

To make the sufficiently reliable forecast of the parameters of the operation of the full-scale structure on the

basis of the corresponding data from laboratory studies, it is necessary to respect the laws of similarity, on the basis of which the models are projected to make the conversion into the nature of the test results, it is necessary that similar phenomena be described by analogous differential equations of the same type (Zhang 2015; Liu et al. 2018).

In addition, one must respect the similar conditions of unambiguity when calculations contain the geometric dimensions of the work, the boundary and initial conditions, the physical characteristics of the flow in nature and on the model (Zhang et al. 2018).

The process of movement of water through drainage structures occurs under the effect of the forces of gravity (Shemshi & Kabiri-Samani .2016). To respect the similarity of the forces taking place in the full-scale work and on the reduced model, it is necessary to determine the scale of the forces (White 2009):

$$m_F = \frac{F_n}{F_m} = \frac{m_n a_n}{m_m a_m} = \frac{\rho_n W_n L_n T_n^2}{\rho_m W_m L_m T_m^2} = \frac{\rho_n L_n^2 V_n^2}{\rho_m L_m^2 V_m^2} \quad (1)$$

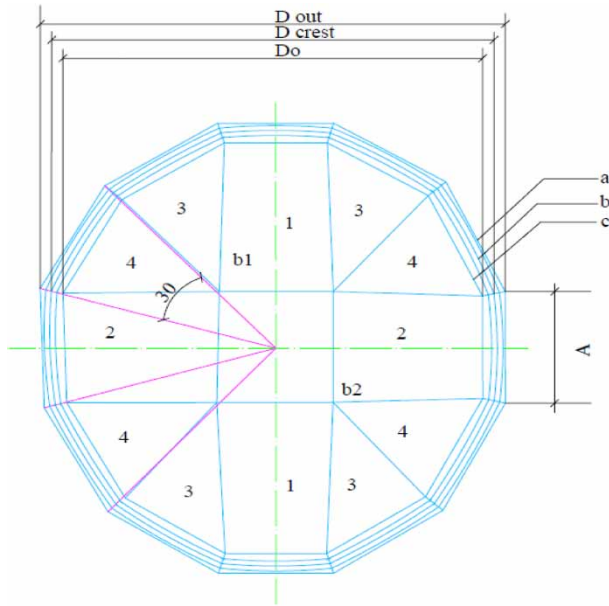


Figure 5 | Plan of the shaft of the polygonal cross-section with a 12 holes water inlet funnel. The following elements are indicated: (1, 2): Trapezoidal faces; (3, 4) - Corner faces. A: Length of the outer edge of the weir. D_{ext} : Diameter of the inscribed circle of the outer limit of the funnel. D_{crest} : Diameter of the inscribed circle of the crest of the funnel. b_1 and b_2 : The sides of the rectangular exit section of the shaft. a: The outer edge of the water intake funnel. b: The axis of the water intake funnel. c: The interface line of the inner surface of the funnel and the corresponding face of the shaft.

where, F_n and F_m – forces of the same nature in the full-scale work and on the reduced model; m_n and m_m – corresponding masses; a_n and a_m – accelerations in the full-scale

structure and on the scale model; ρ – density of the liquid, W – its volume; L – characteristic length; T – time; indices – ‘n’ and ‘m’ – denote, respectively, the full-scale work and the reduced model.

In accordance with formula (1) we have:

$$\frac{V_n^2}{g_n L_n} = \frac{V_m^2}{g_m L_m} = Fr = idem$$

Thus, the processes taking place in structures under these conditions are defined by the criterion of Froude Fr representing the ratio of the kinetic energy of the flow in the section given to half of the potential energy of the mass of the liquid in the given section.

Under these conditions, the similarity of the kinetic and hydrodynamic characteristics of geometrically similar works is respected.

In hydro-technical practice during the simulation, the test liquid is the water found in the same conditions as those of nature, that is to say practically with the same physical and mechanical characteristics. In accordance with Newton’s hypothesis, tangential stresses (the specific forces of viscosity) are proportional to the viscosity of the liquid and to the gradient of the speed; this, respecting the conditions of the kinematic similarity of phenomena in nature and for the model, leads to the appearance of the same viscosity forces in nature and for the model. At the same time, as it follows



Figure 6 | Photos of the experimental device with the distribution of the piezometers on the model.

from the expression $P_n = P_m m_L^3$, the pressure in the liquid at the corresponding points decreases in proportion to the simulation scale to the cubic power.

With the decrease in the scale of the simulation there comes the moment when the forces of viscosity cannot be neglected and the results of the studies begin to depend on the Reynolds number, i.e. the results of the studies leave the area of self-similarity and must be transferred to the full-scale work in accordance with the relations expressing the laws of similarity to be completed by the so-called 'scale' corrections.

The magnitudes of these corrections, outside the area of self-similarity, depend on the character of the studied phenomena to which they relate: the flow of water through weirs, the flow of local obstacles, the gradually varied movement of water in long pipes, and the flow through orifices.

Thus, if we examine the movement of water in the water pipe, the determining force there will be the force of friction at the borders of the flow (White 2009):

$$T = \mu \frac{dV}{dn} \omega = \nu \rho \frac{dV}{dn} \omega \quad (2)$$

The similarity of the phenomena (according to the Equation (1)) must be ensured in the event of satisfaction of the relationship:

$$\frac{\rho_n L_n^2 V_n^2}{\rho_m L_m^2 V_m^2} = \frac{\nu_n \rho_n V_n L_n}{\nu_m \rho_m V_m L_m} \quad (3)$$

where, μ - dynamic viscosity; ν - kinematic viscosity.

From (3) we get:

$$\frac{V_n L_n}{\nu_n} = \frac{V_m L_m}{\nu_m} = \text{Re} = \text{idem} \quad (4)$$

That is to say during the simulation of the friction forces at the border of the flow, the Reynolds numbers for nature and the model must be identical.

The introduction into the flow even of small resistances greatly reduces the value of Re_{lim} in comparison with the uniform movement. In the case of the presence in the flow of artificial works (hydraulic installations, cofferdams, adjustment works) where the local resistances are defined most clearly, the Reynolds Re_{lim} boundary numbers on the sections of influence of these works are of the order of $10^3 \dots 10^4$. In order to obtain reliable test results from the model, the Reynolds number must not be less than $\text{Re}_{\text{lim}} = 5 \times 10^3$, which is in the range of indicated quantities of Re_{lim} . In this case, the regimes of the model are in a self-similarity zone according to the Reynolds number where its influence on the results of the studies is excluded or it is negligible.

The scales (m_i) of the main physical quantities corresponding to the linear scale when the Froude and Reynolds numbers are equal for the full-scale structure and the reduced model can be determined using Table 3 in the case of using water as a test liquid (Chanson 2004).

In accordance with Table 3, the scale coefficients for converting the main characteristics of the model spillway into full-scale work are:

Table 3 | Coefficients of passage from the parameters of the model to those of the full-scale structure

№	Parameter	Law of similitude		№	Parameter	Law of similitude	
		Fr = idem	Re = idem			Fr = idem	Re = idem
1	Length	m_L	m_L	9	Work	m_L^4	m_L
2	Area	m_L^2	m_L^2	10	Discharge	$(m_L)^{5/2}$	m_L
3	Volume	m_L^3	m_L^3	11	Unit flow	$(m_L)^{3/2}$	1
4	Time	$m_L^{1/2}$	m_L^2	12	Pressure	m_L	$(m_L)^{-2}$
5	Velocity	$m_L^{1/2}$	$(m_L)^{-1}$	13	Dynamic viscosity	$(m_L)^{3/2}$	1
6	Acceleration	1	$(m_L)^{-3}$	14	Kinematic viscosity	$(m_L)^{3/2}$	1
7	Mass	m_L^3	m_L^3	15	Roughness according to Manning	$(m_L)^{1/6}$	$(m_L)^{5/3}$
8	Force	m_L^3	1	16	Coefficient of Chézy	1	$(m_L)^{-3/2}$

- Velocity scale m_v :

$$m_v = \sqrt{m_L} = \sqrt{60} = 7.75. \tag{5}$$

- time scale m_t :

$$m_t = \sqrt{m_L} = \sqrt{60} = 7.75. \tag{6}$$

- flow conversion m_Q :

$$m_Q = m_L^{5/2} = 60^{5/2} = 27885.5. \tag{7}$$

The discharge of 1 l/s on the model corresponds to the flow of 27.89 m³/s for the full-scale structure and at the maximum discharge of the structure in full size of 740 m³/s corresponds to the flow of 26.54 l/s on the model.

- conversion of the pressure expressed in meters of water column m_p :

$$m_p = m_L = 60.$$

The coefficient of hydraulic friction λ corresponding to these drafts is determined by Sheffinson formula (Carlier 1980):

$$\lambda = 0.11 \cdot \sqrt[4]{\frac{\Delta}{4R}} = 0, 11 \cdot \sqrt[4]{\frac{0, 01}{4 \cdot (78 \div 30)}} = 0.008 \div 0.0105. \tag{8}$$

For the full-scale structure with the medium quality concrete surface with the roughness coefficient $n = 0.014$ and the indicated hydraulic radii, the hydraulic friction coefficient $\lambda = 0.009 \dots 0.011$. Thus, the hydraulic processes taking place on the model correspond entirely to the hydraulic processes *in situ*, and the adopted scale of the simulation is equal to 1/60 of the natural size and the materials used ensure the hydraulically similar regimes for the whole range of flow rates main of the spillway.

The load H_{mod} on the model of the spillway was measured relative to the crest of the weir of the inlet funnel of the spillway in wells set at the corresponding dimension for the full-scale structure at the dimension equal to 555.0 m. When measuring the load on the model, we took as the reference dimension of the piezometer

panel a dimension corresponding to the conversion on the model of the dimension of: 555.100/60 = 925 cm.

The level $\nabla Z_{Upstream}$ of the full-scale structure was determined in this case by the relationship:

$$\nabla Z_{Upstream} = 555.0 + 60 \times (\nabla Z_{Upstreammod} - 925).$$

To obtain the high precision of the load measurements, the test installation was equipped with a piezometer separate from the upstream reach ensuring the load measurements to the nearest 0.1 mm.

Hydraulic studies of the flow capacity of the spillway in polygonal section wells were carried out in a wide range of loads on the crest from the receiving funnel: from $H_{o,min} = 0.25$ m to the heads, creating a complete flooding of the water intake funnel with a pressure

$H_{o,max} = 7.0$ m, in which it begins to function as a water intake of a deep intake device for the height of the section of exit from the mine outlet $a = 2.5, 3.36, 4.5, 5.0, 5.5$ and 6.0 m.

The composition of the investigated options for the model of the mine with a polygonal cross-section is given in Table 4.

The flow rate of the flow through the weir in a well is determined by the next equation (Akan 2006):

$$Q = m b \sqrt{2g} H^{3/2} \tag{9}$$

Table 4 | Composition of experimental studies of a shaft spillway of a polygonal cross-section

Variant studied	The height of the output section of the mating confuser, a, m					
	2.5	3.36	4.5	5.0	5.5	6.0
First variant: free entry			■	■	■	■
Second variant: entry with guide piers in upstream	■			■	■	■
Third variant: entry with guide piers in downstream	■		■	■		■
Fourth variant: guide piers in upstream + shortened chamber				■		
Fifth variant: guide piers in upstream + thin wall in downstream		■		■		

and the velocity of the flow of water through the ridge V_{se} :

$$V_{se} = m \sqrt{2gH} \quad (10)$$

RESULTS AND DISCUSSION

The hydraulic structure depends on several hydrological, geological, and geotechnical conditions which differ from one case to another, but the good thing is that the same approach can be applied for other cases, the same hydraulic approach and the same equations can be applied too, to improve the functioning of the structure and reduce the costs as well, it is for this reason that all design parameters are given in dimensionless values, which allows them to be used in accordance with the theory of hydraulic similarity.

All parameters of the proposed structure of the mine spillway can be used regardless of the climatic and other characteristics of a particular region, therefore a limitation in the use of such works, is conditioned by another more efficient spillway structure, which will be solved by an economic comparison of these variants.

The paper is of course a case study, but this case can be similar to others in terms of the manner of treating the hydraulics structures, so we have written this paper in such a way, that it can be considered as a protocol, where we have described all major steps in the design of such structures, which allows us to calculate a similar mine weir with all the parameters, and which can offer a significant gain in structure coast, Especially in countries, where there is a lack of high level engineering skills and financial resources.

The advantages of the proposed design have been stated over traditionally used mine weirs. This design consists of replacing the circular section of the well with a polygonal section, which ensures the formation of a continuous flow along the entire length of the culvert, and the replacement of the configuration of the plan of the well facilitates the formwork and concreting work, and at the same time improves the hydraulic operation of the discharge surface of the well due to the absence of transverse joints.

The case study is a single case in Algeria, it shows how we can propose alternative solutions for the same hydraulic

problem to make it more efficient and less expensive. This case study can also be added to the global database in order to analyze these hydraulic structures in depth and to suggest further improvements and solutions.

The results and the comparison for all investigated options of the throughput capacity of a shaft spillway with a polygonal cross-section are shown in Figure 7. The present study details the results of the basic version with the design height of the outlet section of the confuser $a = 5.0$ m, since the phenomena observed for other options are similar and differ only in the degree of intensity, depending on the size of the outlet cross-section of the mine spillway.

As can be seen from the graphs of Figure 7, with the height of the output section of the shaft up to $a = 5.0$ m inclusively, the transfer of capacity from the free flow mode of operation of the water intake funnel to submerged one occurs practically without a transition section.

The discharge capacity of a mine spillway is determined by the discharge of two structural elements: a water intake funnel and the output section of the shaft spillway (Akan 2006).

As can be seen in the graphs in Figure 8 which show the discharge characteristics of the water intake funnel when operating with the flow free mode $Q_{int. \text{ funnel}} = f(Z_{NWL})$ depending on the water level of the upstream level and the discharge characteristics of the output section of the shaft $Q_{output. \text{ Section}} = f(Z_{WL \text{ output. Section}}, a)$ depending on the water level in the trunk mine and the height of its output section a :

$$Q = \sigma \times m \times L \sqrt{2gH^{5/2}} \quad (11)$$

where:

$\sigma \leq 1$: submerging coefficient;

m : Flow coefficient of the ridge of the intake funnel;

L : Distance of the crest of the intake funnel;

H : Head on the crest of the intake funnel;

$g = 9.81 \text{ m/s}^2$ - acceleration of gravity.

The flow rate Q of the output section of the shaft is calculated by the equation (Akan 2006):

$$Q = \mu \times \omega_{output} \sqrt{2g(H_0 + Z)} \quad (12)$$

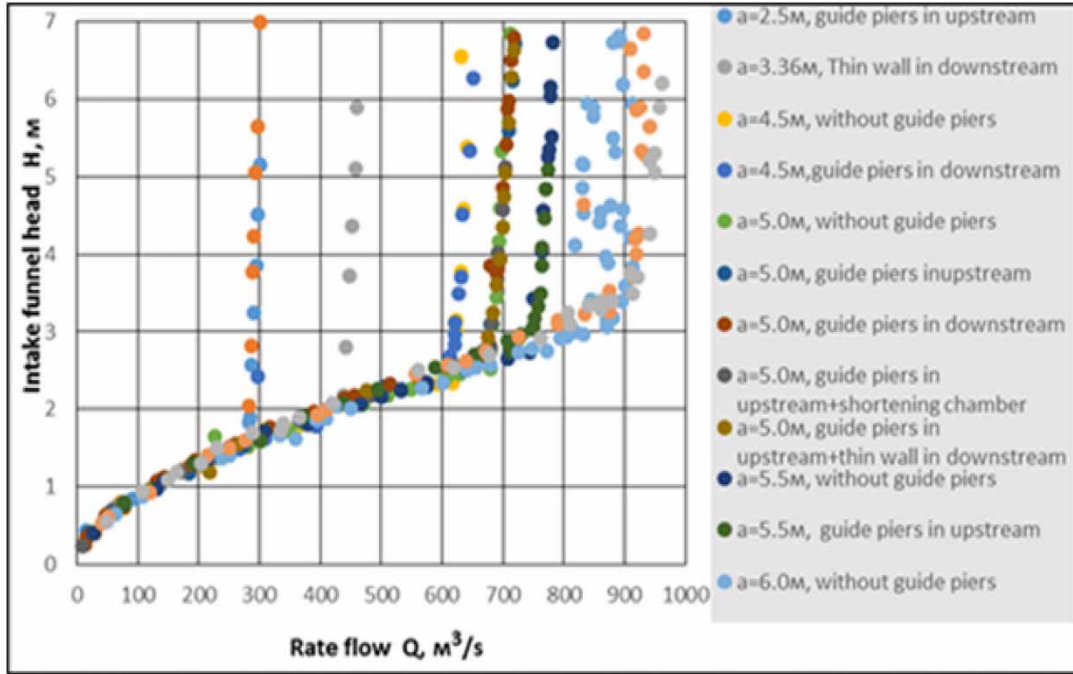


Figure 7 | Comparison of the mine capacity of the investigated options.

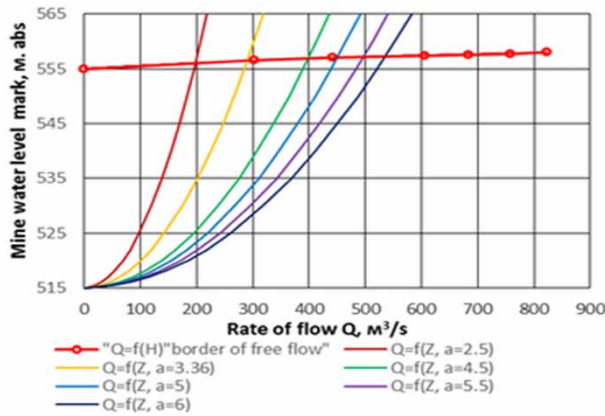


Figure 8 | Flow characteristics of the intake funnel and the input section of the shaft.

where μ is the generalized flow coefficient, determined by the equation (Moise 1970):

$$\mu = \frac{1}{\sqrt{\xi_{outp} \left(\frac{\omega_{outp}}{\omega_1}\right)^2 + \sum_1^n \frac{2g}{C_{iav}^2} \times \frac{Z_0}{nR_{iav}} \left(\frac{\omega_{outp}}{\omega_1}\right)^2 + \alpha + \xi_{outp}}} \quad (13)$$

where:

ω_{out} : The area of the output section of the shaft of the mine;

Z_0 : The height of the shaft – the distance from the crest of the intake funnel to the output section of the shaft;

ω_1 : Inlet cross-sectional area of the water intake funnel;

n : The number of sections into which the shaft of the mine is divided by height when determining hydraulic losses along its length;

C_i : The average value of the Chézy coefficient on the i -th section;

ω_i Average: The average area on the i -th section of the shaft of the mine;

α : The Coriolis coefficient in the output section;

ξ_{outp} : Resistance coefficient of the output confuser of the mine shaft.

The analysis in Figure 9 shows a graph of the maximum discharge pressure of the free flow mode on the crest of the intake funnel over the relative height of the outlet section of the mine shaft. This dependence is well approximated by the Equation (14), is the result of a mathematical treatment of the experimental curve of Figure 9:

$$H_{max} = 2.946 \times \left[\frac{a}{a_0}\right]^{0.678} \quad (14)$$

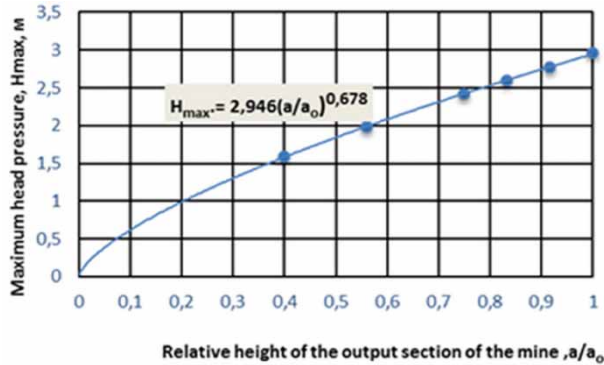


Figure 9 | Dependence of the maximum head pressure of the free flow on the relative height of the output section $H_{\max} = f(a/a_0)$.

Change in the limiting values of the flow coefficient $m_{\max} = f(a/a_0)$ of the inlet funnel and the flow coefficient $\xi_{\text{subm.}} = f(a/a_0)$ of the output section of the shaft of the mine from the relative height of its output section (a/a_0) for the transition point from the free and submerged operation is shown in Figure 10.

Equations (15) and (16) are well approximated by a polynomial of the 2nd degree (are the result of the mathematical treatment of the experimental curves of Figure 10):

$$m = 0.162 \left(\frac{a}{a_0} \right)^2 + 0.283 \left(\frac{a}{a_0} \right) + 0.396 \quad (15)$$

and

$$\xi = 0.158 \left(\frac{a}{a_0} \right) + 0.306 \left(\frac{a}{a_0} \right) + 0.6 \quad (16)$$

Respectively.

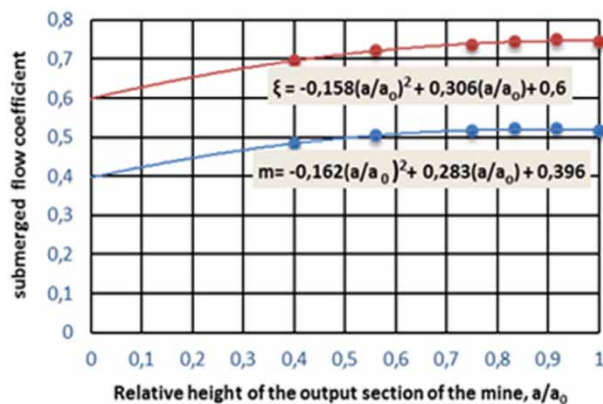


Figure 10 | The dependence of the values of the funnel flow coefficient m and ξ during its submerging $\bullet m = f(a/a_0)$; $\bullet Q = f(a)$.

Figure 11 shows a generalized graph of the coefficient change of the shaft spillway of a polygonal cross-section, depending on the relative head of H_o/H_d , where the head $H_d = 3.4$, $r = 2.55$ m is taken as the shaping head.

The experimental points fall into three sections. Formulas (17, 18 and 19) are the result of a mathematical treatment of the corresponding sections of the experimental curve of Figure 11. In the first section, in the range of relative head $H_o/H_d < 0.55$, an intensive increase in the flow coefficients with an increase in the head of the relative head (H_o/H_d) is observed. In this section, the change in the flow coefficients is well described by the following equation:

$$m = 0.587 \cdot (H_o/H_d)^{0.314} \quad (17)$$

In the pressure range $0.55 < H_o/H_d < 1.08$, the intensity of the increase in the flow coefficients with increasing head of the relative head (H_o/H_d) decreases, although their growth continues. In this section, the change in the flow coefficients is well described by the equation:

$$m = 0.522 \cdot (H_o/H_d)^{0.1587} \quad (18)$$

With a relative head $H_o/H_d > 1.08$, a sharp decrease in the consumption coefficients begins. This is due to the fact that the water level in the shaft reaches the mark of the crest of the water intake funnel, and it starts to work in a flooded mode, in which the funnel starts from the hydraulic point of view inlet of a water intake with a low coefficient of

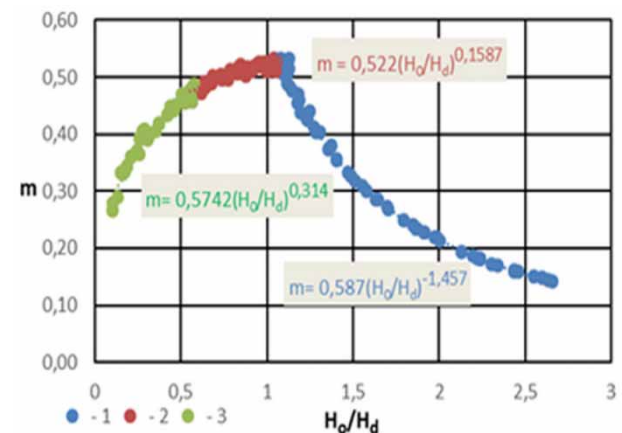


Figure 11 | Graph of the change in the discharge coefficient of a shaft spillway.

resistance. In this section, the change in the flow coefficients is well described by the equation:

$$m = 0.587 \cdot (H_o/H_d)^{-1.457} \quad (19)$$

Flood routing is often the most important hydraulic problem posed by the construction of large dams. This study is a unique case in Algeria, its configuration was tested on a hydraulic model in the laboratory, and proposes a new design for the well spillway of the Djedra dam (eastern Algeria). The procedure consists of replacing the circular shape with a polygonal configuration of twelve (12) sections, its use can greatly simplify the formwork of the structure, and the results also show that the free entry funnel of the weir increases the discharge coefficient by 20%, without exposing it to the risk of cavitation, and in the event of submersion, the adopted design can ensure the crossing of a flood flow estimated with a probability of $P = 0.01\%$, which means the reduction of the flood height of the dam, and thus make the hydraulic structure more economically efficient and more reliable.

According to the preliminary design of the Djedra dam, when using a mine with the outline of the Creager water intake funnel, the estimated flow coefficient was $m = 0.435$, at which the transformed flood flow rate of the $P = 0.01\%$ supply was $Q_{maxcalculated}^{P=0.01\%} = 740 \text{ m}^3/\text{s}$ and passed at the elevated level of the overhead dam. $Z_{PWL} = 558.28 \text{ m}$; and Figure 12(a) shows a graph of the variation of the levels of the upper tail during the passage of a calculated safety flood $P = 0.01\%$. As can be seen in this graph, the mark of the forced level of the upper tail is $Z_{PWL} = 557.60 \text{ m}$, which is 0.68 m less.

As a rule, unregulated water discharge facilities operating in automatic mode are used on rivers with high water floods, when the river's costs can increase by hundreds of m^3/s in a matter of hours. It is impossible to predict the time of occurrence of such floods.

Figure 12(b) shows the calculated hydrograph of the $P = 0.01\%$ supply with the maximum flow rate $Q_{pic} = 915 \text{ m}^3/\text{s}$, as can be seen from the graphs of these hydrographs, with a flood of $P = 0.01\%$, the flow rate of the river increases from zero to $915 \text{ m}^3/\text{s}$ for 3 hours, and in case of a flood $P = 0.01\%$, the flow of the river increases from zero to $637.7 \text{ m}^3/\text{s}$ for 3 hours.

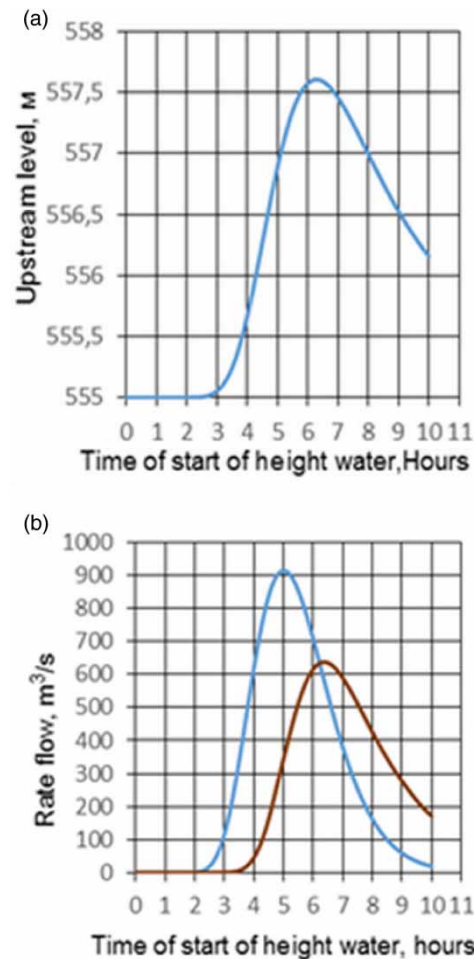


Figure 12 | Top level change graph water during the passage of high water security with $P = 0.01\%$, and hydrograph flood. (a) Top level change graph water during the passage of high water security with $P = 0.01\%$. (b) Hydrograph of flood with $P = 0.01\%$. (— Calculated hydrograph of the river, and — Hydrograph of the transformed flood).

CONCLUSION

The analysis of the results of the experimental research relating to the use of the weir in wells of polygonal section to replace the circular section in a dam, showed the considerable improvement of its flow parameters, this is observed in the graph of dependence on the discharge capacity by a free intake funnel, which takes the form of a soft and moderate line, and its mode of operation in submersion is characterized by values of the flow rate, shown by a vertical line. The graphs of the discharge capacity of the shaft spillway show that a stable hydraulic regime is observed at a low height of the outlet section, this is particularly evident in the flow graph

with free entry into the funnel for the section outlet at a height $a_0 = 6.0$ m. The use of a head in polygonal section of the weir increases the flow coefficient from $m = 0.435$ to $m = 0.527$, that is to say 20%, and the discharge capacity of the flow in the event of a probability flood $p = 0.01\%$ reduced the height of the dam by 0.68 m by adding the circular section.

Throughout the possible operating range, the weir operates in free flow mode, and the installation of a guide piers on the water intake funnel reduces the intensity of the rotational movement of the water in the shaft weir and reduces its unstable operation. Therefore, the most effective way is to reduce the exit section of the well by 10% compared with the end section of the confectioner.

The design of these polygonal cross-section mining weirs are justified by their economic advantages (reduction in the volumes of formwork and reinforced concrete) during construction, and the improvement of their hydraulic operation (increase in the capacity of discharge during strong floods) during their operation, for this purpose, there can be no specific measures that can be recommended, or a special investigation for their future projection in all dams in the world, which confirms the final design of this type of weir which is judged hydraulically satisfactory and recommended for construction of the Djedra dam in the East of Algeria.

DATA AVAILABILITY STATEMENT

All relevant data are included in the paper or its Supplementary Information.

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