Studia Geotechnica et Mechanica, Vol. XXXII, No. 2, 2010

MODEL INVESTIGATIONS OF SIDE CHANNEL SPILLWAY OF THE PILCHOWICE STORAGE RESERVOIR ON THE BÓBR RIVER

JERZY MACHAJSKI

Institute of Geotechnics and Hydrotechnics Wrocław University of Technology,
Wybrzeże Wyspiańskiego 27, 50-370 Wrocław, Poland.

DOROTA OLEARCZYK

Institute of Environmental Engineering,
Wrocław University of Environmental Life Sciences.

Abstract: In frames of works connected with modernizing the outlet installations of the Pilchowice storage reservoir on the Bóbr River, the model investigations of side channel spillway were carried out. The investigations were preceded by analyzing the principles of the functioning of this type of installation and analytical calculations defining its capacity ability. The studies were carried out on a large-scale physical model with a full representation of the conditions of water inflow on side channel spillway and also the conditions of water outflow by multistage cascade. The results of model tests were compared with analytical calculations and on that basis a necessity of the reconstruction of this installation was shown. Such a reconstruction ensures the safety of the reservoir in terms of computational discharges in the current state of a technical solution of outlet installation, significantly exceeding its nominal capacity ability.

1. INTRODUCTION

The equivalent tasks of the Pilchowice storage reservoir are flood protection of the downstream Bóbr River Valley and water storing for energetic purposes. Installed electric power output is equal to 9.20 MW, for six turbines with the capacity of 37.2 m^3/s. During the most frequent occurrence of flood freshet wave period (July–October) in the reservoir a permanent flood storage capacity equal to 26 hm^3 is kept. Water surplus flows through outlet installation, including diverse channel, two bottom outlets, each with 1500-mm diameter, and side channel spillway. Bottom outlets are built into the dam body from left-sided abutment, whereas three conduits with similar parameters are built into diverse channel cross-section, dividing diverse channel into two sections: pressure one and free flow one. From the spillway of 84-m crest length, water flows out downstream through multistage cascade. The maximum discharge of outlet installations is equal to 575 m^3/s; 371 m^3/s of this discharge flow through side channel spillway and cascade [1], [3], [4]. The dam of 69-m height makes a storage reservoir with the maximum capacity of 55 hm^3. It was built in 1904–1912 and its arch body was made of broken stone in concrete. A functional plan of the Pilchowice dam is shown in figure 1.

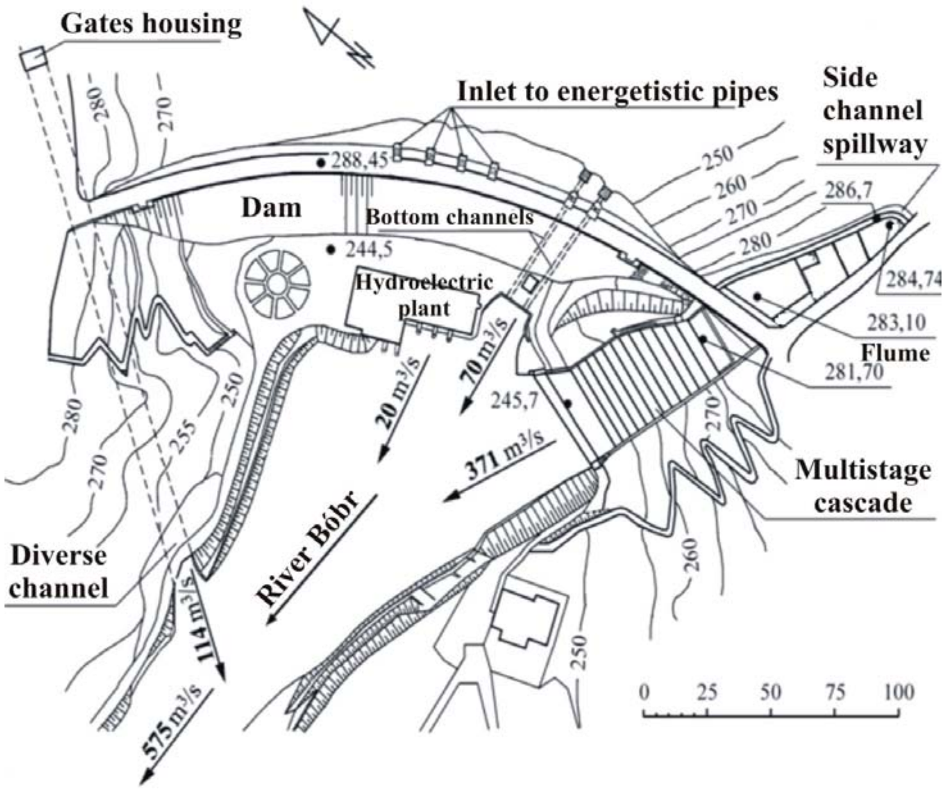

Fig. 1. Functional plan of the Pilchowice storage reservoir

2. THE BÓBR RIVER

The Bóbr River, except the Nysa Kłodzka River, is the most important left-sided tributary of the Odra River. The catchment area of the Pilchowice cross-section equals 1209 km^2. In hydrological calculations relating to the Pilchowice reservoir, a hydrological data from Jelenia Góra gauging station, located upstream, are taken into account. Characteristic discharges in that cross-section are as follows (Polish abbreviations):

- characteristic discharges: minimum NNQ = 0.369 m^3/s, average low flow SNQ = 2.20 m^3/s, mean SSQ = 14.3 m^3/s, average maximum flow SWQ = 98.7 m^3/s,
- maximum discharges in the years: 1915 – 352 m^3/s, 1938 – 425 m^3/s, 1958 – 444 m^3/s, 1965 – 320 m^3/s, 1977 – 452 m^3/s, 1981 – 314 m^3/s, 1997 – 574 m^3/s,
- maximum discharges with a given probability of exceedance: $Q_{50\%}$ = 126 m^3/s, $Q_{10\%}$ = 274 m^3/s, $Q_{1\%}$ = 476 m^3/s, $Q_{0.5\%}$ = 536 m^3/s, $Q_{0.2\%}$ = 614 m^3/s, $Q_{0.1\%}$ = 674 m^3/s.

According to regulations [7] the Pilchowice dam is classified into hydrotechnical structures of the first (I) class of importance. For such structures, not subjected to destruction due to their overflow, the probability of exceedance for computational discharges should be set at 0.5% and 0.1%, but this is valid for a higher discharge with the upper extension at $t_\alpha = 1$ and the confidence level of 0.84. In the Jelenia Góra gauging station, the discharges are equal to, respectively, $Q_{0.5\%} = 536$ m^3/s and $Q^{\alpha}_{0.1\%} = 778$ m^3/s, while in the cross-section of the Pilchowice dam: $\mathbf{Q_{0.5\%} = 605}$ **m^3/s**, $\mathbf{Q^{\alpha}_{0.1\%} = 877}$ **m^3/s**. Computational discharges significantly exceed the outlet installation capacity ability of the reservoir [1], [3], [4].

3. CONCEPTION OF OUTLET INSTALLATION RECONSTRUCTION

In the conception of reconstructing the Pilchowice reservoir's outlet installations, a safety of reservoir was taken into account due to computational discharge values and the capacity ability of the existing outlet installations [1], [3]. The consequences of the occurrence of computational discharges in the reservoir were determined, analysing the possibilities of their reduction to safe discharges both for downstream area and for reservoir. In this regard, the water management instructions are of a prime importance [4], as they state that a damming level should not be lower than 258.00 m a.s.l. (this is the equivalent of dead storage) because of the dam construction [1], [3].

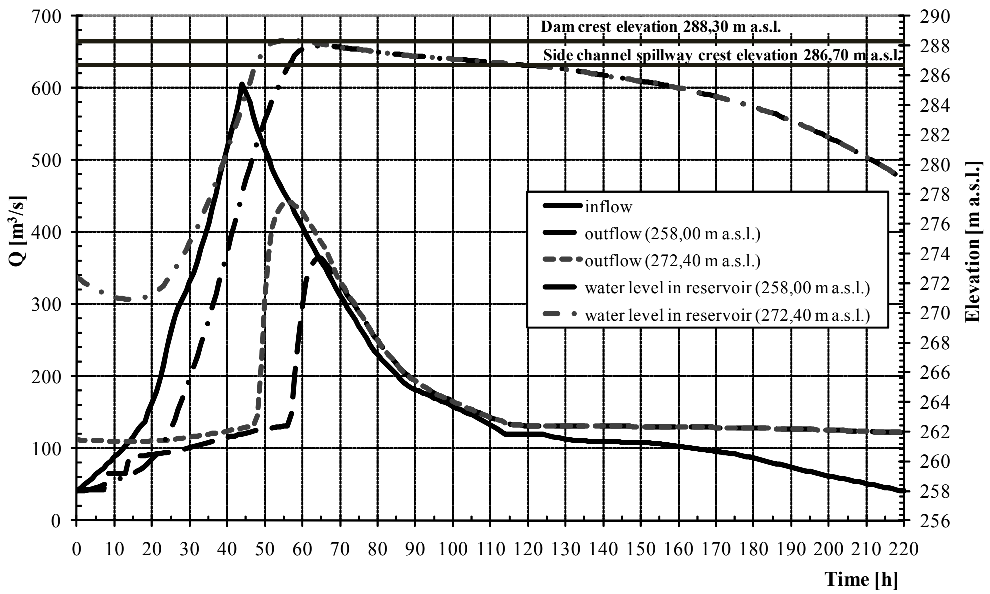

Fig. 2. Transformation of design flood wave $Q_{0.5\%}$ for the existing outlet installation, initial levels of water dammed in reservoir, 258.00 and 272.40 m a.s.l.

The calculations of flood waves transformation [1], [4] showed that for the existing outlet installations there is no possibility of preserving a safe dam crest height above the maximum water level in the reservoir (1.0 m [7]). So for the flood wave $Q_{0.5\%}$, the water level in the reservoir reaches 287.87 m a.s.l., i.e. 0.88 m to dam crest in an initial state of reservoir fulfillment of 258.0 m a.s.l., and 288.29 m a.s.l., i.e. 0.46 m to dam crest in an initial state of the reservoir fulfillment of 272.40 m a.s.l. (allowable level of the damming up of water from the 16th May to the 15th October). For the flood wave $Q^{\alpha}_{0.1\%}$ a water level in the reservoir will exceed the dam crest by about 0.14 m in an initial state of water level equal to 258.0 m a.s.l. and by about 0.52 m in an initial state of 272.40 m a.s.l., which means a dam crest overflow. The results of the transformation calculations are shown in figures 2 and 3.

Having recognized the conditions of reservoir operation, a necessary changes were introduced to improve considerably the operating conditions and the object safety. Two possibilities were taken into account: side channel spillway reconstruction in order to increase its capacity ability by over 100 m^3/s or the reconstruction of diverse channel. Before undertaking a final decision, the faults and advantages of every possibility were taken into account [1].

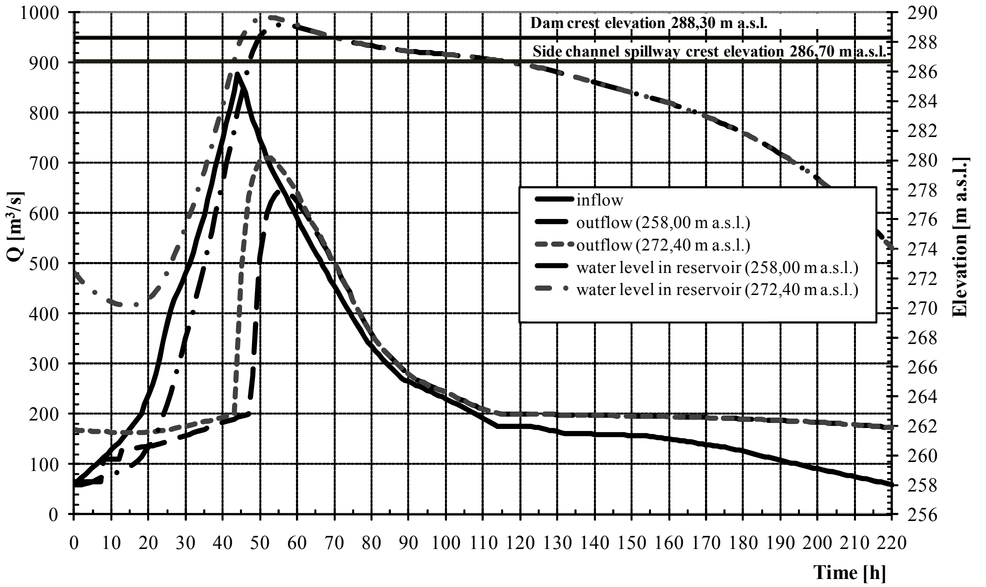

Fig. 3. Transformation of control flood wave $Q^{\alpha}_{0.1\%}$ for the existing outlet installation, initial levels of water dammed in reservoir, 258.00 and 272.40 m a.s.l.

Spillway can be reconstructed without reservoir emptying, and its safety is ensured because flood waves can pass through spillway without any negative consequences. As a result of rebuilding, the channel operating conditions will be changed in such a way that side channel spillway in every conditions works as not submerged. A significant drawback of spillway reconstruction is no possibility of improving the operating conditions of freshet wave routing. At present, the operation of outlet installation is carried out until a damming level in reservoir will reach dam crest, i.e. an outflow will be somewhat over 200 m^3/s. Diverse channel reconstruction not only improves these conditions but also allows the regulation of outflow intensity if a predicted flood wave will be forthcoming. Such a reconstruction is, however, more complex, it needs a reservoir emptying and directing the waters of the Bóbr River to bottom channel built into the dam body. In the period of preliminary works, a hydroelectric plant will not work, and after the installation of overhaul gate's inlet of diverse channel, a water damming-up will be possible, giving, however, lower power production than that obtained under normal conditions of water damming in reservoir.

It should be emphasized that closing a diverse channel reduces the capacity ability of outlet installations by 114 m^3/s, hence it poses a relatively greater threat of crest overflow in the moment of one of computational discharges occurrence. Such a situation arose in 1977 [3]. During repair works of plain valves of diverse channel, a triple wave that lasted several times as long as normal but reached a comparatively small culmination equal to 452 m^3/s came into the reservoir cross-section.

In view of the above mentioned requirements for exploitation safety of reservoir and the suggestion of its owner, first of all the possibility of side channel spillway reconstruction was taken into account.

4. ANALYTICAL CALCULATIONS

The analysis of the work of side channel spillway [1], [5], [6], [9], [10], [12] proved that by a possible reduction of computational waves in reservoir, an outlet installation improvement due to its reconstruction could be easily obtained [1]. Numerical computations of flood wave transformed by the reservoir [1], [4] showed the need for the spillway reconstruction to obtain a lacking capacity ability of the whole outlet installation equal to 100 m^3/s. In the reconstruction of side channel spillway, the following aspects were taken into account: bad state of the ground foundation of spillway, its construction, parameters of its flume, its crest, building an arch bridge onto a downstream section of flume, and constructional designs of multistage cascade, which joined spillway channel with a downstream section of river. The analysis of these limitations permitted elimination some of them.

Spillway construction and its connection to the foundation did not allow any significant lowering of flume bottom in such a way that in whole range of water damming-up in reservoir a side channel spillway could work as not submerged. The possibility of crest spillway lowering was eliminated because a spillway work could quite quickly lost its free character, hence its capacity ability would be decreased. Also the possibility of flume rebuilding was not taken into account due to its location on the slope of a hill along the ledge with a road. An end cross-section of flume where arch bridge was located offered some possibilities. The cross-section of multistage cascade and its capacity ability are considered to be limitations in any reconstruction of flume as well.

Preliminary analyses were carried out only for side channel spillway, but they have not been undertaken for a flow through cascade. Increasing a flow through cascade from 371 to 478 m^3/s, i.e. by more than 100 m^3/s, could change radically hydrodynamic conditions of water impact on its individual stages as a result of an increase in their load and water depth in the cascade cross-section. The overflowing water can cause a washout of slopes which took place in the past, when after the reservoir building the chute replaced cascade and was conducted on a natural rock foundation restricted by the walls of a small height.

It was assumed that spillway capacity ability could be improved by the correction of flume bottom line only in such way as to obtain the change of spillway operating condition from submerged to not submerged. In an initial phase, our own analytical computations of spillway were carried out on the basis of available literature [5], [6], [9], [12].

Side channel spillway fulfils two parallel functions, i.e. that of overflow part and that of flume, which is conducted parallel to overflow edge. This flume takes over the waters overflowing spillway and conveys them to installations that connect side channel spillway to downstream river section. The analysis of water movement conditions within the side channel spillway has changed gradually. At present [5], [6], [9] the analysis is available based on the law of linear momentum conservation assuming that the only force ensuring flow of water in flume results from the slope of water level in the direction of water routing. Next, the energy of the water overflowing the spillway is dissipated due to its mixing with water in flume and simultaneously has no influence on the flow conditions in flume.

Taking account of the above, the purpose of hydraulic computations of side channel spillway is to determine a necessary length of spillway edge, assuming the following exploitation conditions of the reservoir: normal and maximum water levels and the determination of the pattern of water-table lines in flume at the assumed parameters of flume cross-section and the slope of its bottom.

Spillway discharge is determined assuming that it works in the whole range of expected discharges as not submerged. Calculations were carried out using the formula for free spillway with a straight insert on crest:

$$Q = \varepsilon\, 2/3\, \mu\, B \sqrt{2\, g}\; H_0^{3/2}, \tag{1}$$

where:

- ε – the coefficient of side contraction weir, $\varepsilon = 1.0$,
- μ – the discharge coefficient, $\mu = 0.654$,

B – the spillway crest length, B = 84.0 m,

H_0 – the energy height calculated in terms of the spillway crest.

For the calculations three characteristic damming levels in reservoir were taken. They result from water level with regard to weir crest (286.70 m a.s.l.) and dam crest (figure 4) and determine water layer thickness and the resultant energy of stream. The calculation results are as follows:

288.75 (dam crest from downstream) – 286.70 = 2.05 m → Q = 476.15 m^3/s,

288.60 (dam crest from downstream) – 286.70 = 1.90 m → Q = 424.86 m^3/s,

288.30 (dam crest from upstream) – 286.70 = 1.60 m → Q = 328.32 m^3/s.

In a further analysis for flume parameters determination, the discharge equal to 424.86 m^3/s was assumed.

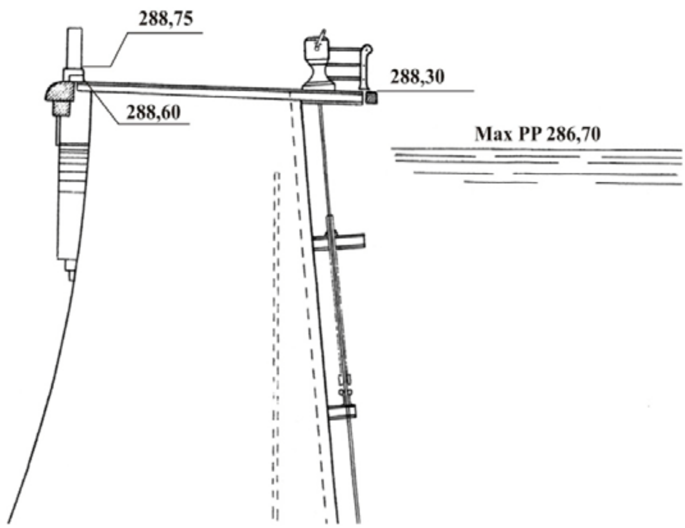

Fig. 4. Height position of dam crest of the Pilchowice reservoir

In the calculations necessary for the choice of flume parameters, a simplified (neglecting frictional head losses) differential equation, describing water flow profile in the flume, is applied [5], [6]:

$$\frac{dy}{dx} = \frac{1}{g}\left(v\frac{dv}{dx} + \frac{v^2}{x}\right), \tag{2}$$

where:

dy – the change of water level along the length of the computational section dx,

v – the mean flow velocity in the cross-section of a given computational step,

x – the length of flume measured from its initial cross-section.

Because the discharge grows linearly with the length of flume it can be assumed that velocity also changes with the flume length, however, to a certain extent, in an arbitrary way, provided that flume has been designed in such a way as to allow such a possibility. It is described by the exponential function [5], [6]:

$$v = a\, x^n , \tag{3}$$

where a, n are the arbitrarily chosen constants.

Equation (3) after differentiation and the reinsertion into (3) is as follows:

$$\frac{dv}{dx} = n\, a\, x^{n-1} = \frac{n\, v}{x} , \tag{4}$$

and after insertion into equation (2) and integration after x:

$$y = \frac{a^2\,(n+1)}{g\,(2\,n)}\, x^{2n} . \tag{5}$$

Equation (5) proves that flume cross-section, its position and bottom slope should ensure the proportionality of water-table line head to velocity head. The constants a and n in equation (3) are arbitrarily assumed and can be taken in such a way as to adapt a profile of water-table in flume to the surroundings. For example, from equation (5) it can be seen that for $n = 0.5$ the profile of water-table line is horizontal, sloped down when $n > 0.5$, sloped up when $n < 0.5$.

The arrangement of water-table lines in flume is determined from the velocity and its cross-section area. The equation describing a required slope of flume bottom is determined from the relation between flume filling and water level in a given flume cross-section. Usually, the position of water-table line at end cross-section of flume is fixed by the so-called control cross-section, in which the conditions of critical motion are fulfilled by a critical depth. This condition allows also the constant a to be determined, provided that the constant n is known.

In more exact calculations, an impact of the roughness of flume material should be taken into account. When $A = Q/v$, $h_f = S_f\,dx$, $h_0 = S_0 dx$:

$$\frac{dy}{dx} = \frac{S_0 - S_f - 2\,\alpha\, Q\, q \,/\, g\, A^2}{1 - \alpha\, Q^2 \,/\, g\, A^2\, D} , \tag{6}$$

where:

S_o – the slope of flume bottom,

S_f – the slope of energy line, calculated from the Manning formula at $n = 0.015$,

α – the correction coefficient of kinetic energy (Saint Venant), $\alpha = 1.10$,

h_f – the energy loss per length unit of flume,

h_0 – the lowering of flume bottom per length unit,

q – the discharge increment per flume length unit, dQ/dx,
A – the area of flume cross-section,
D – the hydraulic depth equal to A/B,
B – the width of flume.

The above differential equation has two solutions: either by applying available numerical methods for this type equations or by applying simplified methods, which in general consist in the introduction of finite differences in place of differentials. For example, the equation determining the change of water levels Δy for the length Δx is [5], [6], [12]:

$$\Delta y = \frac{Q_1 (v_1 + v_2)}{g (Q_1 + Q_2)} \left[(v_2 - v_1) + \frac{v_2 (Q_2 - Q_1)}{Q_1} \right] \Delta x \,. \tag{7}$$

Applying the above relationships, the necessary flume parameters were determined in order to allow a whole discharge corresponding to the spillway capacity ability and to warrant a spillway operation as not submerged, because only in such condition it has the capacity ability equal to 424.86 m^3/s.

The calculations reveal that a free flow of computational discharge through existing spillway is not possible, because its flume is too shallow and too narrow in the upper part. Therefore, the operating conditions the spillway change quickly and it works as submerged. For that reason the flume should be deepened by about 3.0 m with a simultaneous set of its width at 22.50 m. Under such conditions the assumed discharge of spillway can be obtained for damming water level in reservoir for 288.60 m a.s.l. These requirements offer the possibility of the rebuilding in question, mainly in terms of economic calculation. The reconstruction would force the necessity of constructional changes of weir – new static conditions of its work, and a necessity of constructional changes of wall on the opposite side – dynamic load due to vehicular movement on the road. In that situation it is assumed that in model investigations a minimum of works, necessary to improve spillway functioning, i.e. increasing its capacity ability, will be determined.

5. MODEL INVESTIGATIONS

The model investigations were carried out for the whole outlet installations of reservoir [2]. First, the aim of model investigations was to verify the proposed reconstruction of a side channel spillway. The program of investigation consisted of measurements and observations allowing the following parameters to be determined:

– the capacity ability of side channel spillway in terms of the changes of levels of damming up in the reservoir ranging from Max*PP* (286.70 m a.s.l.) to dam crest from downstream (288.75 m a.s.l.) (figure 4),

– the operating conditions of multistage cascade during greater discharge passage,

– the operating conditions of arch bridge in the place where the flume changes to multistage cascade,

– the minimal constructional changes of a spillway that can be relatively easily introduced.

The investigations were carried out on a spatial model at scale of 1:40. This model included all element important for water passage, including side channel spillway with multistage cascade [2]. The water over weir crest and in a flume flows predominantly due to the gravity force, therefore the Froude criterion of similarity is applied to calculate the appropriate forces occurring both in nature and in the model [6], [8], [9], [11]. A viscous force also has a certain contribution to the phenomenon studied. In order to maximally restrict the influence of viscous force, the kind of flow in the model and in nature has to be the same. According to input data, under the conditions of changes in computational discharge passage a turbulent flow occurs, therefore in the model a turbulent flow with relatively great Reynolds number is indispensable [8]. Under model conditions and at variable discharges, the Reynolds numbers from Re = 15 000 for the discharge $Q_{0.5\%}$ to Re = 35 000 for the discharge $Q^{\alpha}_{0.1\%}$ were obtained. Hence, it was inferred that the phenomenon modelled was significantly affected by the gravity force.

In maintaining the similarity of phenomena observed both in model and in nature, the design of a model with appropriate roughness seems of a prime importance. According to input data all the elements of external outlet installation are made of concrete, hence the roughness coefficient n is equal to 0.020. Because the model scale is equal to $\alpha_l = 40$, therefore after recalculation $\alpha_n = 1.8493$ [2]. The resultant roughness coefficient of the material that should be used for essential details of the model outlet installations should be equal to:

$$n_M = \frac{n_N}{\alpha_n} = \frac{0.20}{1.8493} = 0.01081 .$$

This equation proves that the elements of outlet installation should be made of a very smooth material, e.g. Plexiglass® or Vinidur®.

Model investigations were based on the operating conditions of reservoir given in water management instruction [4]. According to this instruction the side channel spillway fulfils a key role in a flood wave routing through reservoir. The operating conditions of side channel spillway, whose capacity ability should be determined, are as follows:

– the level of maximum damming up in reservoir, 286.70 m a.s.l.,

– the permissible exceeding, 288.30 m a.s.l.,

– the level of damming up that cannot be exceeded, 288.75 m a.s.l.

The aim of the changes introduced into the model was to improve the capacity ability of spillway, i.e. to correct the height configuration of flume bottom in relation to spillway crest. The investigations were carried out for three height configurations of flume bottom.

The first one is the existing configuration with clearly outlined steps in flume bottom (change I shown in figure 5).

Fig. 5. View of side channel spillway model for existing configuration (change I)

In the second configuration (change II), two of the highest steps in flume bottom (figure 6) are removed.

Fig. 6. View of side channel spillway model after change II

In the third configuration (change III), all bottom steps are removed, and the line of longitudinal slope of flume bottom is unified (figures 7 and 8). For each of the changes introduced into spillway model the whole cycle of investigations was repeated.

Fig. 7. View of side channel spillway model after change III

6. HYDRAULIC CHARACTERISTIC OF SPILLWAY

The spillway characteristic was determined for the discharges ranging from 0.0 to 400 m^3/s and for the damming up level varying from 286.70 m a.s.l. (spillway crest elevation) to 288.75 m a.s.l. (the elevation of dam crest downstream, figure 4). During investigating a spillway, the bottom outlets, hydroelectric plant and diverse channel were closed. Taking measurements for this characteristic determination, the operating conditions of spillway were observed, mainly in respect of the disturbances at spillway entry cross-section at the side of narrowed flume (figure 8) and in the region of right bridge abutment which constitutes a dam crest extension over a spillway. Moreover, there were observed the water flow along a downstream side of a spillway, the conditions of great discharge entry into a bridge cross-section and the conditions of its flow through the first stage of cascade.

Spillway hydraulic characteristic in the function of water dammed up in reservoir is shown in figure 9 in order to compare three changes introduced into the model. In figure 10, a resultant change of discharge coefficient is presented, also for three changes introduced into the model. Rise of the discharge curve, changing the character of weir work from not submerged to submerged, is visible. This is a situation that for this type of installation should not develop, in every conditions determined by weir parameters and water damming up levels in reservoir this installation should work as not submerged. Because investigations concerned an existing spillway, it was important to determine the conditions for the change in a weir work. The spillway capacity ability obtained based on investigations is close to that calculated, mainly in terms of water which under given conditions can overflow weir, provided that in the whole range of discharges it works as not submerged. This corresponds to the discharge of about 425 m^3/s. Such a discharge can be obtained for the existing spillway lay-out in plan, its crest length, weir shape and such flume parameters as its width, depth and longitudinal slope of bottom.

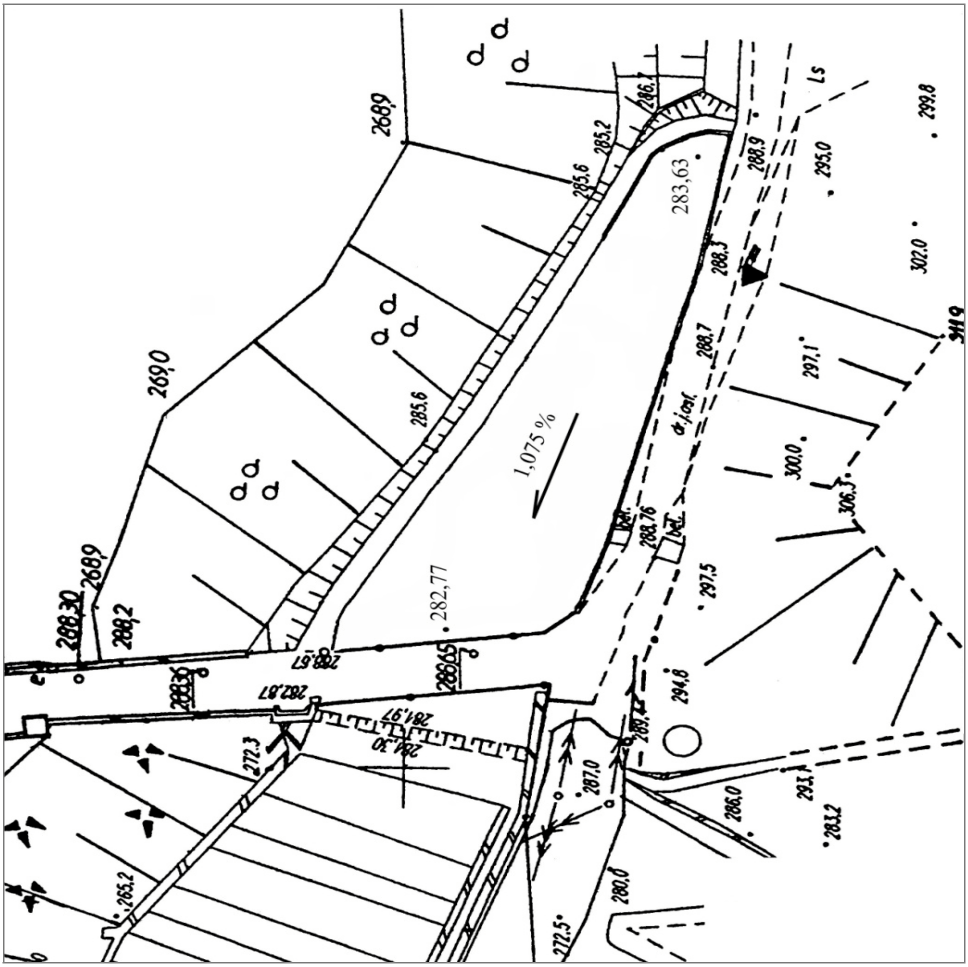

Fig. 8. Side channel spillway under conditions of change III

Under the conditions of change I, for the present solutions of weir and flume the spillway capacity ability is lower, which results from quick changes of its operating conditions from not submerged to submerged. This takes place for the discharge equal to 275 m^3/s (figure 9) and the water damming up in reservoir reaching 288.071 m a.s.l. The maximum spillway capacity ability under these conditions was 350 m^3/s at the water damming up of 288.404 m a.s.l., i.e. water reaches the dam crest. After introducing change II, the minimum improvement in the spillway operating conditions was obtained, whereas its capacity ability was not improved and any changes of damming up levels in reservoir were not obtained. The change III allowed the spillway operating conditions to be improved and caused the expected changes of flow conditions in weir, which occurred at the discharge of 325 m^3/s (figure 9), and increased the spillway capacity ability to the expected 425 m^3/s. This took place for water damming up in reservoir to 288.767 m a.s.l., which slightly exceeded the dam crest from downstream.

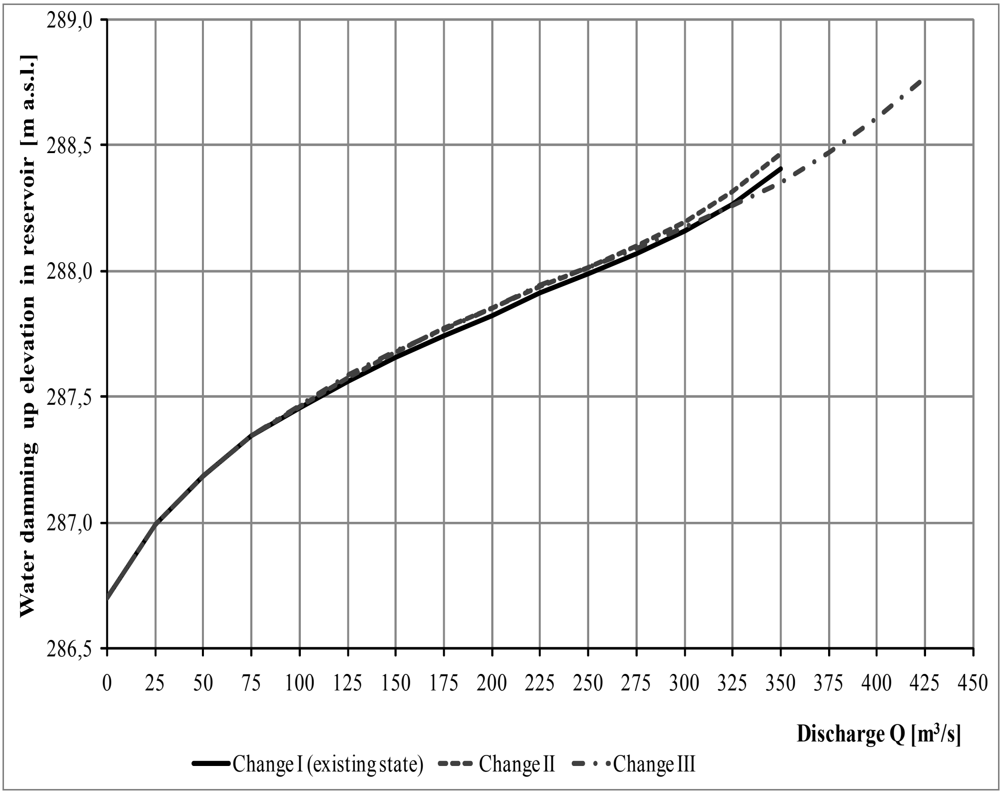

Fig. 9. Hydraulic characteristic of side channel spillway of the Pilchowice reservoir for the existing state and after reconstruction

The spillway operating conditions after introducing change III are evaluated as correct. Any serious disturbances at the input cross-section both from the left and right sides of bridge abutment are not observed, any disturbances at the weir output cross-section are not observed either, stream closely adheres to weir and gently flows in flume cross-section. Some disturbances occur in the upper side of weir during greater discharges as a consequence of too small cross-section and the depth of flume in that region (figure 11).

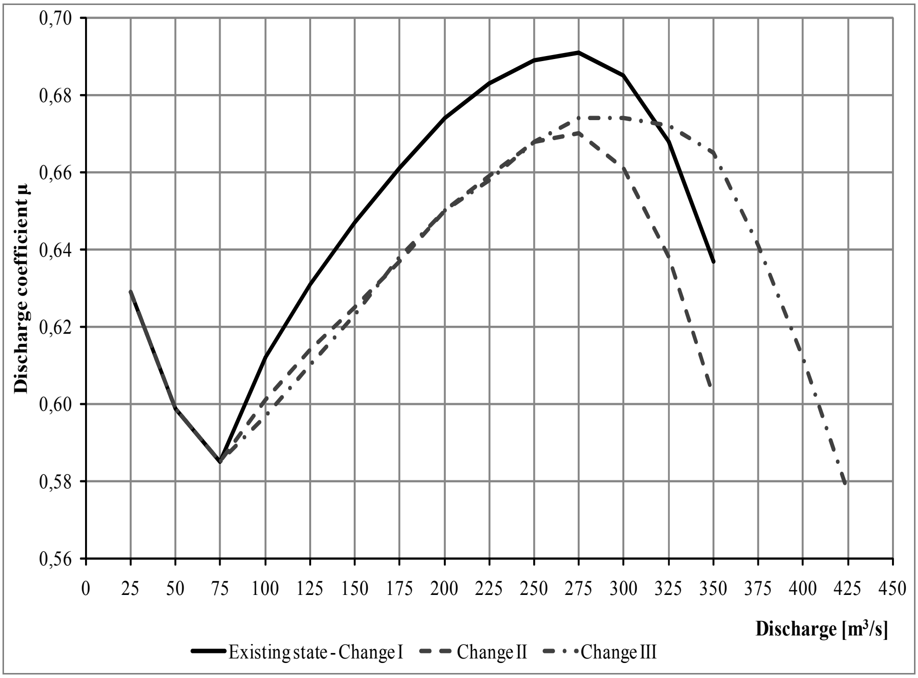

Fig. 10. Characteristics of changes of discharge coefficient of side channel spillway for the existing state and after reconstruction

Fig. 11. Spillway work under conditions of the discharge passage of about 400 m^3/s (change III)

Also an improvement of the conditions of water flow into a bridge cross-section is observed. During the discharge passage of 400 m^3/s a bridge still works as free, only from the left side of its abutment a small water damming up is observed. It is caused by the differences in the angular displacement of flume and multistage cascade axes. At great discharges in the range of 375–400 m^3/s, the bridge construction is exposed to a minimum water hittings, mainly from left side of its abutment (figure 12).

Fig. 12. Road bridge work under conditions of the discharge of about 400 m^3/s (change III)

The investigation confirmed the results reported in technical literature. They reveal that side channel spillways should be designed in such an elevation configuration of flume as to enable their operation in the whole range of computational discharges as not submerged. Their discharge depends on the weir crest length, the thickness of overflowing water and the weir shape (crest shape and its contour) [5], [6,] [9], [12].

7. SUMMARY

The model investigations of side channel spillway of the Pilchowice reservoir are mainly connected with its hydraulic characteristic. The authors discuss the possibility of improving the capacity ability of this installation, hence the improvement of exploitation safety of the whole object. The solutions proposed that correct the configuration of the flume of side channel spillway are verified during model tests. This allow us to draw the following conclusions [2]:

1. In order to improve the exploitation safety of the Pilchowice reservoir, it is necessary to change the existing state of flume of side channel spillway.

2. A change in side channel spillway solution consists in the deepening of a flume bottom by the removal of existing steps and in assuring its constant longitudinal slope equal to 1.075%.

3. A flume deepening increases the capacity ability of this installation by about 75 m^3/s.

4. An improvement in the conditions of computational discharge passage through the bridge cross-section is obtained.

REFERENCES

[1] MACHAJSKI J., RĘDOWICZ W., *Analiza w aspekcie technicznym i ekonomicznym, możliwości zwiększenia przepustowości stopnia wodnego Pilchowice na rzece Bóbr*, Raport serii SPR nr 11/2008, Politechnika Wrocławska, Instytut Geotechniki i Hydrotechniki.
[2] MACHAJSKI J., *EW Pilchowice I. Przebudowa stopnia wodnego – etap II – badania modelowe*, Raport serii SPR nr 4/2009, Politechnika Wrocławska, Instytut Geotechniki i Hydrotechniki.
[3] MACHAJSKI J., RĘDOWICZ W., *Ocena zdolności przepustowej stopnia wodnego Pilchowice I na rzece Bóbr*, Gospodarka Wodna, nr 5/2006, 194–200.
[4] MACHAJSKI J., OLEARCZYK D., *Instrukcja gospodarowania wodą na zbiorniku retencyjnym Pilchowice*, Politechnika Wrocławska, Instytut Geotechniki i Hydrotechniki, 2006.
[5] KHATSURIA R.M., *Hydraulics of Spillways and Energy Dissipators*, Marcel Dekker Publishers, New York, 2005.
[6] NOVAK P., MOFFAT A.I.B., NALLURI C., NARAYANAN R., *Hydraulic Structures*, Taylor & Francis Publishers, New York, 2007.
[7] Rozporządzenie Ministra Środowiska z dnia 20 kwietnia 2007 w sprawie warunków technicznych, jakim powinny odpowiadać budowle hydrotechniczne i ich usytuowanie. DzU Nr 86/2007, poz. 579.
[8] NOVAK P., ČÁBELKA J., *Models in Hydraulic Engineering – Physical Principles and Design Applications*, Pitman Publishers, London, 1981.
[9] VISCHER D.L., HAGER W.H., *Dam Hydraulics*, John Wiley & Sons, Chichester, 1998.
[10] ROGALA R., MACHAJSKI J., RĘDOWICZ W., *Hydraulika stosowana. Przykłady obliczeń*, Wydawnictwo Politechniki Wrocławskiej, Wrocław, 1991.
[11] CUNGE J.A., ACKERS P., *Topics in Hydraulics Modeling*, Proceedings of XXII Congress International Association IAHR for Hydraulic Research, Lausanne, 1987.
[12] TANČEV L., *Dams and Appurtenant Hydraulic Structures*, A.A. Balkema Publishers, London, 2005.