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Proposal of Measures to Enhance the Safety of the Znojmo Dam on the Thaya River during Floods

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The catastrophic flood in August 2002 endangered functional structures of dams with flow rates that significantly exceeded the values considered during their design. Particularly on the Znojmo Dam on the Thaya river, that was put into operation in the year 1966, the nominal discharge considered was $Q = 355\ m^3.s^{-1}$. After the flood of 2002 the nominal discharge had to be increased to $Q = 610\ m^3.s^{-1}$. Due to this change studies of the proposed reconstruction of the functional structures (block) were made. The aim was to increase the safety of the dam. The submitted paper presents results of the hydraulic research of these operational structures.

Keywords: Znojmo Dam, safety of dams, safety overflow, spillway capacity, hydraulic research, contractions, piers, NACA profile, basin, baffle blocks, chute blocks.

1 Introduction

The 1997 and 2002 floods were the reason for reassessment of the design parameters of dams mainly with respect to safe management of extreme water passage. One of the dams which no longer comply with modern parameters is the Znojmo Dam. Therefore the study Galc, Janků, Bubeník (2003) with variants of technical measures for safe management of extreme water passage was made. Variant 2b – exchange of gates combined with raising the dam crest level were chosen as the best modifications. The project of these modifications and hydraulic research followed. Hydraulic tests modelled the conditions at maximum water level in reservoir and at design flow $Q_N = 610\ m^3.s^{-1}$ as well as at an imaginary flood with flow rate of $Q_{10\,000} = 740\ m^3.s^{-1}$.

2 Combined spillway block

Combined spillway block of Znojmo Dam comprises a safety spillway, two bottom outlets and a water intake. The existing spillway has two 8.70 m wide sections closed by 3.0 m high flap gates. These sections are divided by 1.80 m wide central pylon. There are two bottom outlets under the left section and two small hydropower stations (HPS) under the right one. Inlets of bottom outlets and HPS are divided by 3.30 m wide separating piers. The stilling basin is 20.8 m wide, 33.0 m long and 3.5 m deep. To increase the spillway capacity of the Znojmo Dam the following modifications were proposed: To sink the existing spillway crest from 223.00 m a.s.l. to 221.80 m a.s.l. and to exchange the existing flap gates for tainter gates with flaps. The necessary modifications of stilling basin were designed, including increasing the height of basin walls. A breakwater was designed on the dam crest as well as a debris retaining system in front of spillway (see Fig. 1).

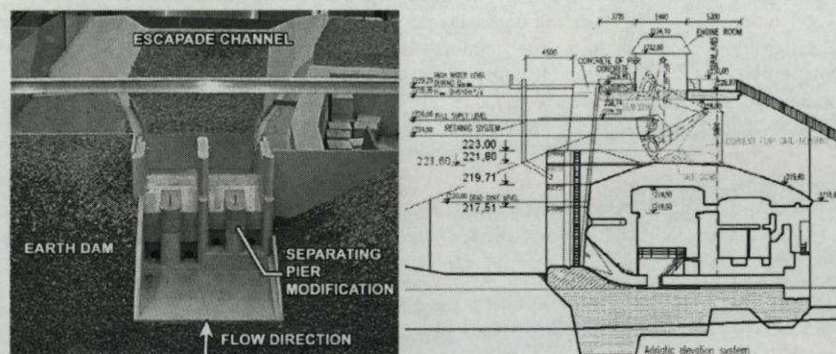

Figure 1: Downstream view of the spillway block of the Znojmo dam and its side section including the designed changes

3 Test channel, model

For the hydraulic research of the spillway block of the Znojmo Dam a physical model was built respecting the rules of Froude's similarity in the length scale $M_l = 35$. The dam model was installed in a glass-walled channel 2.5 m wide and 14.5 m long, part of the equipment of the new fully automated Laboratory of hydraulic research in the F building of the Faculty of civil engineering, Brno University of Technology, Veveri 95, Brno, Czech Republic. The channel forms a part of a hydraulic circuit maintaining a stabilised rate of flow of up to $Q \sim 150\ l.s^{-1}$. The model consisted of the spillway block with parts of the dam, stilling basin and 157 m length (in real scale) of the Thaya riverbed. There were

31 pressure holes in the spillway surface. Upstream face of the dam was covered by gravel with diameter of particles (10 ÷ 15) mm. Bottom of the channel downstream of the basin step was covered by the same gravel which corresponds to the real riprap material. View of the model is shown in Fig. 1.

4 Model Tests

Tests were made in batches. Each of them was aimed to assess one of the block modifications. The modification effects on

- capacity and pressure conditions on the spillway surface,
- energy dissipation in the stilling basin

were evaluated.

During the tests the segment gates were opened, bottom outlets and HPS were closed. All numerical values shown in following text are related to the real dam dimensions.

4.1 Spillway

There are 3.3 m wide separating piers with vertical grooves of cofferdam upstream of each section of spillway (see Fig. 1) whose modification considerably affects the spillway capacity. Effect of the elevation of the top surface of the piers is shown in Table 1.

Table 1: Effect of separating piers elevation on spillway capacity

Elevation of separating piers [m a.s.l.]	Level in reservoir when $Q_N = 610\ m^3.s^{-1}$ [m a.s.l.]	Discharge when level in reservoir is 230.00 m a.s.l. $[m^3.s^{-1}]$
223,00	229,20	703
221,80	229,12	719
219,71	229,01	732
217,51	228,95	739

These results led to the decision to use the piers top elevation of 221.60 m a.s.l., see Fig. 1. Next, 4 variants of separating piers shapes were proposed and tested (see Fig. 2).

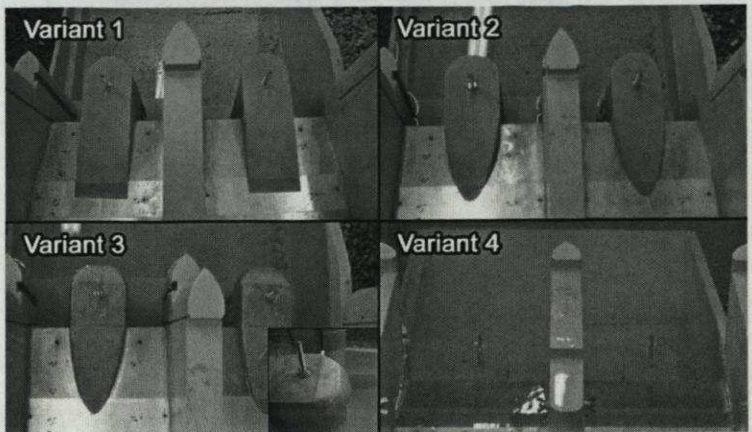

Figure 2: Variants of separating piers shapes

They respected the chosen elevation of top surface and the requirement to keep the shape of the front part of separating piers where there are grooves of cofferdam. The ground-plan shapes of piers (Variants 2 and 3) resulted from the Treftz's transformation described in Boor, Kunštátský, Patočka (1968). The elevation was decreased to 219.71 m a.s.l. at the Variant 4. The effect of modifications of separating piers on discharge coefficient m was investigated.

Five discharges were selected to draw the stage-discharge relation curve: 50; 150; 300; 610 and 730 $m^3.s^{-1}$. The value of discharge measured on inductive flow-recorder was verified by integration of the measured velocity field. Discharge coefficient m was computed from measured values of water depths (w.r.t. the weir crest) and discharges according to the formula:

$$m = \frac{Q}{b_0 \cdot \sqrt{2 \cdot g} \cdot h^{3/2}}, \tag{1}$$

where $b_0 = b - 0.1 \cdot n \cdot \xi \cdot h$

- Q flow over the spillway (estimated by integration of velocity field) $[m^3.s^{-1}]$
- h water depth (w.r.t. the weir crest) [m], where upstream velocity $v_0 \approx 0\,m.s^{-1}$
- b_0 effective length of the spillway [m]
- b total length of spillway b = 17.4 m
- n number of contractions n = 4
- ξ pier contraction coefficient $\xi = 0.0$, see later

The influence of separating piers modifications on discharge coefficient m is small, see Fig. 3.

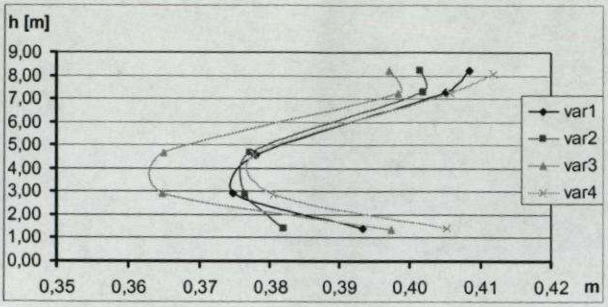

Figure 3: Discharge coefficient m as function of water depth (w.r.t. the weir crest), $\xi = 0{,}0$

The influence of separating piers modifications on the shape of the water surface and distribution of flow velocity values across the profile of segment gates is small. The water surface shapes are approximately symmetrical in both sections. The velocity values are almost symmetrical to the spillway axis.

Results of measurements have shown that the Variant 1 (designer's choice) was the best modification from the hydraulic and structural point of view (easy to build with minimum of demolitions). The other modifications were not effective enough from the point of view of capacity increase. The values of discharge coefficient m for Variant 1 are summarised in Tab. 2.

Table 2: Discharge coefficient m values as function of discharge, valid for Variant 1

$Q\ [m^3.s^{-1}]$	51	143	282	606	725
m	0,392	0,372	0,374	0,398	0,401

The side piers cause flow contractions. It was proved that these have no effect on spillway capacity ($\xi = 0.0$). Wakes cause water level increase of approximately $(1.0 \div 1.1)$ m at side piers when the discharge is $Q = 610\ m^3.s^{-1}$. The improvement of wake effects, i.e. decreasing the wave crest, will have positive effect on necessary gate opening during floods. Therefore several tests of flow improvement were performed. This was achieved by using wing-like steel elements attached to the front part of pier walls. The curved steel plates end by tangent linking to the existing pier wall approximately at the point where the spillway surface starts to slope down. The minimisation of plate dimensions was achieved by taking step-by-step measurements while using different plate shapes and sizes. The plate is shown in Fig. 4. The plates will be placed on the top plane of the piers.

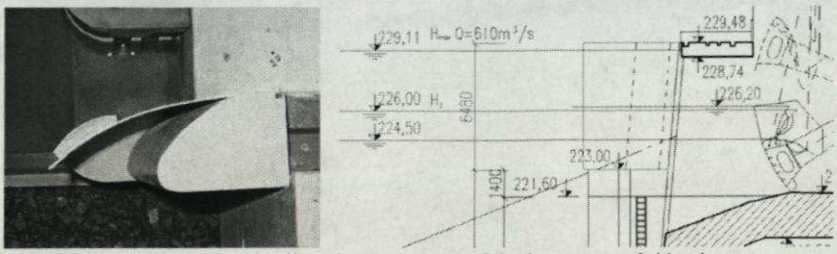

Figure 4: The stream leading plate set on top of the lower part of side pier

The circular trailing end of central pier was replaced by the rear part of NACA profile [Ladson, Brooks, Hill (1996)] with the aim to eliminate the wave formed downstream of the pier. Hydraulic efficiency of flow treatment by NACA profile is shown on Fig. 5. No change of flow upstream of the pier due to this installation has been registered.

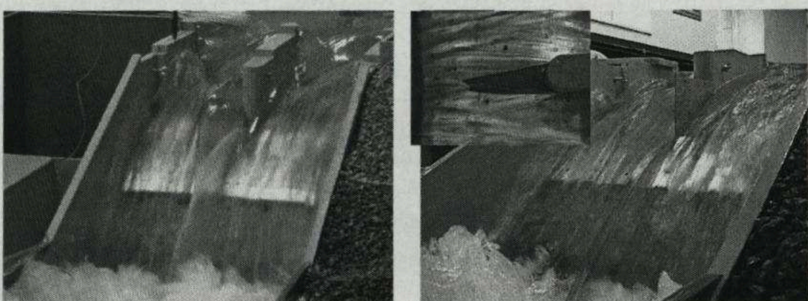

Figure 5: Deformation of level arising behind the end of middle pier and its elimination by the NACA profile when $Q = 610\ m^3.s^{-1}$

No values of pressure sufficiently negative to cause cavitations effect on the spillway surface have been registered during the model measurements - even when the rate of flow reached $Q = 732\ m^3.s^{-1}$.

4.2 Stilling basin

First model experiments have shown that the existent stilling basin of Znojmo Dam (length $L = 33.0$ m, depth $h = 3.5$ m) can no longer comply with the requirement of sufficient energy dissipation at design rate of flow $Q_N = 610\ m^3.s^{-1}$. It was necessary to propose its modification. The client required to keep the elevation of the stilling basin bottom at 207.65 m a.s.l. because the bottom is formed by the natural rock. The preliminary proposal was based on computation performed for the design discharge value $Q_N = 610\ m^3.s^{-1}$ [Stara, Šulc (2004)]. Summary is shown in Table 3.

Table 3: Summary of stilling basin evaluation

Mark	Unit	Value			Description
E_0	m	21,46			Total energy height
b	m	20,80			Basin width
q	$m^2.s^{-1}$	29,327			Specific discharge (on 1 m of basin width)
t	m	5,60			Tailwater depth
d	m	3,50			Basin depth
β		1,1			Boussinesq's number
φ		0,7	0,84	0,9	Velocity coefficient
y_1	m	2,26	1,87	1,74	Upstream sequent depth of hydraulic jump
y_2	m	8,17	9,27	9,7	Downstream sequent depth of hydraulic jump
h_s	m	5,91	7,4	7,96	Height of jump $h_s = y_2 - y_1$
L_v	m	32,46	37,02	39,83	Length of jump by Novák
σ		1,11	0,98	0,94	Backwater rate $\sigma = (t + d)/ y_2$
L_p	m	8,93	9,90	10,26	Streamer landing length from the end of the spillway surface 10,35 m above basin bottom and oblique by angle 35° from horizon
L_p	m	15,21	18,25	19,55	Dtto, baffle face horizontal
$L = L_v + L_p$	m	41,39	46,92	50,09	Total basin length for oblique throw
$L = L_v + L_p$	m	47,67	55,27	59,38	Total basin length for horizontal throw

Values of velocity coefficient for the given type of spillway were taken from Boor, Kunštátský, Patočka (1968). Basin length was calculated as the sum of streamer landing length and the rising hydraulic jump length.

The necessity to extend the existing basin length follows from results in Table 3. To begin with the initial basin length L = 53.5 m was chosen. This was stepwise shortened to L = 48.5 m and L = 46.0 m. Next, from Table 3 it follows that at the current basin depth the requirement of drowned jump ($\sigma > 1.05 \div 1.1$) will not be fulfilled. Therefore new baffle blocks in the basin and at the end of the spillway surface were designed to increase energy losses.

Two types of chute blocks were designed at the end of the spillway surface. They differed in shape, number, and position, see Fig. 6. It was shown that the position of the central block of type 1 was not suitable due to wave formation downstream of the end of central pier. The wave concentrated the flow to the centre of spillway. The upper face of the central block directed the flow further down the basin. Irregular inflow resulted in 3D flow in the basin and in the tailwater. Therefore the chute blocks type 2 were designed. No relation between the chute blocks and upstream water level was registered.

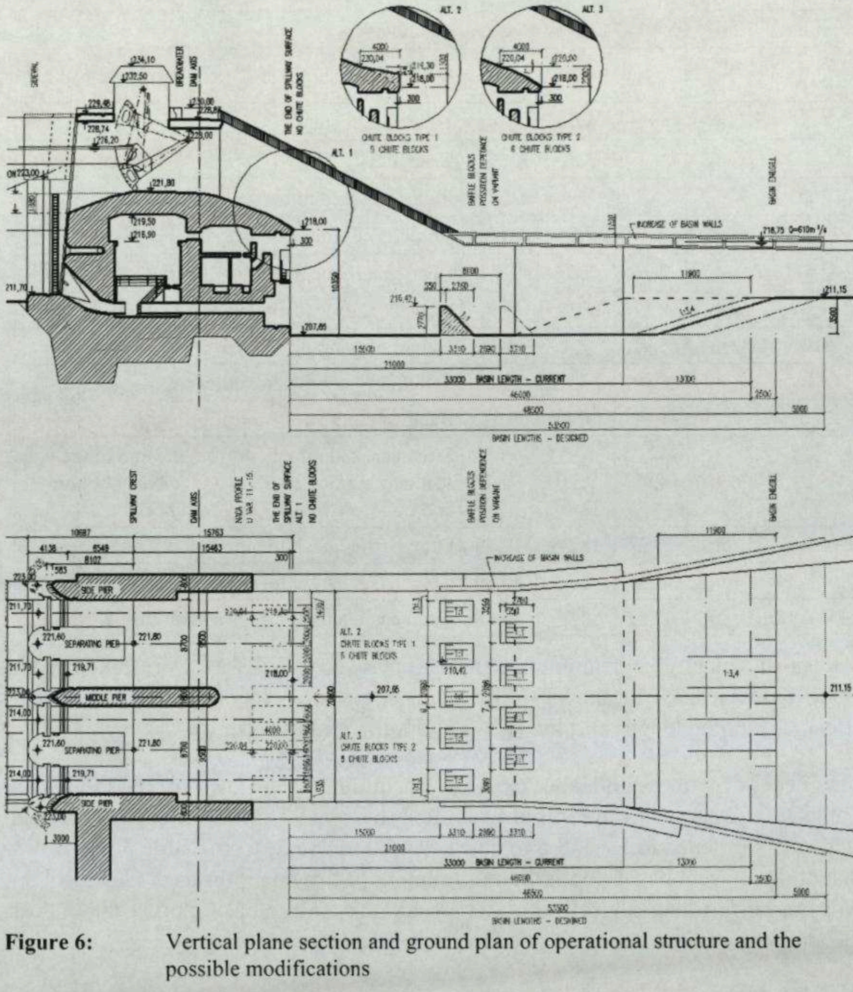

Figure 6: Vertical plane section and ground plan of operational structure and the possible modifications

Baffle blocks in the basin have a crucial effect on extent of bottom deformations irrespective of the basin length. Optimal number of baffle blocks is five according to the evaluation of test results. Baffle blocks may be laid out in one or two rows. Two rows arrangement is more suitable when using the chute blocks, see Fig. 7.

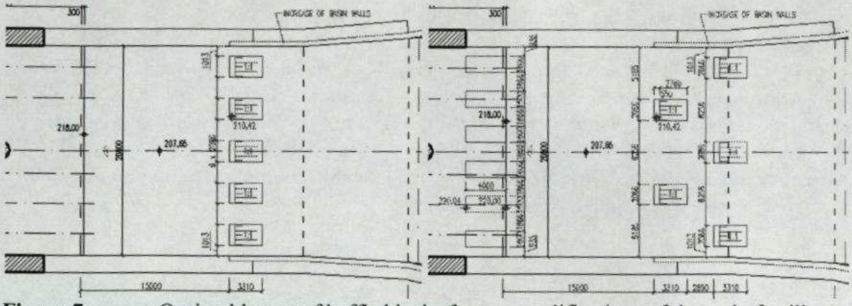

Figure 7: Optimal layout of baffle blocks for two modifications of the end of spillway

Dimensions and position of baffle blocks in the basin were designed similar to the baffle blocks in the Peterka's basin Type III published by Čábelka, Novák (1964), Seturk (1994). Shape and position of baffle blocks are shown in Fig. 6. Fifteen variants of number and distribution of baffle blocks combined with the end of spillway surface modifications and length of basin values were tested. Criteria to find the optimum stilling basin modification were the extent of deformations of the bottom of the channel downstream of the sill. Chute blocks combined with two rows of baffle blocks in the basin contribute to the decrease of the escapade channel deformations. The effect of designed chute blocks on effective basin length and dissipation of energy in the basin was not proved. The use of chute blocks considerably increases the aeration of stream entering the space above the basin and spreads more uniform load on it's bottom. The use of the chute blocks type 2 was proved as more suitable. Due to the spray formed by water impinging on baffle blocks in basin a full concrete barrier about 1.2 m high was added on top of the basin walls. Further modifications of basin walls were not necessary, even when the basin prolongation led to it's new partially divergent plan.

With tailwater depth t = 5.6 m and discharge $Q = 610\ m^3.s^{-1}$ the stream velocity at the basin sill will be $v = 4.0\ m.s^{-1}$. These parameters are decisive for design of channel wall protection downstream of the sill. Heavy riprap 15 - 20 m long with concrete filling in an approximately 5 - 7 m long section was recommended.

5 References

Boor, B., Kunštátský, J., Patočka, C. (1968): Hydraulika pro vodohospodářské stavby. SNTL, Praha 1968

Čábelka, J., Novák, P. (1964): Hydrotechnický výzkum II. SNTL, Praha 1964

Galc, F., Janků, O., Bubeník, M. (2003): VH soustava Vranov – Znojmo, převedení N-letých průtoků, studie. VODNÍ DÍLA TBD a. s., Brno 2003

Ladson, Charles L., Brooks, Cuyler W. Jr., Hill, Acquilla S. (1996): Computer Program To Obtain Ordinates for NACA Airfoils. National Aeronautics and Space Administration, Langley Research Center - Hampton, Virginia 1996

Seturk, F. (1994): Hydraulics of dams and reservoirs. Water resources publications, Colorado USA 1994

Stara, V., Šulc, J. (2004): Hydrotechnický modelový výzkum VD Znojmo. Závěrečná zpráva VUT v Brně FAST ÚVST-LVV, Brno 2004

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