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MODEL INVESTIGATIONS OF SIDE CHANNEL SPILLWAY OF THE ZŁOTNIKI STORAGE RESERVOIR ON THE KWISA RIVER

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PROBLEM STATEMENT

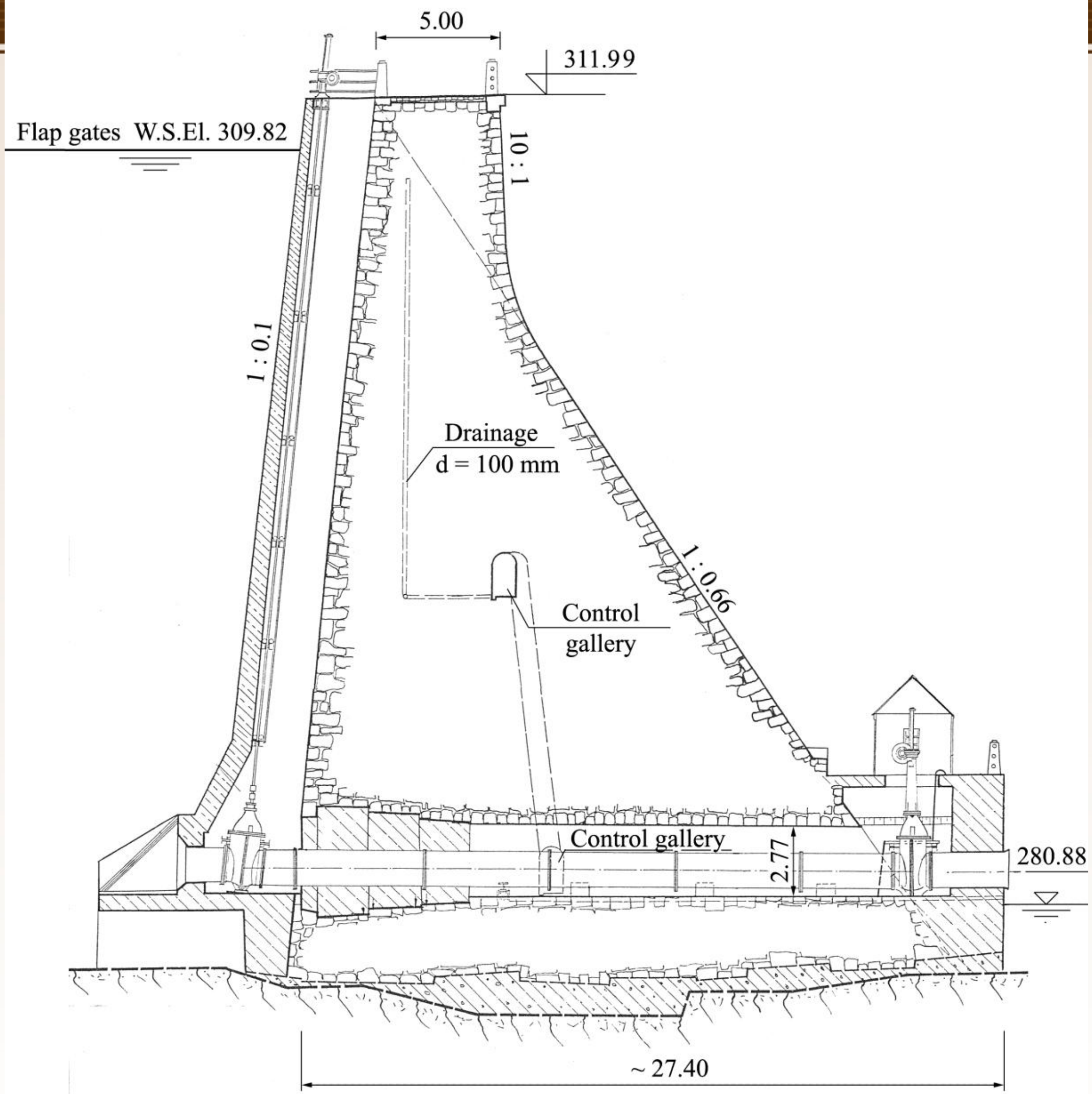
- Regulations of Minister of Environment from 1997 in matter of technical conditions for hydro-engineering structures and their location
- Złotniki dam's technical class of importance has changed from II to I
- New computational discharges – design and so called controlled for I class
- Question has arised – if new discharges will safely pass through reservoir and outlet installations ?
- Authors tried to verify and to solve this problem

OBJECT CHARACTERISTICS

- Złotniki dam – hydro-engineering structure of I class of importance

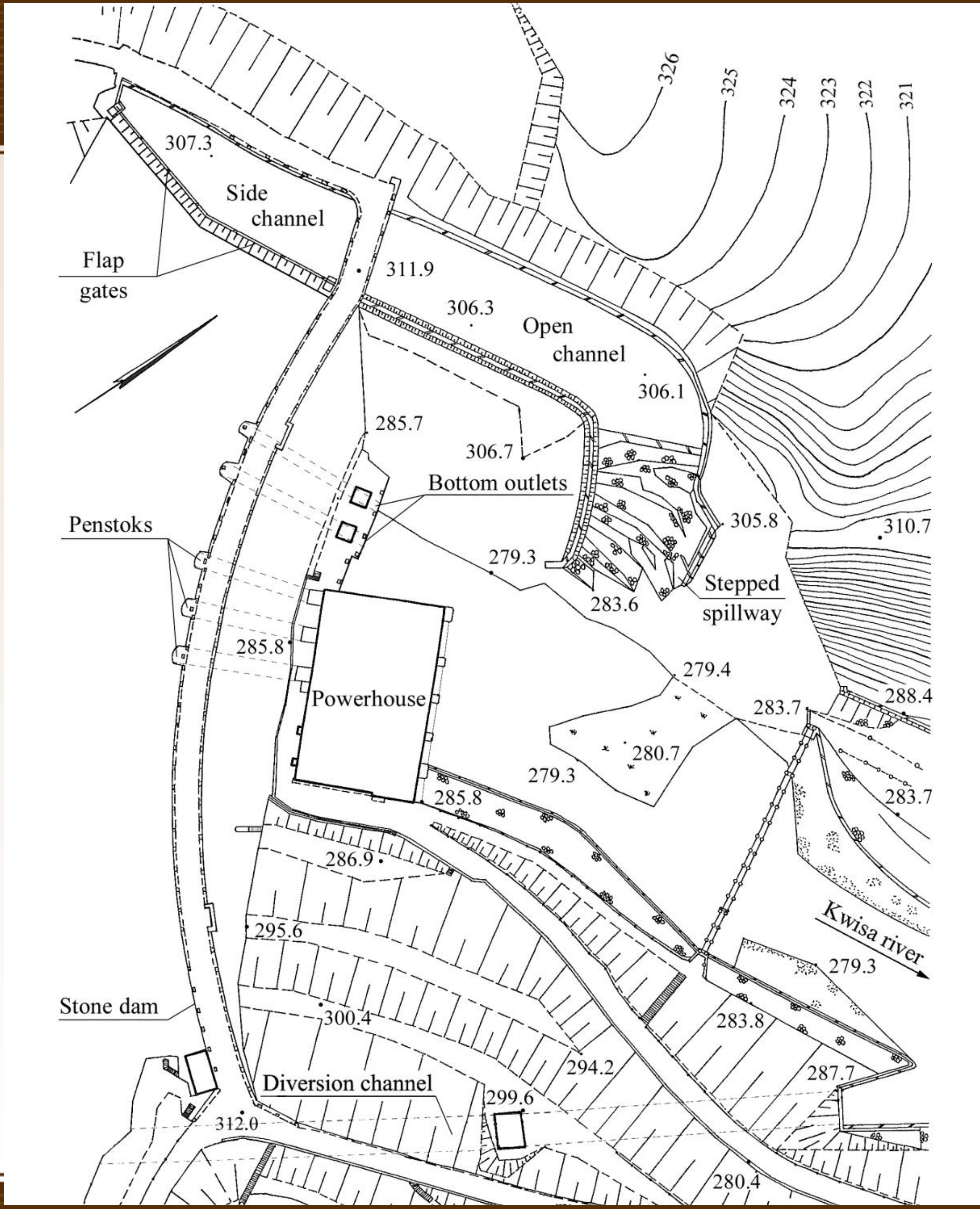
  (according to Polish regulations)
- location: km 95+540 of the Kwisa river course
- dam: arch body made of broken stones in concrete
- maximum height – 36 m
- capacity V_{zb} = 12,10 mln m³.
- multitasks storage reservoir
- basic tasks: flood protection and energetic purposes

DAM
CROSS - SECTION



OBJECT CHARACTERISTICS

FUNCTIONAL PLAN OF ZŁOTNIKI RESERVOIR



OUTLET INSTALLATIONS

- side channel spillway
- multistage cascade,
- diverse channel
- bottom outlets

OUTLET INSTALLATIONS
SIDE CHANNEL SPILLWAY

OUTLET INSTALLATIONS
MULTISTAGE CASCADE

OUTLET INSTALLATIONS
DIVERSE CHANNEL

OUTLET INSTALLATIONS

Side channel spillway:

- two sections of 22,50 m length each, divided by concrete pillar

Multistage cascade:

- hewn in the rock massif

Diverse channel:

- two conduits with 1400 mm diameters

Bottom outlets:

- two conduits with diameters: 1400 and 1000 mm

Electric power station:

- three conduits with diameters: 1800, 1700 and 1600 mm,
- three turbines, installed electric power output 4,42 MW, capacity 20,29 $m^3 \cdot s^{-1}$

OUTLET INSTALLATIONS

Capacity ability for water level dammed up to dam crest

- **side channel spillway – estimated about 380,80 $m^3 \cdot s^{-1}$,**
- **own analytical calculations – should equal 250 $m^3 \cdot s^{-1}$ (Machajski, 2009),**
- **bottom outlets – determined on the basis of field measurements: conduit DN 1400 – 31,20 $m^3\ s^{-1}$, for conduit DN 1000 – 15,40 $m^3\ s^{-1}$,**
- **diverse channel – total for two conduits DN 1400 – 56,80 $m^3\ s^{-1}$.**

ANALYTICAL CALCULATIONS

Spillway discharge was determined assuming that it will work in the whole range of expected discharges as not submerged.

Calculations were carried out using formula for free spillway, with straight insert on crest:

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

where:

ε – coefficient of side contraction weir, ε = 1,0

μ – discharge coefficient, μ = 0,654

B – spillway crest length, B = 45,0 m

H_o – energy height calculated with relation to spillway crest

ANALYTICAL CALCULATIONS

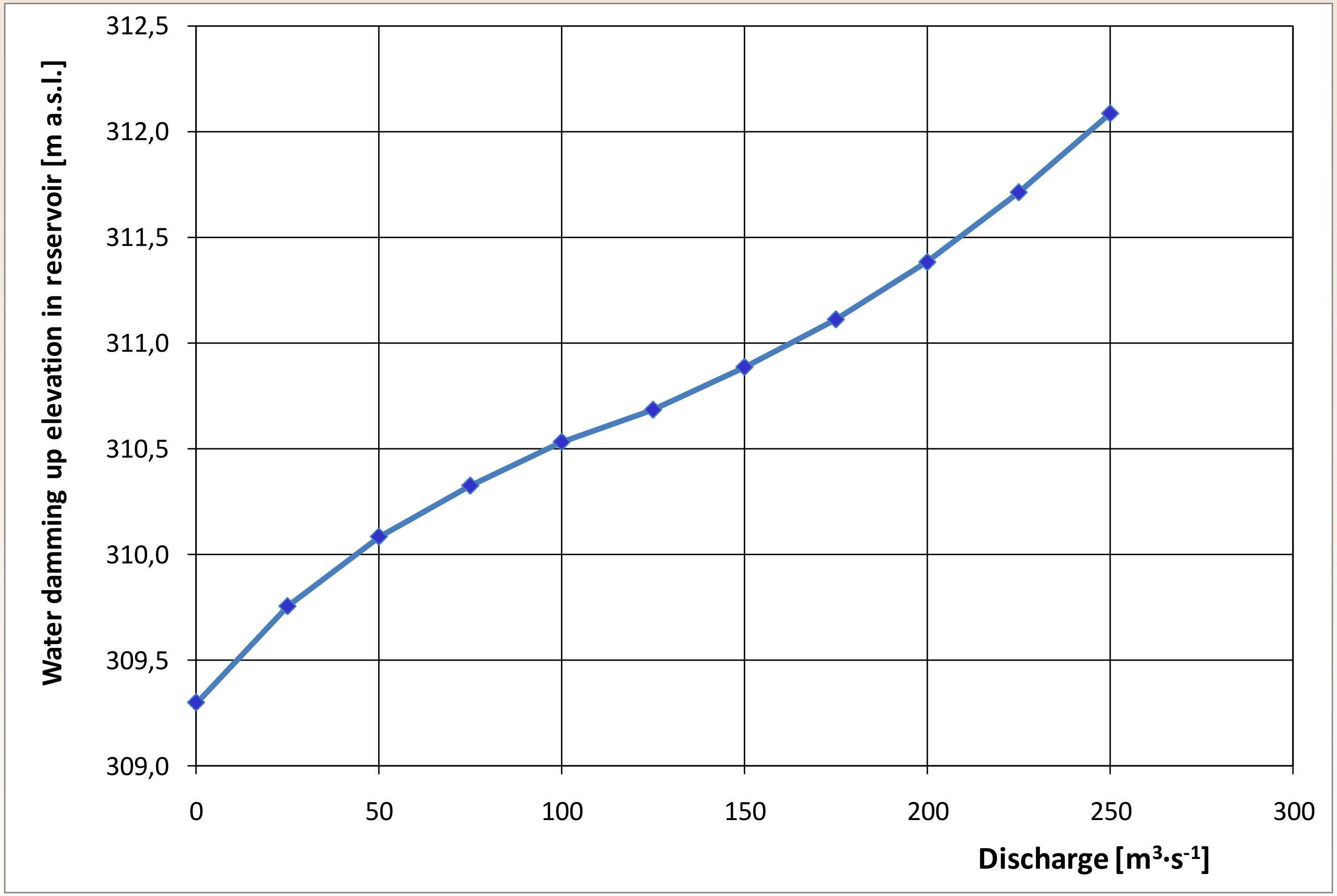
They result from water level with regard to weir crest (309,30 m a.s.l.) and dam crest, determining water layer thickness and the resultant energy of stream.

The calculation results are as follow:

312,00 (dam crest) – 309,30 = 2,70 m → Q = 385,58 $m^3 s^{-1}$

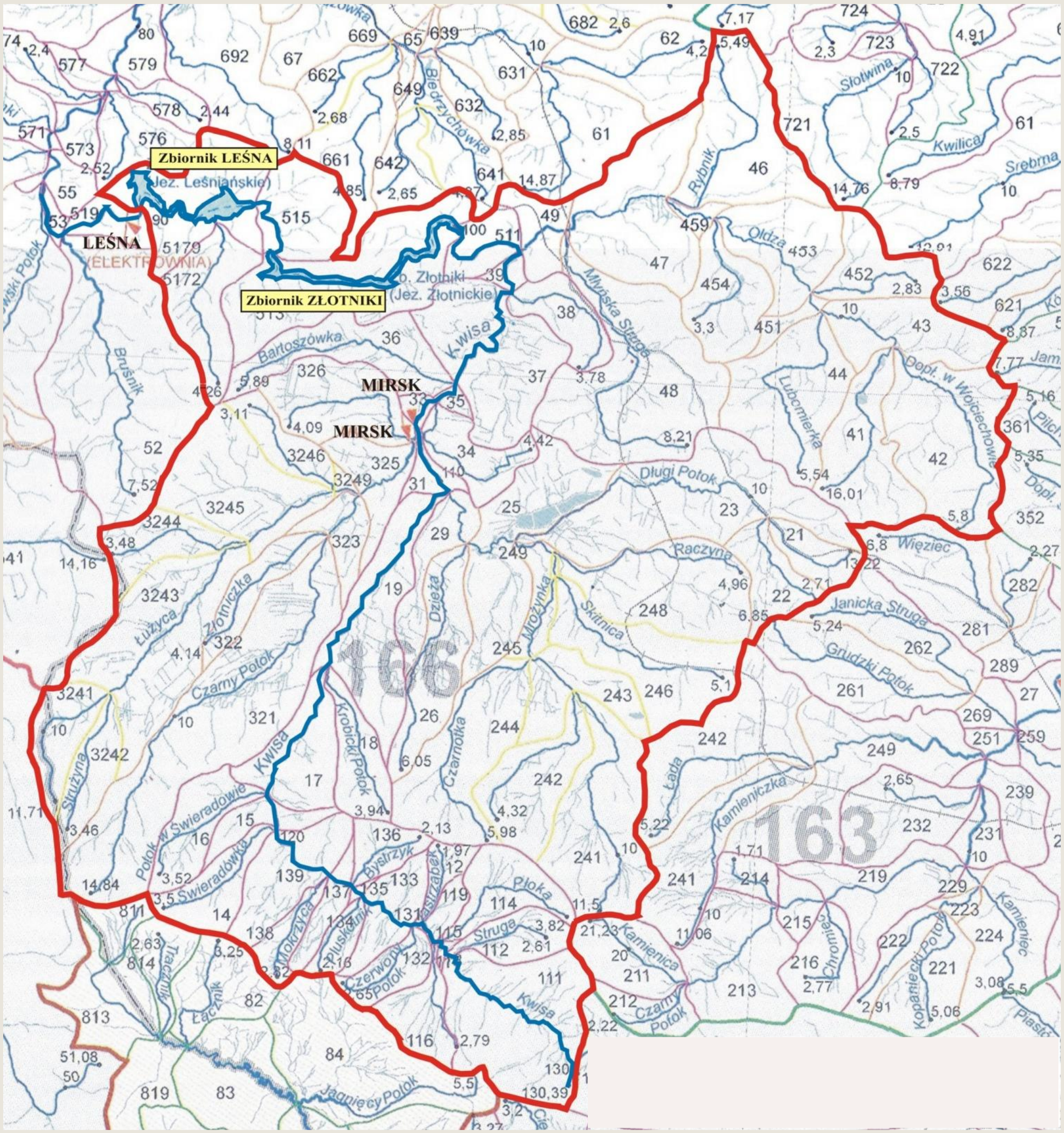
In a further analysis for flume parameters determinations, the discharge equal to 385,58 $m^3 s^{-1}$ was assumed.

ANALYTICAL CALCULATIONS



Hydraulic characteristic of side channel spillway of the Złotniki reservoir for the existing state (analytic)

THE UPPER KWISA RIVER BASIN



HYDROLOGICAL CHARACTERISTICS TO ZŁOTNIKI DAM CROSS - SECTION

Catchment area – 288,14 km²

River length – 36,2 km

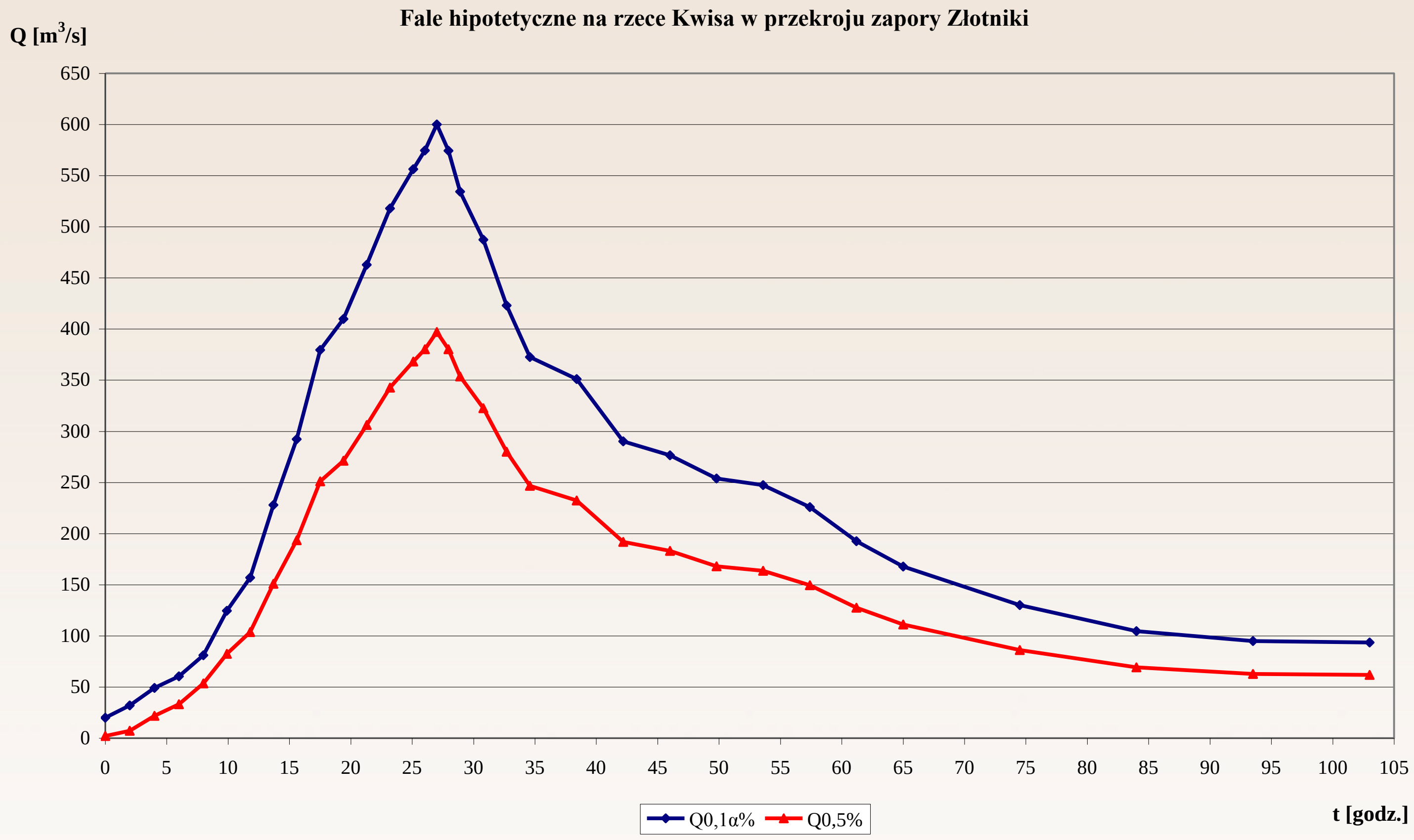
Average slope of catchment – 3,5 % (in mountain), 1 % on foothills

The most frequent occurence of flood waves period: July – October

Characteristic discharges (Polish abbreviations) and maximum discharges with a given probability of exceedance – transferred from Mirsk gauging station:

- **NNQ = 0,214 $m^3\ s^{-1}$ – minimum**
- **SNQ = 0,817 $m^3\ s^{-1}$ – average low flow**
- **SSQ = 4,449 $m^3\ s^{-1}$ – mean**
- **SWQ = 94,07 $m^3\ s^{-1}$ – average maximum flow**
- **WWQ = 323 $m^3\ s^{-1}$ – maximum (1981)**
- **$Q_{50\%}$ = 85 $m^3\ s^{-1}$**
- **$Q_{1\%}$ = 351 $m^3\ s^{-1}$**
- **$Q_{0,5\%}$ = 397 $m^3\ s^{-1}$ (design discharge)**
- **$Q_{0,1\%}{}^{\alpha}$ = 600 $m^3\ s^{-1}$ (controlled discharge)**

HIPOTETICAL WAVES

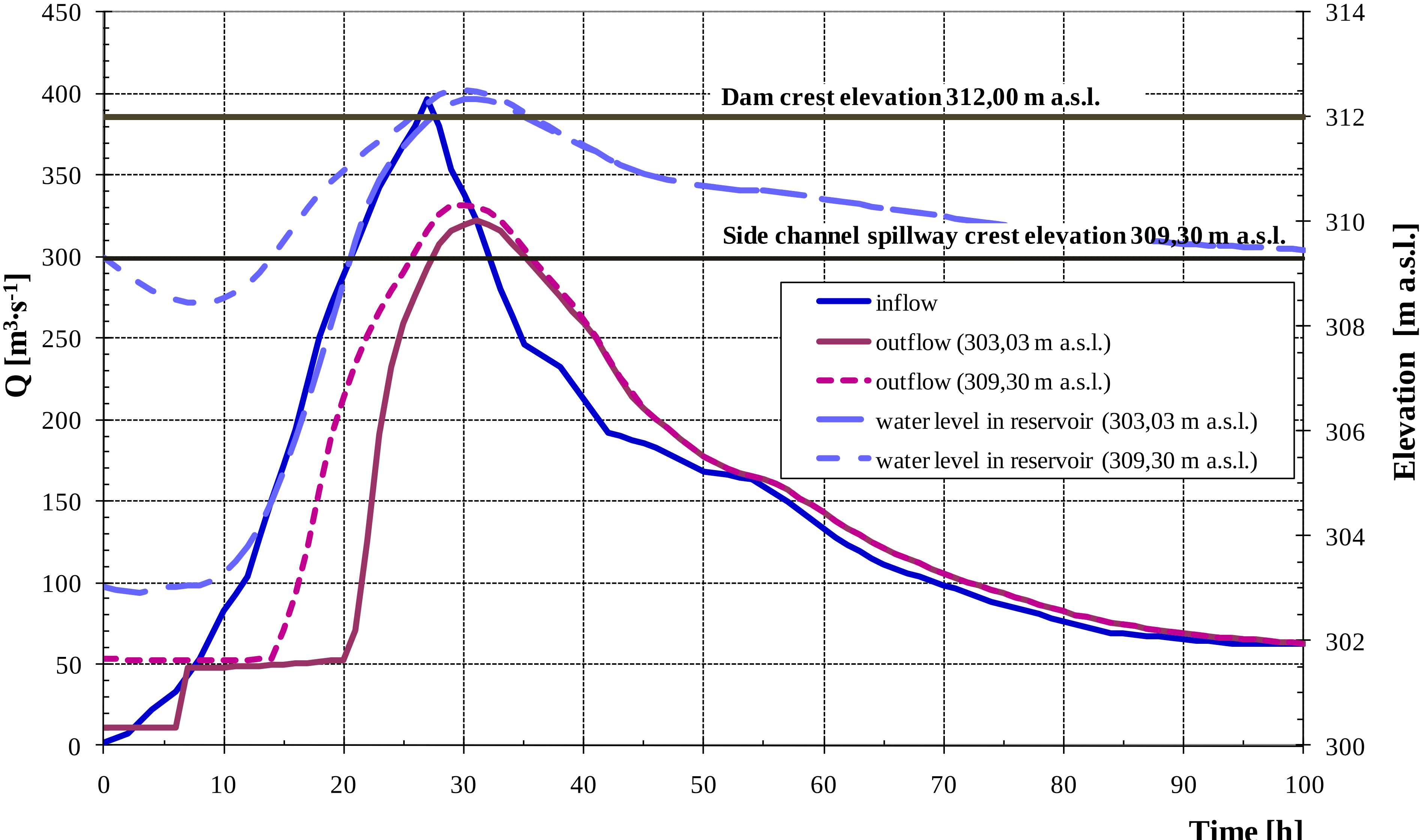


Probability of computational discharges for hydrotechnical structure of the I class of importance, not subjected to destruction due to their overflow (Regulations ME, 2007):

$p_{des.}$ = 0,5 %, $p_{contr.}$ = 0,1 % with the upper extension,

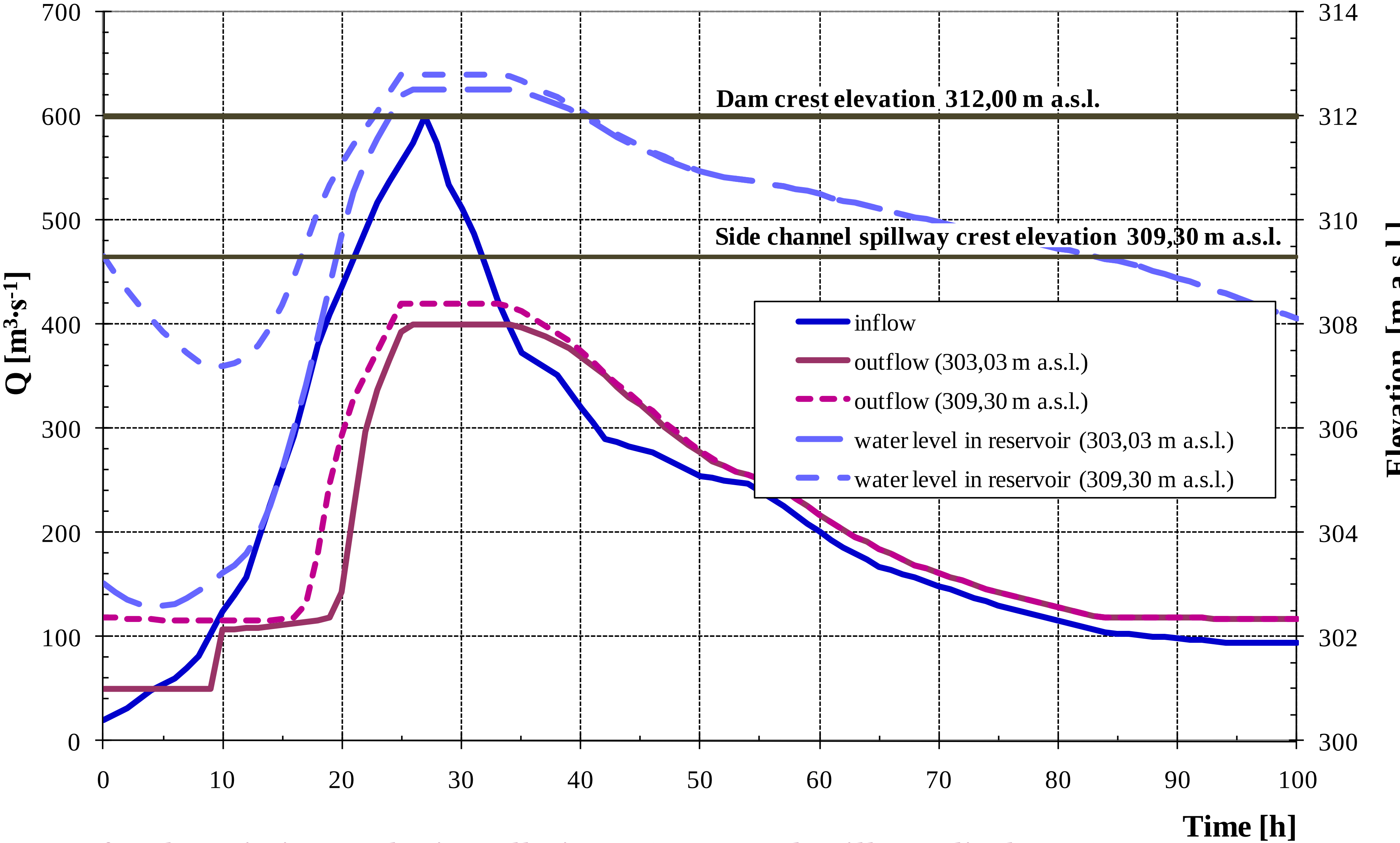
Computational discharges: $Q_{des.} = 397\ m^3\ s^{-1}$, $Q_{contr.}^{\alpha} = 600\ m^3\ s^{-1}$.

$Q_{0,5\%}$ FLOOD WAVE TRANSFORMATION



for the existing outlet installations (corrected spillway discharge), initial levels of water dammed in reservoir 303,03 i 309,30 m a.s.l.

$Q^{\alpha}_{0,1\%}$ FLOOD WAVE TRANSFORMATION



for the existing outlet installations (corrected spillway discharge), initial levels of water dammed in reservoir 303,03 i 309,30 m a.s.l.

ANALYTICAL CALCULATIONS

Calculations of flood waves transformation showed that for existing outlet installations

there is no possibility to preserve a safe dam crest height above the maximum water level in reservoir

equal to 1,0 m .

For flood wave $Q_{0,5\%}$ passage:

- **initial state of water level 303,03 m a.s.l. – water level in the reservoir reaches 312,35 m a.s.l.**

  – 0,35 m above the dam crest

- **initial state of water level 309,30 m a.s.l. – water level in the reservoir reaches 312,50 m a.s.l.**

  – 0,50 m above the dam crest

For flood wave $Q^{\alpha}_{0,1\%}$ passage:

- **initial state of water level 303,03 m a.s.l. – water level in the reservoir will exceed 0,50 m dam crest**
- **initial state of water level 309,30 m a.s.l. – water level in the reservoir will exceed 0,80 m dam crest**

WHAT MEANS A DAM CREST IS OVERFLOW IN EACH COMPUTATIONAL CASE

CONCEPTION OF OUTLET INSTALLATION RECONSTRUCTION

Free flow of computational discharge through existing spillway is not possible:

because flume in its upper part is too shallow and too narrow

Solutions:

- **the flume should be deepened by about 2,50 m with simultaneous set of its width at 15,0 m (under such conditions the assumed discharge of spillway can be obtained for damming water level in reservoir for 312,00 m a.s.l.)**

Drawbacks and advantages:

- **the reconstruction would force a necessity of constructional changes of weir – new static conditions of its work and a necessity of constructional changes of wall on the opposite side,**
- **spillway can be reconstructed without a necessity of reservoir emptying,**
- **its safety is ensured because flood waves can pass through spillway without any negative consequences,**
- **lack of possibility of improving the operating conditions of freshet wave routing (at present the operation of outlet installation is carried out untill a damming level in reservoir will reach dam crest, i.e. an outflow will be somewhat over 100 $m^3 s^{-1}$.**

MODEL INVESTIGATIONS

The model investigations were carried out for the whole outlet installations of reservoir.

First, the aim of model investigations was to verify a proposed reconstruction of a side channel spillway.

Program of investigation consisted of measurements and observations for determination:

- **capacity ability of side channel spillway, in terms of the changes of levels of damming up in the reservoir ranging from 309,30 m a.s.l. to dam crest from upstream equal to 312,00 m a.s.l.,**
- **operating conditions of multistage cascade during greater discharge passage,**
- **operating conditions of arch bridge at passage section from flume to multistage cascade,**
- **impact of introduced in minimal range – possible to comparatively easy realization, spillway constructional changes.**

MODEL INVESTIGATIONS

- spatial model at scale 1 : 40,
- included all important for water passage component elements, in this side channel spillway with multistage cascade,
- for studied problem – water over weir crest and in a flume flows predominantly due to gravity force, therefore the Froude criterion of similarity was applied to calculate the appropriate forces occurring both in nature and in the model,
- for studied phenomenon also a certain contribution has a viscous force – for limitation to minimum an influence of viscous force, the same kind of flow in the model as in a nature had to be assured,
- according to input data, under the conditions of computational discharges passage the conditions of turbulent flow occur, therefore it is required that on model turbulent flow should also occurred with relatively great the Reynolds number – under model conditions and at variable discharges the Reynolds numbers from Re = 15 000 for discharge $Q_{0,5\%}$ to Re = 35 000 for discharge $Q^{\alpha}_{0,1\%}$ were obtained.

MODEL INVESTIGATIONS

In maintaining the similarity of phenomena observed both in model and in a nature, the design of model with appropriate roughness seems of a prime importance.

According to input data all elements of external outlet installation are made of concrete, hence roughness coefficient *n* is equal to 0,020.

Because the model scale is equal to $\alpha_l = 40$, therefore after counting $\alpha_n = 1{,}8493$.

The resultant roughness coefficient of the material that should be used for essential details of the model outlet installations should be equal to:

$$n_M = \frac{n_N}{\alpha_n} = \frac{0{,}020}{1{,}8493} = 0{,}01081$$

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

MODEL INVESTIGATIONS

The aim of changes introduced into the model was to check a possibility of capacity ability improvement of spillway, i.e. to reconstruct or to build some elements, that were not present on object earlier.

Changes introduced into the model consisted in setting:

- **Change I – existing state, to confirm a consistence of currently valid for spillway a discharge curve with a calculated one**
- **Change II – a constant width of flume,**
- **Change III – correction the height configuration of flume bottom, i.e. its lowering by about 1,50 m,**
- **Change IV – concerned to check an impact of dividing pillar and steering walls on the conditions of water flow through side channel spillway,**
- **Change V – deepening of flume bottom about 2,50 m.**

VIEW OF SIDE CHANNEL SPILLWAY MODEL FOR EXISTING STATE

CHANGE I

VIEW OF SIDE CHANNEL SPILLWAY MODEL AFTER INTRODUCING CHANGES

CHANGES II, III, IV

VIEW OF SIDE CHANNEL SPILLWAY MODEL AFTER INTRODUCING CHANGES

CHANGE V

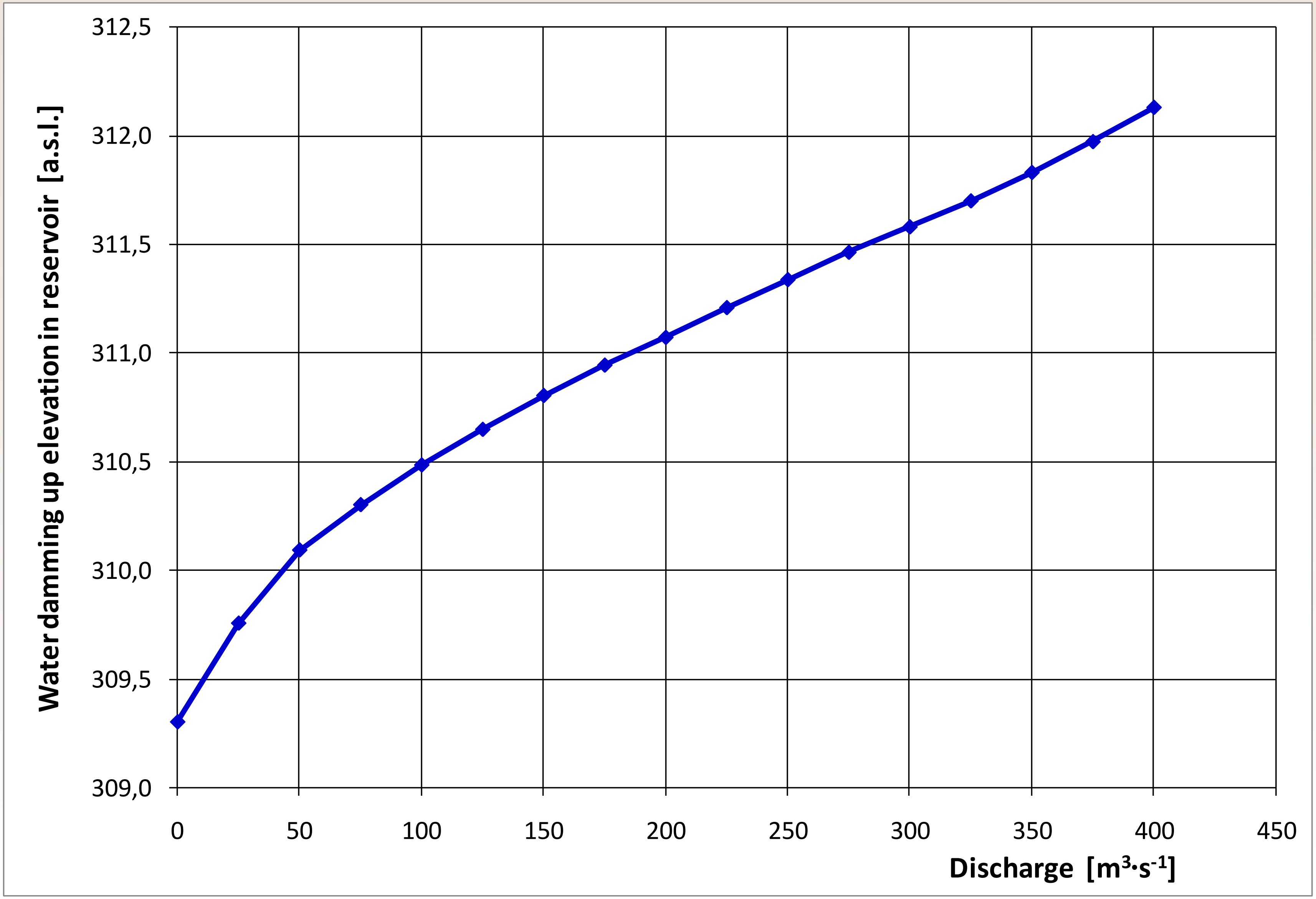
MODEL INVESTIGATIONS RESULTS

- **Change I – for 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 125 m^3s^{-1} and the water damming up level in reservoir reaching 310,70 m a.s.l. The maximum spillway capacity ability under these conditions was 245 m^3s^{-1} at the water damming up level of 312,00 m a.s.l., i.e. water reaches dam crest.**
- **Change II – a 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.**
- **Change III – allowed the spillway operating conditions to be improved and caused the expected changes of flow conditions in weir, which occurred at discharge of 175 m^3s^{-1} , but did not increase the expected spillway capacity ability.**
- **Change V – increased the spillway capacity ability to the expected 400 m^3s^{-1}. This took place for water damming up in reservoir to 312,13 m a.s.l., which slightly exceeded the dam crest from downstream.**
- **Change V – the spillway work conditions are evaluated as correct. Any intensive 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 contour and gently flows in flume cross-section in the cascade direction.**

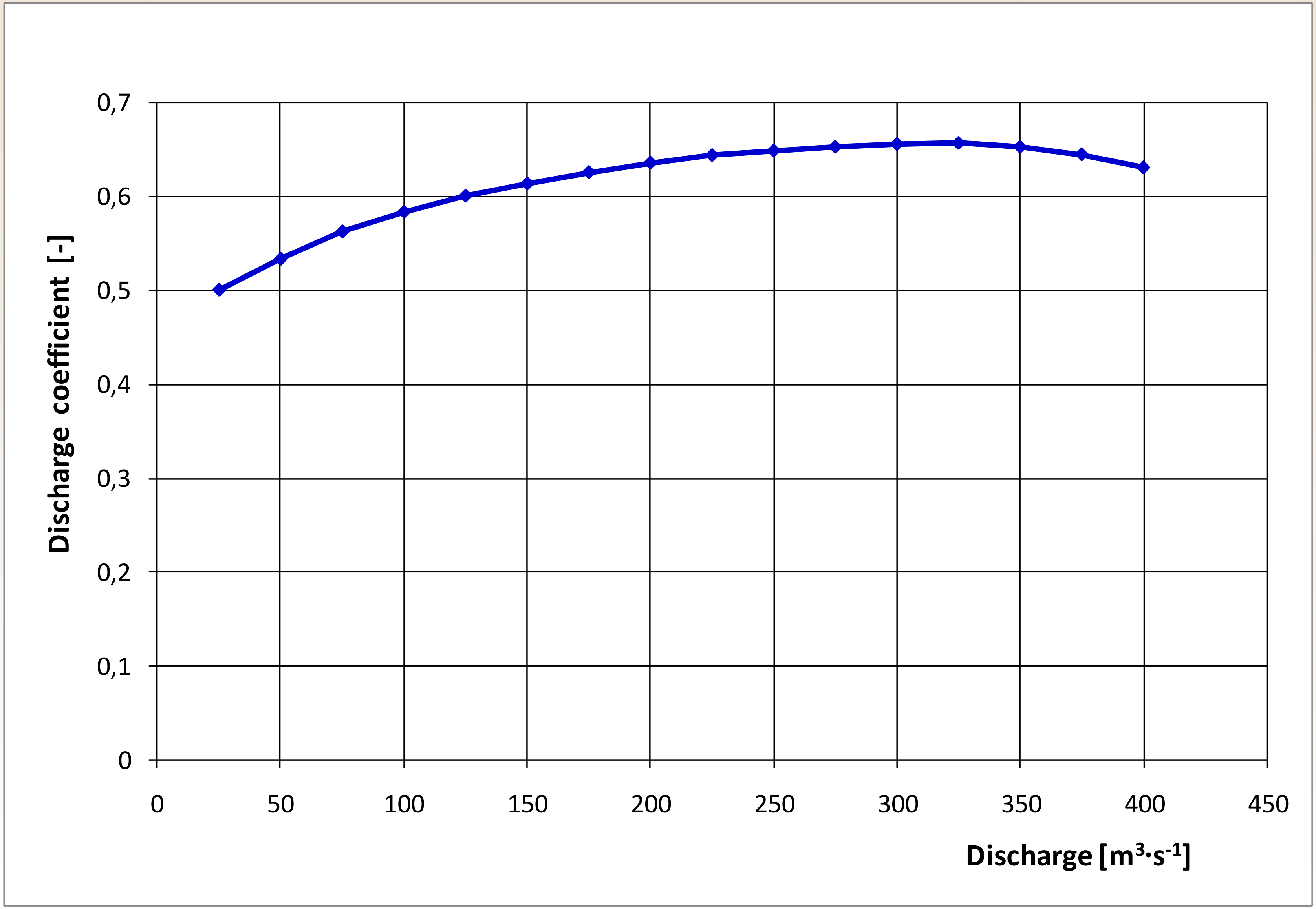
MODEL INVESTIGATIONS RESULTS

The spillway capacity ability obtained is close to that calculated, mainly from the point of view of water quantity that under given conditions can overflow by weir, provided that in the whole range of discharges it works as not submerged. This corresponds to discharge of about 385 m^3s^{-1}, that could be obtained for existing spillway lay-out in plan, its crest length, weir shape and flume parameters

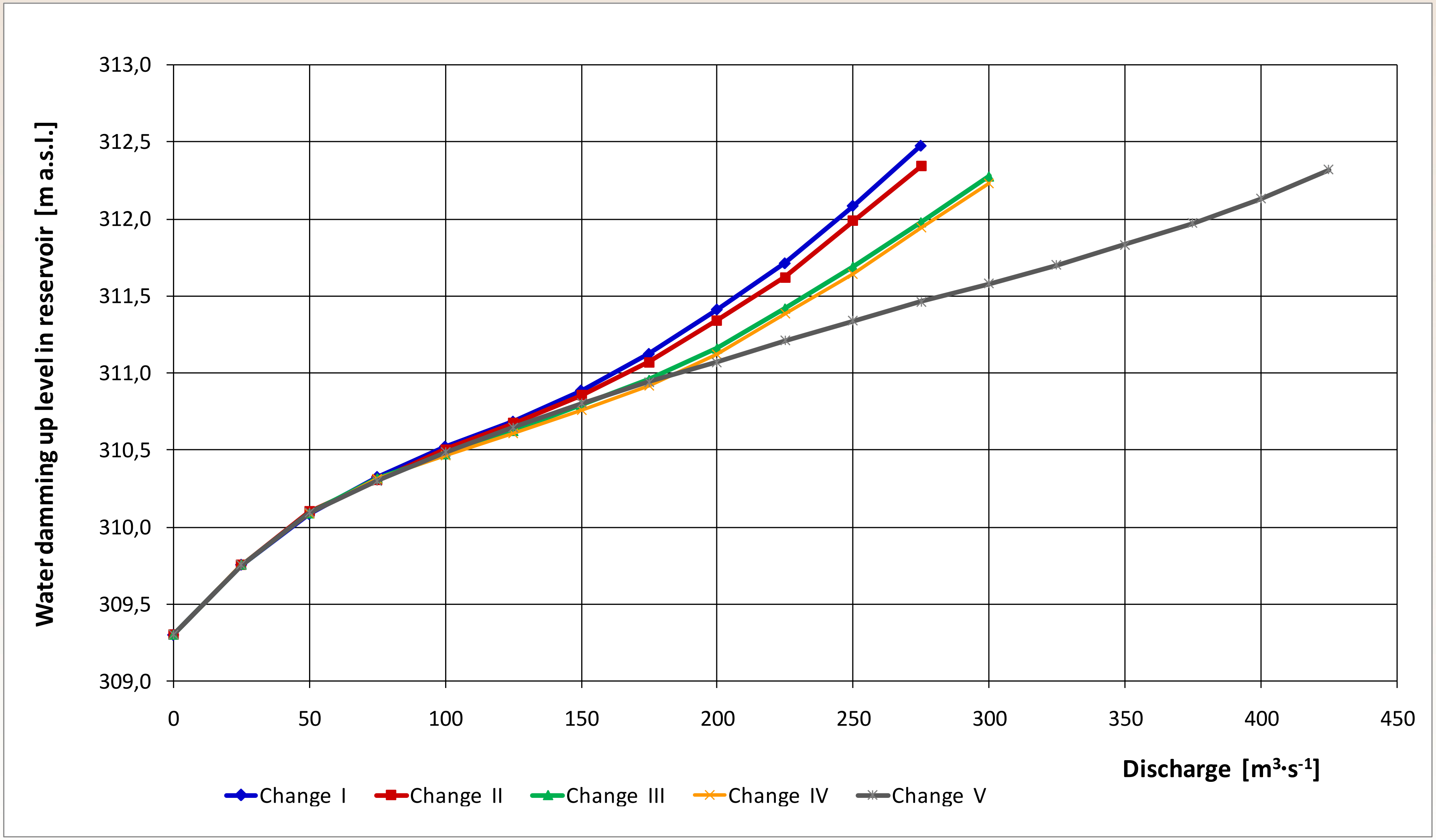
HYDRAULIC CHARACTERISTIC OF SIDE CHANNEL SPILLWAY FOR PROPOSED SOLUTIONS



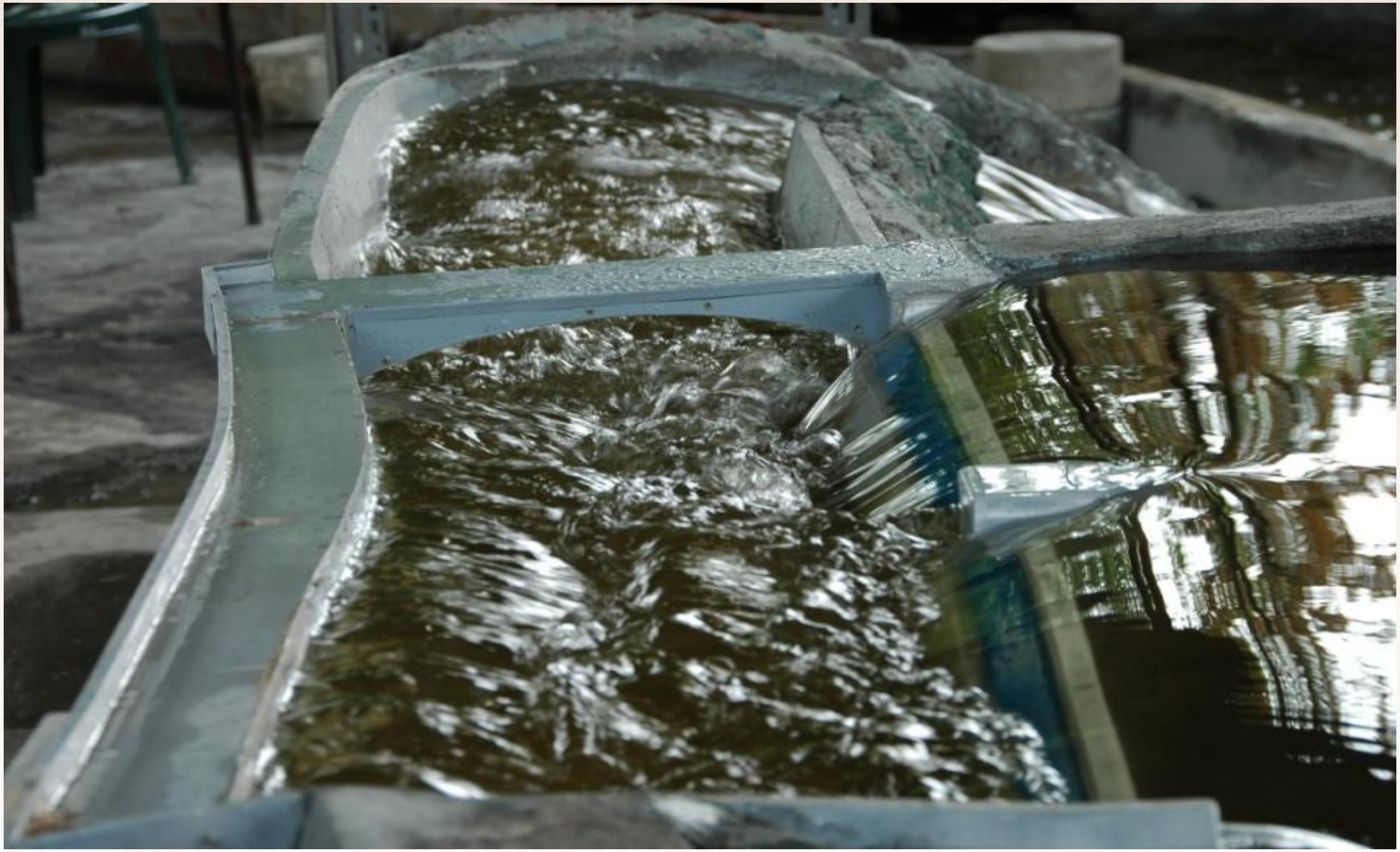
CHANGES OF DISCHARGE COEFFICIENT OF SIDE CHANNEL SPILLWAY FOR PROPOSED SOLUTIONS



COMPARATIVE HYDRAULIC CHARACTERISTIC OF SIDE CHANNEL SPILLWAY FOR CHANGES INTRODUCED INTO THE MODEL



SPILLWAY WORK UNDER CONDITIONS OF DISCHARGE PASSAGE OF 400 m^3s^{-1}



SUMMARY

Proposed solutions that correct the configuration of the flume of side channel spillway were verified during model tests. This allowed to draw the following conclusions:

1. **In order to improve the exploitation safety of Złotniki reservoir it is necessary to introduce the proposed changes in existing state of flume of side channel spillway.**
2. **A change in side channel spillway solution consists in flume bottom deepening by about 2,50 m in relation to present depth, and increasing of its width up to 15 m.**
3. **A flume deepening increases the capacity ability of this installation by about 135 m^3s^{-1}.**

THANK YOU FOR YOUR ATTENTION