NUMERICAL MODELLING OF THE INLET STRUCTURE OF KISDELTA FLOOD CONTROL RESERVOIR

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ABSTRACT. -Numerical modelling of the inlet structure of Kisdelta flood control reservoir. This study aims to introduce a methodology on how the targeted use of a 1D and a 2D hydraulical numerical model can provide indispensable data for the design of an flood control structure. With the numerical modelling of the Körös-Valley river system (1D) and the inlet structure of Kisdelta flood control reservoir on the river Crişul Alb/Fehér-Körös (2D) the main geometrical data and hydraulic parameters were defined.

Key-words: One-dimensional (1D) calculations, two-dimensional (2D) calculations, flood control reservoir, inlet structure, Crişul Alb, Fehér-Körös

1. Introduction

There was a great progress in water management field in the past 30-40 years. The major achievements in applied science enable us to design a control structure not only based on hydrological calculations or physical modelling. Calculations with hydraulical numerical models give further information which – we can say without any exaggeration – is indispensable for any design now days. In this study we explain the numerical model calculations for the modernisation of the Kisdelta flood control reservoir's inlet structure on the river Crişul Alb/Fehér-Körös (Rátky-Rátky 2010/a). Firstly we describe the one dimensional (1D) numerical model results followed by the application of the two dimensional (2D) model results and the design data provided by them.

2. One-dimensional (1d) hydraulical calculations

The objective was to define the main geometrical, hydraulical parameters for the inlet structure in order to achieve the needed water level decrease on the river system by opening the Kisdelta flood control reservoir. The length of the river section and the abandonment of the project determined that it can only be simulated with a 1D numerical model.

Based on the 1D calculations we could specify: inlet sill elevation; required size of the openings; maximal discharge capacity; impact distance of the reservoir opening for assumed operating conditions.

The following question can also be answered: with different possible design and an assumed operating conditions: how can the required water level reduction be achieved.

Modelling the outlet process: how long does it takes to empty the reservoir to the sill level; what is the maximal outflow discharge.

The methodology for the calculation is the numerical solution for the basic equation of free surface, gradually changing, one-dimensional, unsteady flow. This is well known and widely applied in the water management sector. We have already used in the early 80's the results of a one dimensional model calculation for planning and designing the Körös Valley flood control reservoir. For this reason we do not elaborate this procedure, which is today part of technical university curriculum.

2.1. Main size of the inlet structure and impact distance on floodpeak reduction of the reservoir

Basic parameters

Hydraulic simulations were carried out for the 1995 flooding conditions – as boundary condition. Hereafter it will be considered as the design hydrological load. The considered river sections for the purpose of our calculation are on Fehér-Körös from township Kisjenő to the estuary; on Fekete-Körös from Zerind to the estuary; and on the Kettős-Körös to the cross section with water level gauge at Köröstarcsa.

The geometrical and hydrological data was provided by the experts of the KÖRKÖVIZIG (Körös Region Environmental and Water Directorate). Such as riverbed geometry, river cross sectional data, parameters at water level gauges at various locations, Kisdelta rule curve. They also provided the needed water level, discharge data for the calibration and data for the initial and boundary conditions.

The model was calibrated with great accuracy. The difference between the measured and calculated water levels was maximal ±5 cm with the exception of one cross section with water level gauge (at Doboz). The calibration was acceptable for discharge as well. The smallest difference in flow rates was ΔQ=0.5 % near the initial cross section at Gyula, and the largest difference at Sarkad was 17 % ($\Delta Q=|Q_{calculated}-Q_{measured}|$).

Test cases

Main parameters of the 17 tested cases are the following:

Changing characteristics	range	unit	Nº of cases
inlet sill level	88.10 – 88.60	m a.B.s.l.	4
sector gate height	4.8 – 6.3	m	4
water level at opening	design water level-0.1m – -0.6 m	m	3
Nº of 8 m openings	2 – 5	db	4
Operation	'maintaining constant water level' 'dam explosion'		2

All potential combinations of cases could not be calculated because the number such cases is as high as 715. Considering the technical and financial aspects, there are less variations feasible.

Calculation results

The description of the original project stipulated that „The planned inlet structure has to be developed to conform the previously calculated impact distance affected by opening the reservoir on the Körös River System." (Minutes 2009) The needed impact distance and the water level decline data are from calculations of 1998, carried out for the Kisdelta flood control reservoir's Design Supporting Report (Rátky 1998). Designs were only considered if impact distances and reduced water level data could be kept as established earlier.

Out of all the investigated cases seven were found which achieved the required water level drop. Further experience of design engineers and other considerations for example economical reasons, uncertainty in the forecast, operation time requirements, the most appropriate design was found to be the following, with:

$$Z_{sill} = 87.60 \text{ m a.B.s.l.}; \quad 4*8 \text{ m wide opening}$$

An inlet structure with such main dimensions has a maximal discharge capacity – calculating with the design flood of December 1995

$$Q_{reservoir} = 280 \text{ m}^3/\text{s}.$$

We note that our decision was based only on hydraulical considerations (differences in water levels), however for the final decision other aspects should be considered such as:

Cost effective considerations based on actual calculations;

Reliability of forecast and their feasibility; More accurate information on operations being done on the Romanian section of the river and it should also be considered that the utilization of the Kisdelta flood control reservoir is not the only way to protect against flood. All the Körös-valley reservoirs must be studied, designed and operated together in order to take their interaction into account.

3. Two-dimensional (2d) hydraulical calculations

The above mentioned calculations could not provide sufficient detailed and elaborate results because of the cross sections' distance and the one-dimensional limitations. To calculate the flow conditions in the vicinity of the inlet structure a more accurate hydraulical calculation is needed. A 2D numerical model can provide necessary data for design such as water depth, depth averaged velocity components at infinite number of points. With the results of the 2D calculation the following aspects can be specified: how to direct the water inflow – location,

length, height and angle of the sidewalls; bed up- and downstream of the inlet structure – bed levels, length of area covered with concrete or rip-rap, energy dissipation – size of the stilling basin or the number, location, height and angle of the ground sills.

3.1. Bases for the mathematical approach and the numerical solution

We refer our literature on the mathematical bases and the method of the numerical solution. The basic equations are the depth averaged three dimensional Reynolds equations which are valid for viscous, incompressible fluid and give the time averaged turbulent motion at a certain spot. The initial Reynolds equations were developed from the Navier-Stokes equations and can be found in our university notes and literature (i.e. Németh 1963, Liggett 1975, Abbott-Basco 1989). More information can also be found in the following study (Rátky-Rátky 2010/b).

We have used for (2D) calculations the CCHE2D depth averaged model, which was developed in America (US) (Zhang 2006).

The numerical solution is derived with a finite element method which is described in details in a study by (Jia&Wang 2001). The area of our interest was covered with a mesh of 250x660 elements with varying size in both directions: $\Delta x \approx 0.1\text{-}2.8$ m and $\Delta y \approx 0.06\text{-}1.1$ m.

The results of the calculation gave the water depth at 165 000 nodes and depth averaged velocity components in x, y directions with relation to time.

For the numerical solution of the basic equations a numerically closed system is needed. Besides some not detailed relations geometrical data, **n** Manning roughness coefficient and the boundary conditions are indispensable.

3.2. Calculaton of hydraulical characteristics in the immediate proximity of the inlet structure during the filling process

Basic data

Geometrical data

The area of our interest and the majority of the inlet's geometrical dimensions were identical for all of our cases. The following data were available to us:

- AutoCAD drawings of the immediate proximity of the inlet structure, top and cross sectional view based on the preliminary plans of the Konstruktőr Ltd. (See Figure 1 where the final design is also presented based on our calculation results.)

- River Fehér-Körös bed geometry and the uncovered floodplain geometry were estimated on the basis of the data provided for the 1D model, cross sections at 2+480 and 2+600 rkm.

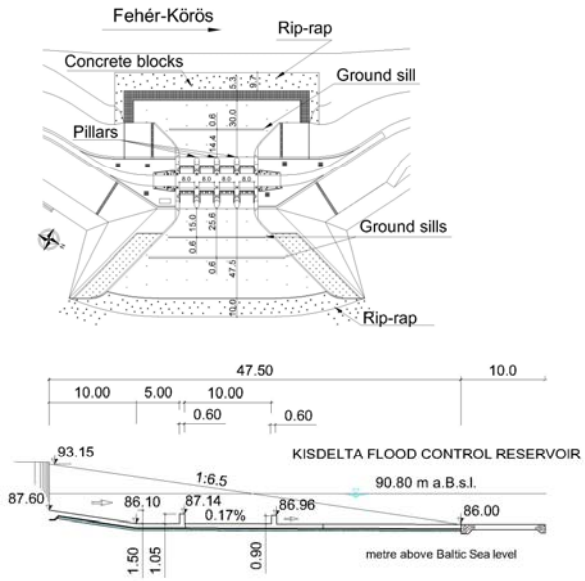

Figure 1. Top and cross sectional view, with final design parameters

Our investigation limited to an area of ~55 000 m^2 (see Figure 2.). The represented area of the model was:

- 70 m downstream form the pillar end of the inlet structure in the reservoir,
- to the sides: starting 15m from the pillar end, with the angle of 60° a trapezoid shape configuration;
- on the upstream side: the River Fehér-Körös between the cross sections 2+450 and 2+690 rkm and ;
- dyke on the left bank of the River Fehér-Körös.

Figure 2. shows the riverbed elevations (topographical map) developed form a digital terrain model. It can be seen the lower riverbed elevations of Fehér-Körös, the sloping ground of the floodplain, the heights of the dykes, the planned side walls with diminishing height and the planned vertical ground sills of the stilling basin.

Calibration of the model

When a structure has not been constructed the classical method for model calibration can not be used, i.e. using previously measured data. We can justify the selection of the appropriate parameters by comparing - with given discharge capacity – the upstream water level elevations calculated using 2D models with the results of the 1D calculations. Furthermore comparisons can be done with physical model measurements and with the results of theoretical hydraulical calculations.

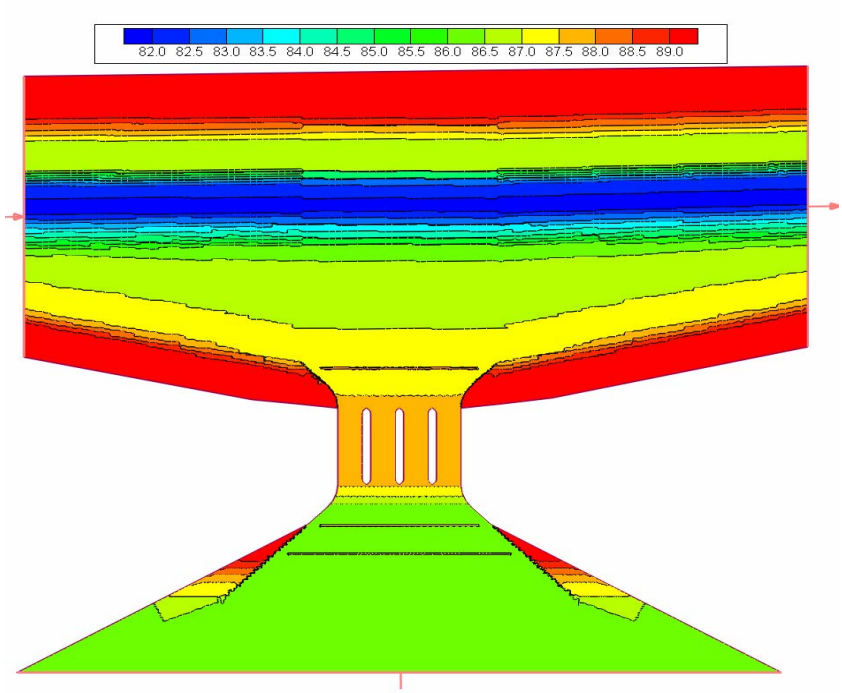

Figure 2. Topographical map developed from digital terrain model, **m a.B.s.l.**

Based on the 1D model calculations the Kisdelta flood control reservoir inlet's maximal discharge capacity and the upstream water level with design conditions (and Z_{sill}=87,60 m a.B.s.l.; sector gate height 5.3 m; 4 openings; opening 0.5 m before reaching design flood water level) $Q_{max,reservoir}$=281 m^3/s, $Z_{upstream}$= 91.01 m a.B.s.l

Based on the 2D model calculations we stipulated flow rates of 280 m^3/s with the boundary conditions. By the suggested parameters for construction the following upstream water level was calculated:

$$Q_{max,reservoir} = 280\ m^3/s, \quad Z_{upstream} = 90.96\ m\ a.B.s.l.$$

It can be seen that the water level resulted form the 2D model is within 5 cm difference with the 1D model result.

The second review option is comparing results of the measurements of the physical model. They measured values of the upstream water level $Z_{upstream}$=90.83-90.85 m a.B.s.l. on the undistorted (λ=40) physical model for the Kisdelta flood control reservoir inlet structure – without downstream damming effect – in case of 280 m^3/s flow rate. (Vízmérleg-VKKI 2010). The water level difference was 11-13

cm between the 2D model calculation and the physical model measurements. Considering the unavoidable inaccuracies with the measurement process (1.0 mm water surface oscillation in the model result in 4 cm discrepancy in real terms) the physical model can not be calibrated, therefore this magnitude of difference is acceptable.

The third review option is comparing with the results of calculations using theoretical formulas. Almost all the theoretical calculations are using the well known Poleni formula to specify flow rate, but using different mainly laboratory, in situ measurement results to define discharge coefficient, μ-factor. A lot of publications are dealing with this issue, but none of them considers all the main characteristics of the Kisdelta flood control reservoirs inlet structure. By short reviewing the literature and using only equations for side weirs we have chosen to investigate 16 formulas. The discharge coefficient was $M=2/3\ \mu(2g)^{1/2}$ and varied between 1.00–1.60 $m^{1/2}/s$ (Rátky 2004). Using these values for the inlet structure we estimated the discharge coefficient as 1.45 $m^{1/2}/s$. With this value of discharge coefficient and the given flow rate of 280 m^3/s should have resulted in 90.91 m a.B.s.l. water level. We can accept the 5 cm difference considering the approximation of the theoretical formula and the uncertainty in estimating the discharge coefficient. Calculating the discharge coefficient form the 2D model result (280 m^3/s and 90.96 m a.B.s.l.) gives 1.42 $m^{1/2}/s$.

The results can also be compared with measurements from the physical models completed earlier. By the VITUKI laboratory measured value for the discharge coefficient was 1.49 $m^{1/2}/s$ on the physical model for Szamos-Kraszna flood control reservoir's inlet structure with plain stilling basin. (VITUKI 2007/a). For the Hanyi-tiszasülyi flood control reservoir's inlet structure also with plain stilling basin the discharge coefficient was found to be 1.54 $m^{1/2}/s$ (VITUKI 2007/b). Considering the fact that these models were not calibrated as well (because this was not possible) and these values were measured with plain stilling basin conditions, in addition that they analyzed a front weir instead of a side weir, our value of 1.42 $m^{1/2}/s$ calculated from the 2D model seems realistic.

Even though we can not prove with direct measurements the reliability of the 2D model results, based on the above described five comparisons we can say that the model can provide accurate enough data for design such as water levels and depth averaged velocities.

Analyzed cases

We have analyzed 30 different cases what we can not describe in detail here. We only provide information of what geometrical parameters were changed for each test case and the interval of these parameters.

Changing parameters	range	unit	Nº of cases
Number of downstream vertical sills	0 – 3	piece	5
Height of downstream vertical sills	35 – 120	cm	6
Size of the opening by the downstream sills	0 – 31	m	8
Number of upstream sills	0 – 1	piece	2
Height of upstream vertical sill	30 – 70	cm	2
Size of opening by the upstream sills	0 – 3.0	m	3
Number of small sills	0 – 4	db	3
Height of small sills	90 – 135	cm	4
Length of small sills	2 – 3	m	2

Operational data

Initial conditions: For every case the so called. '**cold start**' was the initial condition. For the upper boundary of the Fehér-Körös at 2+690 rkm cross section we have specified values 90.70 or 90.80 m a.B.s.l.. For the downstream boundary at the 2+450 rkm cross section 90.00 or 90.80 m a.B.s.l. respectively. The value of the initial water level was chosen to be 86.60 m a.B.s.l. at the reservoir downstream boundary.

Boundary conditions: We have analyzed steady state conditions. Starting from the initial condition we run the model until the area of our interest came to steady state conditions.

Upstream boundary conditions:

At the Fehér-Körös cross section located at 2+690 rkm: We have specified constant flow rate of Q_{inflow}=570 m^3/s with an exception of one case. We generated this value from 1D model calculation; For the case of (20.) Q_{inflow}=280 m^3/s constant flow rate was the upstream boundary condition. (This value was as well from 1D model calculation).

Downstream boundary conditions:

At the Fehér-Körös cross section located at 2+450 rkm: We have specified constant rating curve with exception of one case. We have estimated the constant rating curve on the basis of measurements during the flood at Gyula in December 1995 until January 1996.For the case (20.) we used closed downstream boundary with (Q_{out}=0 m^3/s).

At the reservoir's downstream boundary: We have specified water level of $Z_{downstream}$=86.60 m a.B.s.l. with exception of two cases; For the case (21.) and (22.) an open downstream boundary condition was used.

Results of calculations

For our cases the primary basis for comparison is the maximum velocity leaving the downstream stilling basin concrete covering reaching the rip-rap ($v_{max,gravel}$) and leaving the rip-rap reaching the soil ($v_{max,soil}$) respectively.

The results can be analyzed, evaluated in many different ways. We would like to point out just a few for the design important aspects. After consulting with the design engineers we set that only cases worthwhile to discuss where:

$$v_{max,gravel} \leq 3.5 \text{ m/s} \quad \text{and} \quad v_{max,soil} \leq 2.5 \text{ m/s}.$$

There are only few cases conforming both the above two condition (11.), (12.), (20.), and (22.) but we only briefly discuss two here.

In the case **(**20) the upstream boundary condition on the River Fehér-Körös (at 2+690 rkm) was 280 m^3/s and the downstream (at 2+450 rkm) a no flow boundary. (The total flow coming on the river Fehér-Körös goes to the reservoir between the pillars, and no water flows further in the river.) We analyzed the scenario of modelling the phenomena instead of a side weir as a front weir (weir axis is perpendicular to the main river flow) as appearing in the physical models (*VITUKI 2007/a, 2007/b*) or as a better approximation as a side weir with all water flowing through the openings (*Vízmérleg-VKKI 2010*). We compared the results of cases (14.) and (20.) to show the impact. The major difference between the two cases was only in the flow rate from the river Fehér-Körös and the flow passing through the inlet structure. We indicated the depth averaged velocity distribution between the pillars for cases (14.) and (20.) on Figure 3. In case of on flowing discharge, at the left hand side opening (upstream opening at Fehér-Körös) the velocity distribution is more irregular than when there is no on flowing water. We found similar but smaller difference in the velocity distribution at the right hand side opening. We can assume that, if the flow was not coming from the side but straight to the inlet structure, the difference between the velocities would be even bigger (in case of straight flow to the inlet, the velocity distribution would be more regular). Because of the error in the rough approximation falls on the safe side therefore our opinion is that assuming a straight flow or from the side without actual on flowing water in the modelling (either physical or numerical) would result in smaller maximum velocities than in the reality.

Suggested case for construction is the (22.) where we have used open outflow boundary condition. The open outflow boundary defines the water surface elevation on the bases of kinematical wave theory. The applied equation assumes that wave progression only from upstream determined. Of course this is only an approximation of the real phenomena.

In the following figures we present the most important hydraulic parameters resulted from our calculation.

Figure 4. shows the velocity distribution for the area under and between the pillars. The selected velocity interval clearly shows the area where the depth averaged velocity is larger than 2.5 m/s. It is marked on this figure where and how the rip-rap is situated. It can be seen on the bases of the calculated velocities, that there is no need for such a long rip-rap and the side lengthening are ’exaggerated’ as well.

On the Figure 5. we presented the cross sectional distribution of the depth averaged velocity reaching the rip-rap (v_gravel) and reaching the soil (v_soil). First time in this case were the side lengthening in the model included, which are 10 m wide. The velocity reaching the soil after the lengthening are indicated separately with (v_soil_length). The maximum velocity reaching the soil is 2.09 m/s in the side trails, – the rip-rap reduced the velocity with ~0.3 m/s – so the maximum value of velocity is located at the middle of the 10 m wide rip-rap lengthening. In the middle the velocity is relatively even with peak values of 2.20-2.28 m/s (only at one point reaching v_soil_{max} = 2.38 m/s). We have also presented the computed depths of the water at the beginning of the rip-rap and at the soil. At the bottom of the figure it is also shown where the rip-rap is and where the lengthening is situated.

On Figure 6. the distribution of the velocity vectors are shown.

The energy distribution of the flow can be studied through the *Froude-number*. Surprisingly the flow is not always supercritical between the pillars. Primarily at the downstream section $Fr > 1$, because of the smaller water depth. Of course, above the vertical sills, as well as downstream of the 2nd sill where no more damming occurs, is supercritical flow. (The supercritical flow is significant from the point of view of energy loss; it is well known that most of the energy loss occurs in the hydraulic jump.)

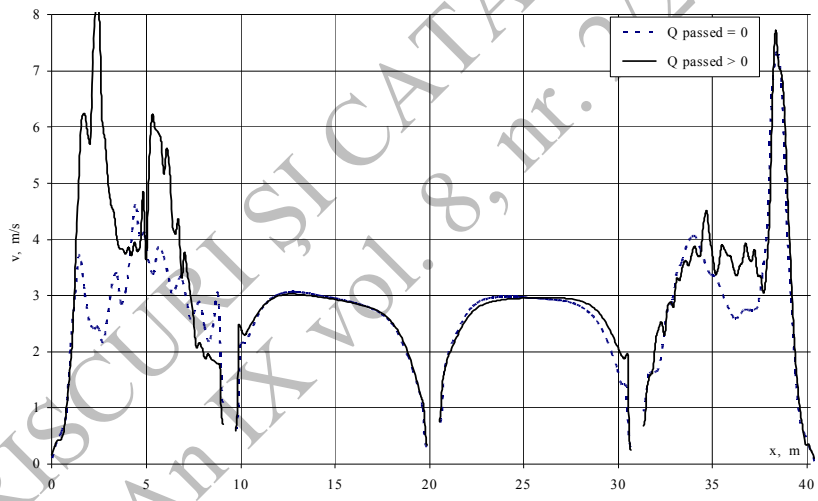

Figure 3. Depth averaged velocity distribution between the pillars in case of no on flowing discharge and with on flowing discharge.

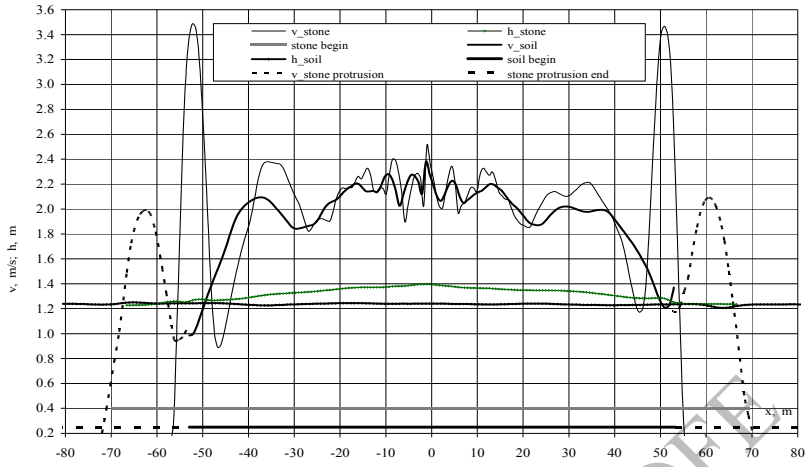

Figure 5. Cross sectional distribution of the depth averaged velocity reaching the rip-rap (**v_gravel**) and reaching the soil (**v_soil**).

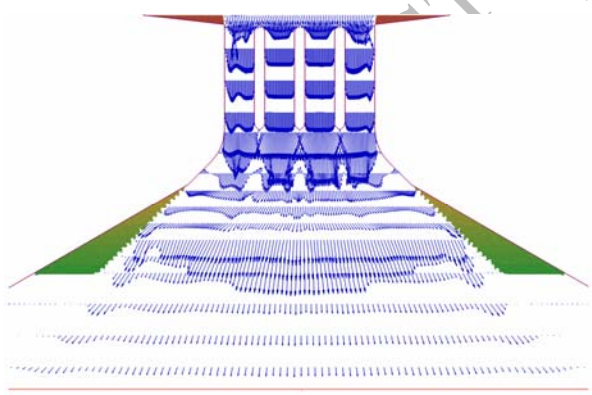

Figure 6. Spatial distribution of the depth averaged velocity vectors.

4. Suggested design on basis of hydraulic considerations

Considering the cost effectiveness, the options for the downstream stilling basin design configurations and estimated costs, the uncertainties in forecast, the time requirement for start up the operation, we suggest the following parameters for the Kisdelta flood control reservoir's inlet structure sill height and width of opening.

87.60 m a.B.s.l. and 4*8 m wide openings.

An inlet structure wit the above dimensions has a maximal discharge capacity at the filling process of

$$Q_{reservoir} = 280\ m^3/s.$$

under the agreed design conditions of the December 1995. flooding. We considered maximum emptying rate of 60-65 m^3/s under the design conditions.

In our opinion ~6-7 days is the timeframe to empty 55-60 % (14-15 $10^6 m^3$) of the completely full reservoir – as the most optimistic assumption. Better scenario should not be assumed. Further emptying the reservoir becomes more and more slow and unreliable to predict, because one can expect stagnation of ebb or a new flood wave can come. Working closely with the designers – considering the results of the numerical calculations along with the design and construction aspects, the following design is suggested for the upstream (floodplain) side and downstream (reservoir) side of the inlet structure.

Main geometrical up- and downstream dimensions for the construction suggested variant (see **Figure 1.**).

Constructions and geometrical data provided by the design engineers based on earlier investigations, and have not been changed by the calculations, are not further detailed here.

Downstream side (reservoir's side)

Concrete cover

length: 47.5 m, width: 40.2 – 118.3 m.

Rip-rap

length: 138 m, width: 10 m

lengthening on the sides : length: 22 m, width: 10 m

Downstream ground sills for energy dissipation:

1st sill located 15.0 m from the downstream end of the pillars

width: 0.60 m, length: 52.0 m,

height: 1.05 m, (elevation: 87.14 ma.B.s.l.),

Length of openings between ground sill and side wall: 3.0, 3.0 m.

2nd sill located 25.6 m from the downstream end of the pillars,

width: 0.60 m, length: 73.2 m,

height: 0.90 m, (elevation: 86.96 m a.B.s.l.),

Length of openings between ground sill and side wall 3.0, 3.0 m.

Upstream side **(**floodplain**)**

Concrete covering

length: 30 m, width: 40.2 – 90 m.

Concrete blocks for energy dissipation,

length: 100 m, width: 5.3 m,

lengthening on the sides: length: 11.8 m, width: 5 m.

Rip-rap

length: 110 m, width: 9.7 m
widening on the sides length: 17 m. width: 5 m

Ground sill at upstream side for energy dissipation (on the Fehér-Körös floodplain):

Ground sill located 14.4 m from the upstream end of the pillars,
width: 0.60 m, length: 50.8 m
height: 0.70 m, (elevation: 88.01 m a.B.s.l.),
length of the openings between the vertical sill and the side wall:3.0, 3.0 m.

Studying the above described inlet structure under design hydrological conditions the maximal depth averaged velocities reaching the rip-rap and the soil on the downstream (reservoir) side are

$$v_{max,gravel} \leq 3.5 \text{ m/s and } v_{max,soil} \leq 2.5 \text{ m/s},$$

on the upstream (floodplain) side respectively

$$v_{max,gravel} \approx 2.80 \text{ m/s and } v_{max,soil} \approx 2.30 \text{ m/s}.$$

The stipulated parameters for the modernization of the inlet structure of the Kisdelta flood control reservoir can be kept such as impact distances and water lowering. These are based on earlier calculations prepared and agreed on for the Körös river system. It was with our calculations justified under design flood conditions agreed for designing the flood control reservoir.

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