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REHABILITATION OF HYDROTECHNICAL STRUCTURES IN THE ACTUAL CONDITIONS OF CLIMATE AND ANTHROPIC CHANGES

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Abstract

In the eastern part of Romania, there are a lot of reservoirs of water whose age exceeds 30-40 years. They were built taking into account the existing norms and the specific environmental conditions at that time.
Over time, there have been numerous changes in the parameters that define how these water reservoirs are exploited, which have questioned their safety in operation. The deforestation that has taken place over the last 30 years in Romania and the climate change have led to the emergence of extreme weather and hydrological phenomena that have put hydrotechnical structures to a challenging test. Lately, securing the ability to transit the floods through medium and small hydrotechnical arrangements safely has become a stringent problem that requires finding solutions. The fact that the hydrotechnical structures are unique and the solutions adopted in a situation cannot be adopted altogether and in another work, leads to difficulties in the safety of these types of structures. The more the problem becomes worse when the initial use of the work needs to be changed.

Key words: floods, hydrotechnical structures, rehabilitation, wave trap

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1. Introduction

The impact of climate change, increasingly visible on a global scale is one of the greatest challenges of the contemporary world. In this context, the interest in analyzing the effects of climate change has increased, involving the observation, understanding and modeling of the five components composing the climate system - atmosphere, oceans, soil, cryosphere and biosphere - in an integrated manner (Davies and Simonovic, 2005; Mosoarca et al., 2017).

Changes in climate together with anthropic activities seriously impacted water resources in the world and led to the emergence of extreme weather and hydrological phenomena, which threats various hydrotechnical structures (Craciun et al., 2010; Hraniciuc et al., 2017 a, b; Kundzewicz et al., 2014; Martinez-Grana and Gago, 2018; Muzik, 2013).

The analysis of emergence and variation of floods due to intense and long-lasting rain generated by extreme weather is a key issue for the assessment and management of flood risks and protection of hydrotechnical works. Floods may induce landslides, sudden failure of inhibiting structures etc. Besides, human activity, particularly land-use changes and mining have had a major impact on the climate changing, flooding, quality of the environmental factors (Craciun et al., 2010; Greggio et al., 2018; Kundzewicz et al., 2014; Sarauskiene et al., 2015).

A critical evaluation of recent scientific literature on climate change and the impact of extreme events has been made by the Intergovernmental Panel on Climate Change (IPCC) in its *Report on Managing*

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the Risks of Extreme Events and Disasters to Advance Climate Change Adaptation. In the appendix of the IPCC Report, floods are define as: *"the overflowing of the normal confines of a stream or other body of water or the accumulation of water over areas that are not normally submerged. Floods include river (fluvial) floods, flash floods, urban floods, pluvial floods, sewer floods, coastal floods, and glacial lake outburst floods."* (IPCC, 2012; Kundzewicz et al., 2014).

The carbon cycle of the planet is to a large extent conditioned by the existence of forests. Deforestation plays a determining role in global climate change as a consequence of the accumulation of carbon dioxide in the atmosphere. It is estimated that more than 1.5 billion tons of carbon dioxide are released into the atmosphere due to deforestation, mainly forest felling and burning, every year (Longobardi et al., 2016; Sohng et al., 2017). Other effects of deforestation include soil erosion and floods, as trees have the role of retaining water and strengthening soil (Lawrence and Vandecar, 2015; Rusu et al., 2017).

As everywhere, there are currently profound changes in forests in the Romanian mountainous area, since massive lands were deforested. From a hydrological point of view, this reduces the soil capacity to retain water from rain, significantly increase water flows and speeds in rivers (Iacobescu et al., 2012; Hraniciuc et al., 2013, 2016). Also, the effects of climate change are evident. Rains have gained increased intensity, which leads to intensification of water amount fallen on a hydrographic basin (Dorondel, 2016; Mihai et al., 2017).

By overlapping the two phenomena described above, a profound change in the hydrological situation occurs in the mountain area, especially in small hydrographic basins. In rainy periods, river flows are increasing and floods become very intensive. This leads to increased pressure on existing hydrotechnical structures (Boariu, 2016; Cercel et al., 2015). They have to deal with much higher floods than those that were taken into account when designed. Hence, it is necessary to redevelop the hydrotechnical structures located in the small hydrographic basins so that they can pass through floods specific to the current period (Cercel et al., 2015; Boariu and Bofu, 2016, 2017).

In this context, the present paper presents a case study in Romania addressing the role of hydrotechnical structures in different flooding conditions. Initially, these works are thought with the role of conducting rainwater downstream of a tailings deposit resulting from mining activities.

At present, it is necessary to think a solution able to respond to the changed in use, as recreation place, so that the work should have the ability to pass the floods corresponding to the current conditions and to be in harmony with the environment.

2. Case study

2.1. Description

The Delnita hydrotechnical structure in Suceava County was originally thought to be a tailings storage facility resulting from mining activities in the Fundu Moldovei area. The reservoir was designed for decanting the tailings from the chemical treatment, a component of the mining operation Fundu Moldovei. In the initial phase, for which it was built, the Delnita hydro-technical arrangement was made of two dams with two cascade reservoirs. The first dam, the upstream one, has a height of 4.20 m and a dam crest width of 3.60 m. Both slopes are 1:2. Upstream and downstream grooves are protected by concrete slabs. The dam is fitted with a concrete drainage pipe with a diameter of 3.20 m. The length of the dam is 73 m and the height of the dam crest is 916.50 m. The pipeline continues along the length of the downstream Delnita storage tank, passes through the downstream dam and then naturally connects with the river bed (Fig. 1).

The role of the upstream dam in Delnita's arrangement was to create plugs for pluvial waters that were taken over by the concrete pipe and were discharged beyond the dam downstream without affecting the tailings deposition in the downstream storage tank. Considering that the upstream dam only served as a plug for the concrete pipe, the surface of the dam is relatively small. Thus the surface corresponding to the level of the dam crest is only 0.58 ha and the volume corresponding to the same level is 9000 m^3.

Fig. 1. View to the downstream dam with the concrete pipeline carrying the flow from Delnita

The downstream dam has a length of 99.50 m and a maximum height of 9.90 m. The dam crest width is 3.60 m. The slopes are 1:1.5, for the upstream slope and 1:2, for the downstream slope. The upstream slope is protected by concrete slabs, while the downstream slope is grassy.

By building the downstream dam there was a water storage whose surface at the level of the crow is 1.96 ha and its resulting volume is 88310 m^3. This storage capacity was to be used to decant the tailings resulted from chemical treatment at mining in the area (Fig. 2). For the drainage and evacuation of water from the tailings, a 400 mm diameter concrete pipe was laid down to the downstream dam. Currently, both reservoirs are emptied, not used as intended for the original project. Dams are in good condition.

Considering that there are no mining activities in the area, there is the problem of changing the use of hydrotechnical structures, from sterile storage to accumulating water to a place for recreation, provided the transit capacity of the current floods is ensured.

In order to achieve the proposed objective, the first scenario was the rehabilitation of both dams and the realization of two reservoirs. This has proved to be unfeasible due to the small volume of upstream reservoir. If a flood was recorded and the upstream reservoir was full at the normal exploitation level, due to the low storage capacity of the accumulation, the dam would have been discharged. This has resulted in the solution for recreational use only of downstream storage.

For the optimum safe operation of the Delnita accumulation it was proposed to carry out the following works (Fig. 3):

- an intake tower at the downstream dam, which is to be connected with the pipe entering the dam;
- the intake tower's access bridge;
- waterproofing the uphill slope protection from the downstream dam;
- making a spillway provided with a fast and wave trap channel;
- rehabilitation of the wave trap from the bottom drain and the realization of a rear apron in its continuation;
- making a drain on the right slope of the reservoir, drainage to deplete in the channel of the large water spill;
- achieving a water entry threshold in the 3.20 m pipe at the upstream dam.

Although initially the bottom drainage duct that is shaking the dam was designed to fully transpose the flood flow, it was currently planned to design a large water spillway in the right slope area. If it had been left as a single drainage drain, this would have entered pressure when a higher flood had been recorded than the design. Attempts to avoid the pressure of this pipeline from the start in order to avoid the negative effects induced by this.

The following hydrological and hydraulic calculations had as main objectives the transit of downstream flood waves under safe conditions and the avoidance of the bottom drainage duct from Delnita dam. For Delnita accumulation, the hydrological data have been updated:

- surface of the associated river basin (F) = 4.5 km^2;
- the average altitude of the river basin (Hm) = 1180 m;
- maximum flows with different probability of overtaking:

$Q_{max\ 1\%}$ = 35.0 m^3/s;
$Q_{max\ 2\%}$ = 28.0 m^3/s;
$Q_{max\ 5\%}$ =20.0 m^3/s;
$Q_{max\ 10\%}$ =14.5 m^3/s;

- multi-annual average flow (Q_{med})= 0.054 mc/s;
- monthly average, minimum annual leakage 80% = 0.010mc/s;
- flooding volumes with different probability of overtaking

$V_{1\%}$ = 504000 m^3;
$V_{2\%}$ = 408000 m^3;
$V_{5\%}$ = 290.000 m^3;
$V_{10\%}$ = 211.000 m^3;

- flood duration:

t_{cr} = 3 h;
t_{tot} = 15 h;

- coefficient of form of flood, g = 0.27

Hydrographs of floods with different probability of overtaking are presented in Table 1 and Fig. 4.

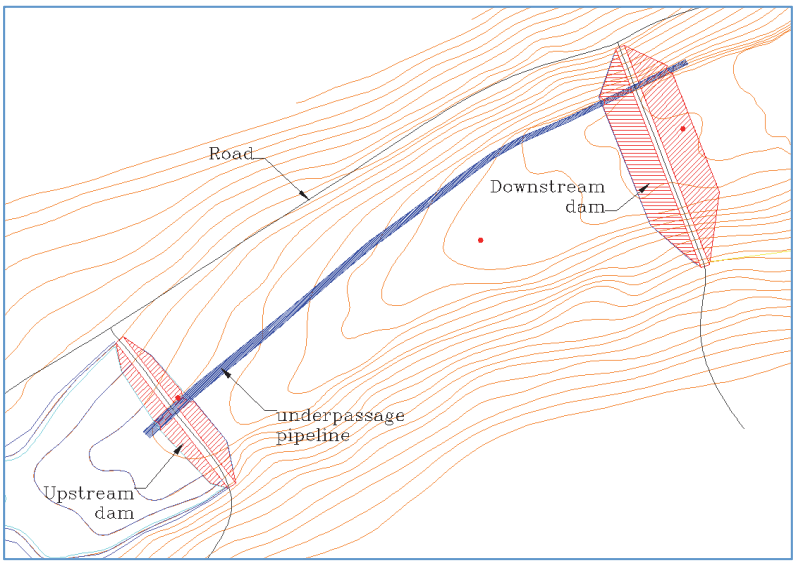

Fig. 2. Current situation of Delnita reservoirs

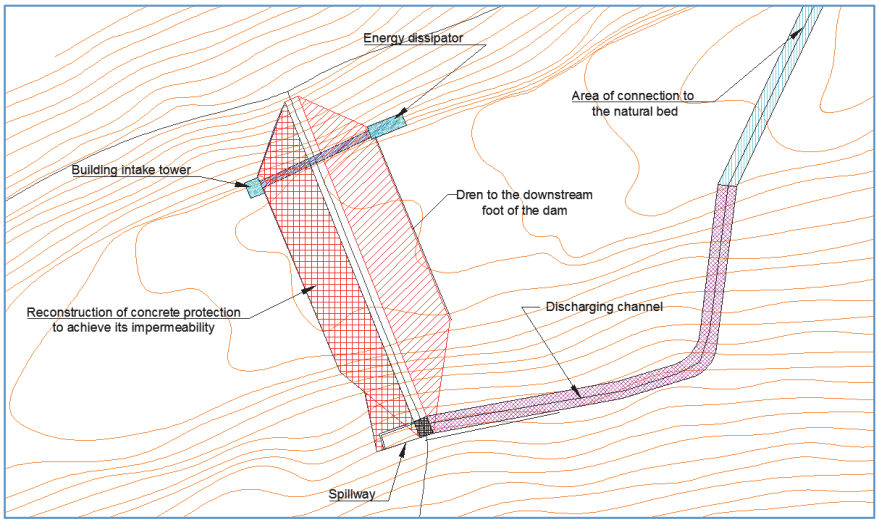

Fig. 3. Proposed works

Table 1. Water flows with different probability of overtaking

p% / *t[h]*	*0*	*0.5*	*1*	*1.5*	*2*	*2.5*	*3*	*3.5*	*4*	*5*	*6*	*8*	*10*	*12*
1	0.00	1.44	3.29	6.30	13.30	26.20	35.00	31.50	26.60	19.20	12.20	5.95	3.15	1.44
2	0.00	1.15	2.63	5.04	10.60	21.00	28.00	25.20	21.30	15.40	9.80	4.76	2.95	1.15
5	0.00	0.82	1.88	3.60	7.60	15.00	20.00	18.00	15.20	11.00	7.00	3.40	1.80	0.82
10	0.00	0.59	1.36	2.61	5.51	10.90	14.50	13.00	11.00	7.98	5.08	2.46	1.30	0.59

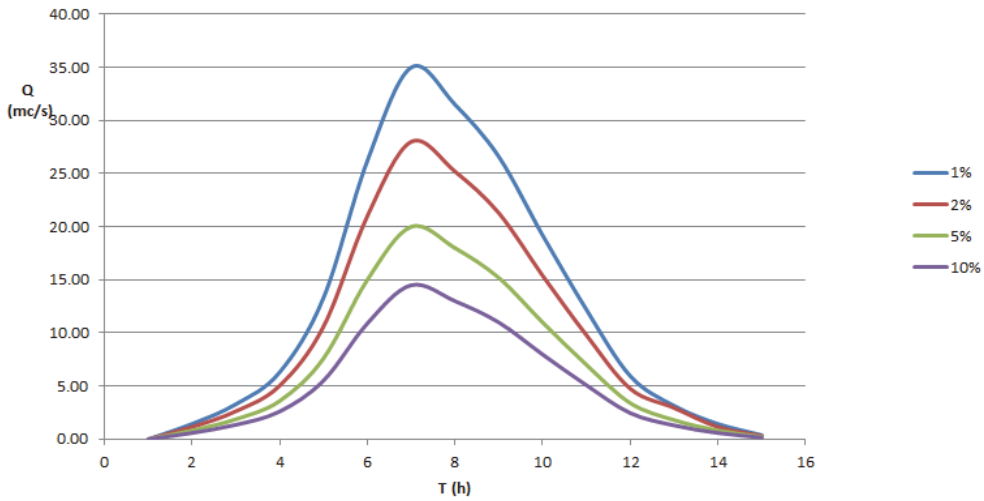

Fig. 4. Upgraded flood hydrographs

2.2. Calculation of flood transit capacity

In order to have a more coherent description of the works, considering that most of the works to be done are at the downstream dam, in the description that follows the dam Delnita will be the downstream dam.

2.2.1. Intake tower at Delnita dam

Intake tower will serve to provide servicing flows for downstream utilities and will also be able to evacuate some of the flood flows. In the horizontal section it will have the dimensions of 4.00 x 5.60 m. The intake tower height will be 9.90 m and the wall thickness of 40 cm. The tower will be equipped with the access window of the water in the intake tower, having the dimensions of 2.00 x 1.50 m. The elevation of the upper part of intake tower will coincide with the respective height of the dam crest 915.10 mdMN. At the bottom, the intake tower will have a share of 905.20 mdMN. The intake tower will connect with a diameter of Dn 3200 mm and this will act as a bottom

drain. The intake tower's foundation will have the same section as this (4.00 x 5.60 m) and the depth will be 2.50m from the bottom of the monk. The excavation share of the foundation will be 902.70mMN (Fig. 5).

For evacuation of water in the floods, the intake tower will be disposed at the top with overflow windows. The share of their thresholds will be 913.10 mdMN. The windows will be arranged throughout the monument's perimeter, less down the wall and will have a total length of 9.62 m. The openings will be 2 x 2.33 on the sides and 2 x 2.15 on the intake tower's upwards. To prevent the flood entrance, these windows will be equipped with metal grills. Access to the intake tower will be through two access holes arranged in contact with the side walls. The egg access holes will be covered with metal caps. Perimeter, the monk will be placed at the top with his hand.

2.2.2. Making a spillway provided with a connection channel, a quick offtake channel, a wave trap, and a canal connection

In order to prevent the bottom drain from entering under pressure in the case of an important flood, it was intended that the evacuated flow from the reservoir of Delnita be divided into two dischargers: drainage and spillway. The spillway is to be done in the right shoulder of the dam. The concrete wall of the spillway will be positioned at the same height as the window sill of the intake tower (913.10 mdMN). The length of the spillway will be 12.50m. In fact, the overflow threshold will consist of building a concrete basin with a length of 10m and a width of 2.50 m is inserted into the slope. After crossing the overflow threshold, the water will drop to 0.60 m and enter an exhaust duct. It will have a rectangular section to the exit of the dam and then a trapezoidal section. Its length will be 88.50 m and the trail will be along the slope.

This longer route has been chosen because the recipient is intent on making a pasture in the area immediately downstream of the dam, which will not be affected by the passage through the canal. In the dam crest area, to facilitate access from one shore to another, the channel will be made in two C2 type boxes. The width of the channel will be over its entire length of 2.50m. The slope of the slopes downstream of the dam will be 1:0.75. The channel will be concreted up to 10m downstream of the dam and downstream it will be protected with a layer of shotcrete (Fig. 6).

The offtake channel will be arranged in the continuation of the connection channel, it will have the rectangular section, the width at the base of 3.0 m and the height of 1.50 m. The length of this channel is 49 m. The channel will be concreted with concrete type XF1 C25 / 30. To reduce the speed, the channel will be disposed with macro roughness.

The wave trap. The width of the wave trap will be the same as that of the quick channel. The depth of the energy dispensing chamber will be 2.50 m above the ground level and 1.0m from the canal connection. The length of the wave trap will be 13.50m. When making this construction, use XF1 C25 / 30 concrete. At the contact area between the wave trap and the bed liner, a rear apron (small boulder Dn> 300 mm and G> 50 kg) will be disposed with a length of 10 m.

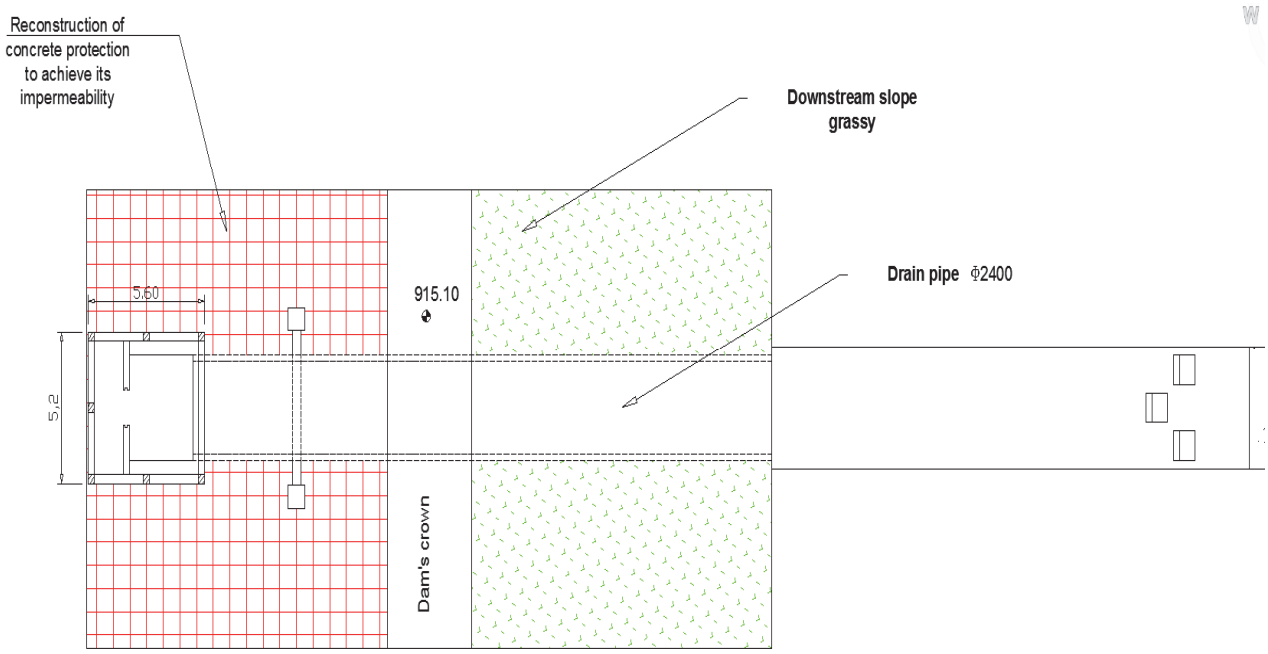

Fig. 5. Horizontal section with the works provided at the dam intake tower

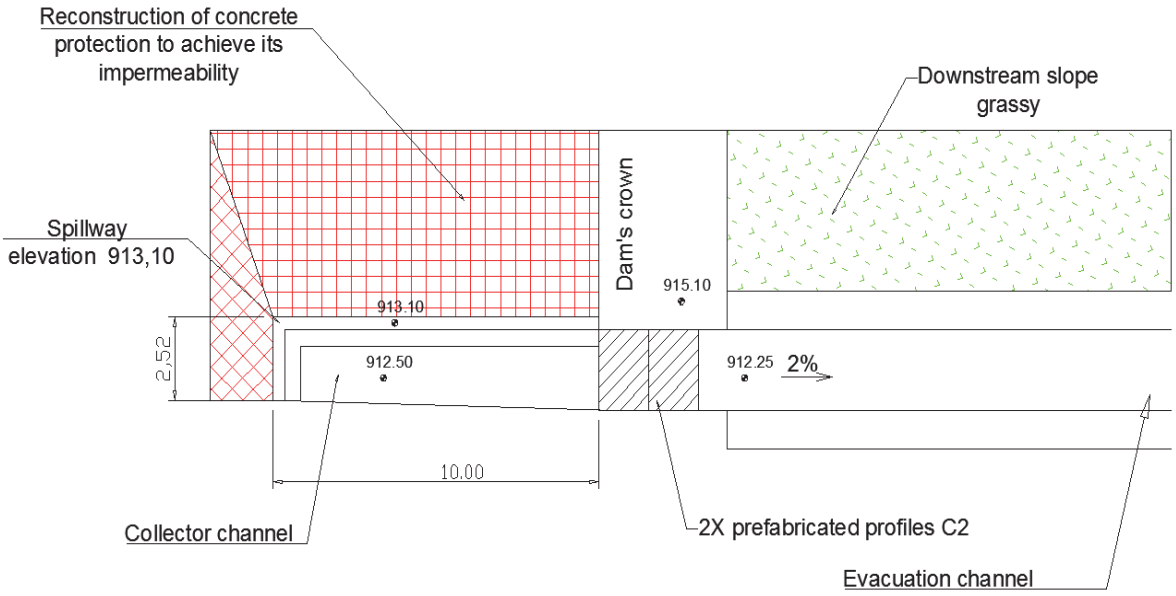

Fig. 6. Horizontal section through the works proposed at the spillway

The sewerage channel will pass the water from the wave trap to the natural bed. The section of this channel will be trapezoidal with a width of 3.0m and slope slopes of 1: 1. The channel will have a length of 58.0 m. For the first 30 m, the channel will be churned and after this portion it will have the configuration of a natural bed.

The wave trap from bottom drain will be cleaned first of all by earth deposits and increased wind. After removing the vegetation layer, the walls and foundation raft will be stamped with STNB Φ8 mesh type. The mesh will be fixed to the walls and foundation raft through the anchors. A layer of concrete XF3 c25 / 30 with a thickness of 10 cm will be available over the reinforcement. At the bottom end of the bottom drain, a concrete threshold will be made over its entire width. The threshold will be anchored in the foundation rock of 1m and its upper part will be 80 cm above the bottom clearance. The role of this threshold is to dissipate water energy. For the passage of small and medium waters discharged through bottom drainage, three metallic pipes with diameter Φ350 mm will be provided. After the bottom end of the bottom drain, a stone rear apron with a weight G> 50kg will be disposed on a length of 12 m.

For the assessment of the flood transit capacity, an assessment of the volumes of water that can be stored in the Delnita reservoir was made. These are summarized in Table 2.

In order to determine the discharge capacities of the spillway and the bottom drain intake tower assembly, attenuating made calculations for different spans of the spillway front and different drops of the spill threshold were made. The following are the final results of the mitigation calculation for the following evacuation technical data:

- opening the spillway from the right slope area is 12.50 m;
- the total opening of the discharge to the intake tower's windows is 10.40m (4 x1.90 at the side windows and 2 x 1.40 at the front window);
- the elevation for the spillway and for windows to the intake tower coincides and is of 913,10mdMN.
- the normal water retention level in the lake is 912.90 mdMN.

Table 2. Variation of surfaces and volumes in the reservoir of Delnita depending on altitude

Elevation (mdMN)	*Surface (m^2)*	*Volume (m^3)*
905.2	0	0
906	363	97
907	2200	1248
908	4580	4563
909	6230	9941
910	7985	17023
911	10198	26083
912	12945	37616
913	15438	51775
914	18358	68635
915	22403	88962
915.1	22900	91224

Calculation of attenuated volume and water level in the full lake assumption at Normal Retention Level (NNR) at 1% probability of exceedance:

- the maximum depth of water in the lake (914.07 mdMN)
- the required mitigation capacity, 12890 m^3;

- volume corresponding to the discharge height Whd = 50159 m^3;
- volume of discharge attenuation Wat_dev = 15979 m^3;
- maximum spillway flow, Qdmax = 34.74 m^3/s;

Figs. 7 and 8 show the variation in water level in the lake, the natural water hydrograph and attenuated hydrograph with 1% probability.

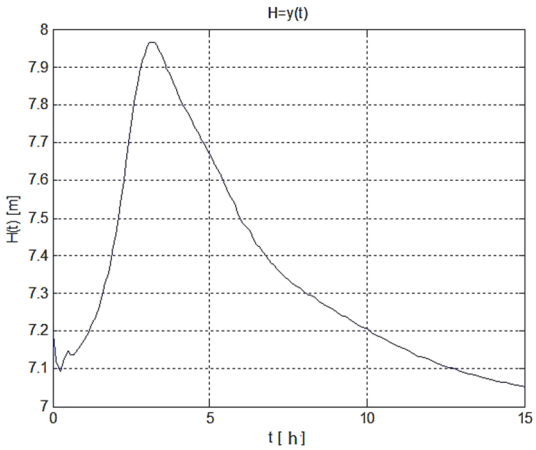

Fig. 7. Variations in water level in the lake

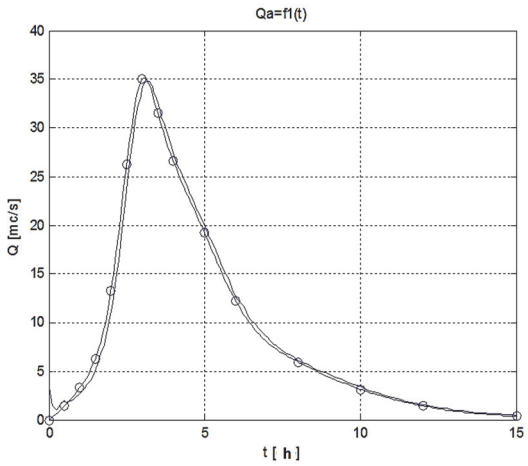

Fig. 8. Natural water hydrograph and attenuated hydrograph with 1% probability

Calculation of attenuated volume and water level in the full lake assumption at Normal Retention Level (NNR) at 5% probability of exceedance:

- the maximum depth of water in the lake (913.57 mdMN)
- the required mitigation capacity, 7606 m^3;
- volume corresponding to the discharge height Whd = 50159 m^3;
- volume of discharge attenuation, Wat_dev = 10696m^3;
- maximum spillway flow, Qdmax = 19.81m^3/s;

Figs. 9 and 10 show the variation in water level in the lake, the natural water hydrograph and attenuated hydrograph with 5% probability.

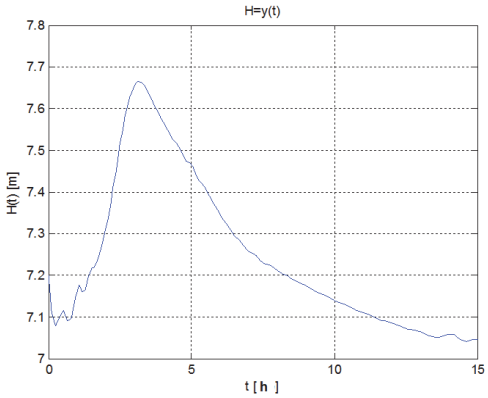

Fig. 9. Variations in water level in the lake

It is noted that Delnita reservoir has the ability to attenuate floods with probabilities of 5% and 1% assuming that the water in the reservoir lake is at the corresponding NNR share. It can be noticed that in the conditions of the proposed works in the case of a 1% probability of a flood, a guard height (between the level of water in the lake corresponding to the 1% vapor and the dam crest) of 1.24 m is ensured. From the two charts on the attenuation of the flood wave, it can be noticed that in the accumulation Delnita can make a transit of the flood wave and not a proper mitigation of the flood wave. This is due to the large difference between the reservoir storage capacity and the volumes of calculation and verification floods.

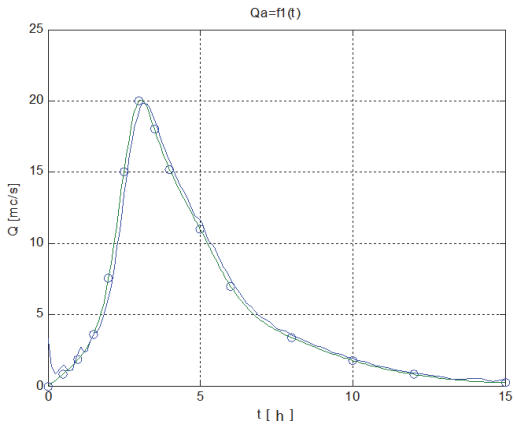

Fig. 10. Natural water hydrograph and attenuated hydrograph with 5% probability

2.3. Hydraulic calculations

2.3.1. Calculation of drain discharge capacity of the bottom drain pipe

Bottom drain discharge capacity is done in two hypotheses: it works with free-flow and underpressure flow. In the hypothesis that flow through the pipe is free, then the calculation is made using Chezy's formula (Eq. 1):

$$Q = A \cdot C \cdot \sqrt{R \cdot i} \quad (1)$$

where: Q - the flow rate; A - Area of flow section; C - Chezy's coefficient; R - hydraulic radius; i - pipe pitch.

The calculation is tabular and for different water levels in the pipe result flows capable of being discharged (Table 3). In the assumption that the bottom drainage enters under pressure the calculation of the pipe outlet capacity can be calculated with Eq. (2), where h is the load on the bottom drain. The data are provided in Table 4. Taking into account the two calculations presented above, the limnetic key of the bottom drainage pipe results (Fig. 11).

$$Q = \mu \cdot A \cdot \sqrt{2 \cdot g \cdot H} \quad (2)$$

Table 3. The hydraulic calculation of bottom drain evacuation capacity with free level

H (m)	*Elevation (mdMN)*	*P (m)*	*A (mp)*	*R*	*n*	*C*	*i*	*Q (mc/s)*	*V (m/s)*
0.00	905.2	0.00	0.00	0.00	0.018	0.00	0.02	0.00	0.00
0.20	905.40	1.41	0.18	0.13	0.018	39.42	0.02	0.36	1.99
0.40	905.60	2.09	0.50	0.24	0.018	43.77	0.02	1.51	3.03
0.60	905.80	2.53	0.89	0.35	0.018	46.68	0.02	3.48	3.92
0.80	906.00	2.97	1.33	0.45	0.018	48.59	0.02	6.12	4.60
1.00	906.20	3.38	1.80	0.53	0.018	50.02	0.02	9.29	5.16
1.20	906.40	3.79	2.28	0.60	0.018	51.04	0.02	12.77	5.60
1.40	906.60	4.19	2.77	0.66	0.018	51.85	0.02	16.52	5.96
1.60	906.80	4.60	3.24	0.70	0.018	52.40	0.02	20.15	6.22
1.80	907.00	5.04	3.69	0.73	0.018	52.74	0.02	23.55	6.38
2.00	907.20	5.52	4.09	0.74	0.018	52.85	0.02	26.31	6.43
2.20	907.40	6.11	4.42	0.72	0.018	52.64	0.02	27.98	6.33
2.40	907.60	7.53	4.52	0.60	0.018	51.03	0.02	25.27	5.59

Table 4. Hydraulic calculation of the bottom drain discharge capacity by working under pressure

Elevation (mdMN)	*A (m^2)*	*Q (m^3/s)*	*V (m/s)*
907.6	4.52	14.75	3.26
908	4.52	17.04	3.77
908.5	4.52	19.52	4.31
909	4.52	21.72	4.80
909.5	4.52	23.72	5.24
910	4.52	25.56	5.65
910.5	4.52	27.28	6.03
911	4.52	28.89	6.39
911.5	4.52	30.42	6.73
912	4.52	31.88	7.05
912.5	4.52	33.27	7.36
913	4.52	34.61	7.65
913.5	4.52	35.89	7.94
914	4.52	37.14	8.21
914.5	4.52	38.34	8.48
915	4.52	39.51	8.74
915.1	4.52	39.73	8.79

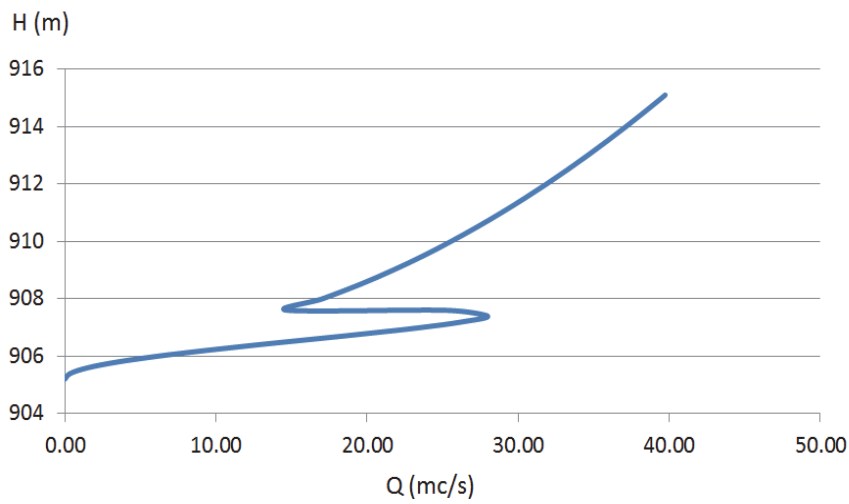

Fig. 11. Limnimetric key of bottom drainage

It can be noticed that the bottom drainage has the capacity to evacuate flows higher than the floods with a probability of 1%, but in this water it would record very high velocities (> 8m / s). To avoid recording such speeds in calculating the discharge windows of the intake tower, the spillway and the attenuation of a flood, it was taken into account that the discharge out of the windows of the intake tower was less than that at which the duct would enter under pressure.

From the mitigation calculation and taking into account the dimensions of the discharge windows and the spillway, the maximum flow rate to be transited through the bottom drain intake tower assembly, in the case of a 1% probability flood, will be 15.84m^3/s.

2.3.2. Calculation of the evacuation capacity of the windows of the intake tower

The windows of the intake tower were positioned at the 913,10mdMN threshold. These windows will be 6: 4 lateral each with a width of 1.90m and 2 fronts with a width of 1.40m each. Calculation of the evacuation capacity is made using the spill formula (Eq. 3), with data from Table 5 and plotted in Fig. 12.

$$Q = m \cdot b \cdot \sqrt{2 \cdot g} \cdot H^{1.5} \quad (3)$$

where: *m* - flow coefficient taking into account the type of the spillway; *b* - the discharge front width; *g* - gravitational acceleration; *H* - the water load on the spillway.

2.3.3. Calculation of the spillway discharge capacity

The threshold of the spillway was set at 913.10 mdMN and the displacement front width would be 12.50 m. Calculation of the evacuation capacity is made using the spillway formula (Eq. 4), with data from Table 6 and plotted in Fig. 13.

$$Q = m \cdot b \cdot \sqrt{2 \cdot g} \cdot H^{1.5} \quad (4)$$

where: *m* - flow coefficient taking into account the type of the spillway; *b* - the discharge front width; *g* - gravitational acceleration; *H* - the water load on the spillway.

2.3.4. Calculation of the exhaust channel transit capacity in the area of the two C2 profiles

For a C2 profile having the dimensions h = 2.75.m, L = 2.40 m and l = 1.60 m, the light (flow section) is assumed to be hc = 1.80 and Lc = 2.00 m. C2 profiles will also be in one access door from one shore to the other. Under these conditions, the top profile will be positioned 20 cm below the dam crest, so that a 20 cm concrete slab can be poured over the profile. This results in a 912.15mdMN profile foundation. The 40 cm difference between the channel radius at the spillway and the profiles is to be concreted.

Two C2 profiles in the flow section will be used. In the calculation of the flow able to be transited was also kept the load-induced loads caused by the walls of the profiles. It was used in flow rate coefficient ζ=0.85. Calculation is done using Chezy's formula (Eq. 1).

Table 5. Hydraulic calculation of the evacuation capacity of the spilling windows of the intake tower

Elevation (mdMN)	*H (m)*	*M (-)*	*B (m)*	*Q (m^3/s)*
913.1	0	0.36	10.4	0.00
913.35	0.25	0.36	10.4	2.07
913.6	0.5	0.36	10.4	5.86
913.85	0.75	0.36	10.4	10.77
914.07	**0.97**	**0.36**	**10.4**	**15.84**
914.35	1.25	0.36	10.4	23.17
914.6	1.5	0.36	10.4	30.46
914.85	1.75	0.36	10.4	38.39
915.1	2	0.36	10.4	46.90

Table 6. Hydraulic calculation of the discharge capacity of the spillway

Elevation (mdMN)	*H (m)*	*M (-)*	*B (m)*	*Q (m^3/s)*
913.1	0	0.36	12.5	0.00
913.35	0.25	0.36	12.5	2.49
913.6	0.5	0.36	12.5	7.05
913.85	0.75	0.36	12.5	12.95
914.07	**0.97**	**0.36**	**12.5**	**19.04**
914.35	1.25	0.36	12.5	27.85
914.6	1.5	0.36	12.5	36.61
914.85	1.75	0.36	12.5	46.14
915.1	2	0.36	12.5	56.37

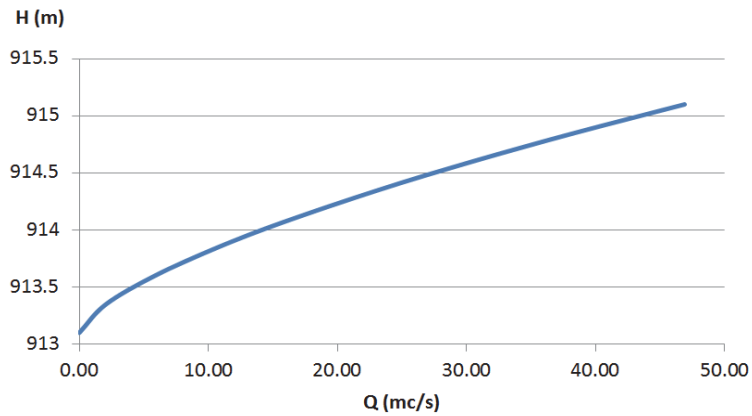

Fig. 12. The limnimetric key of the intake tower discharge windows

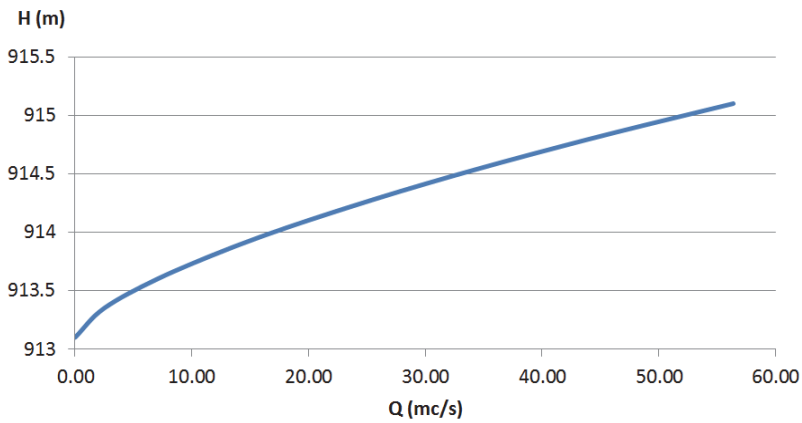

Fig. 13. The limnimetric key of the spillway

The calculation is presented in Table 7, for different water levels in the pipe result flows capable of being discharged. From Table 7 it can be noticed that the flow discharged on the spillway in the event of a 1% probability of flooding (19.04 m^3/s) can be evacuated without channel problems in the area of the C2 profiles without difficulty.

2.3.5. Hydraulic calculation of the exhaust duct downstream of Delnita dam

The evacuation channel downstream of Delnita Dam (starting at 15m downstream of the dam crest) will have a trapezoidal section with a width of 2.50 m, a depth of 1.50 m and slope slopes of 1: 0.75. Calculation is done using Chezy's formula (Table 8).

2.3.6. Hydraulic calculation of the rapid channel and the wave trap from the spillway

In order to dissipate the water energy after the exit from the rapid channel, it was proposed to create an wave trap basin. By doing so, a close jump is planned. To do this, a pool with depth d and length l will be executed at the downstream foot of the dam. Calculation of the depth d and the length l is done through the calculation steps:

a) Calculate the contracted depth (Eq. 6):

$$h_c = \frac{q}{\varphi \cdot \sqrt{2g \cdot (H + P - h_c)}} \tag{6}$$

b) Calculate h_c"
c) In order to achieve a slightly drooping jump, determine the depth of the water in the basin:
$h = 1.10\ h_c$"
d) The length of the basin is calculated with the relation:
$L_b = 5\ h_c$"

Replacing with the project data will result in the Eq. (7):

$$h_c = \frac{q}{\varphi \cdot \sqrt{2g \cdot (H + P - h_c)}} = \frac{6.46}{1.1 \cdot \sqrt{2 \cdot 9.81 \cdot (1 + 7.92 - h_c)}} \tag{7}$$

By replacing the Eq. (8), the solution is: h_c = 0.46 m, and the othe parameters take the values given in the Eqs. (9-10).

$$h_c = \frac{1.325}{\sqrt{(8.92 - h_c)}} \tag{8}$$

Table 7. Hydraulic calculation of water outlet through the two C2 profiles

Cota mdMN	*H (m)*	*P (m)*	*A (mp)*	*R*	*n*	*C*	*i*	*$Q_{cap.}$ 1 profil (m^3/s)*	*$Q_{cap.}$ 2 profil (m^3/s)*	*V (m/s)*
912.5	0	0	0	0.00	0.016	0.00	0.025	0.00	0.00	0.00
912.6	0.1	2.2	0.2	0.09	0.016	41.91	0.025	0.34	0.68	1.70
912.8	0.3	2.6	0.6	0.23	0.016	48.95	0.025	1.90	3.79	3.16
913	0.5	3	1	0.33	0.016	52.04	0.025	4.04	8.08	4.04
913.2	0.7	3.4	1.4	0.41	0.016	53.91	0.025	6.51	13.02	4.65
913.4	0.9	3.8	1.8	0.47	0.016	55.18	0.025	9.19	18.38	5.10
913.5	**1**	**4**	**2**	**0.50**	**0.016**	**55.68**	**0.025**	**10.58**	**21.17**	**5.29**
913.6	1.1	4.2	2.2	0.52	0.016	56.11	0.025	12.01	24.02	5.46
913.8	1.3	4.6	2.6	0.57	0.016	56.83	0.025	14.93	29.86	5.74
914	1.5	5	3	0.60	0.016	57.40	0.025	17.93	35.85	5.98
914.2	1.7	5.4	3.4	0.63	0.016	57.86	0.025	20.98	41.96	6.17

Table 8. Hydraulic calculation of evacuation channel capacity

h	*p*	*a*	*r*	*n*	*C*	*i*	*q*	*v*
0.00	0.00	0.00	0.00	0.02	0.00	0.02	0.00	0.00
0.10	2.75	0.25	0.09	0.02	37.25	0.02	0.40	1.59
0.30	3.25	0.81	0.25	0.02	44.07	0.02	2.52	3.11
0.50	3.76	1.44	0.38	0.02	47.34	0.02	5.97	4.14
0.70	4.26	2.12	0.50	0.02	49.46	0.02	10.46	4.93
0.90	4.76	2.87	0.60	0.02	51.06	0.02	16.09	5.61
1.00	**5.02**	**3.26**	**0.65**	**0.02**	**51.70**	**0.02**	**19.21**	**5.89**
1.10	5.27	3.67	0.70	0.02	52.30	0.02	22.65	6.17
1.30	5.77	4.54	0.79	0.02	53.38	0.02	30.40	6.70
1.50	6.28	5.47	0.87	0.02	54.29	0.02	39.20	7.17

$$h_c'' = \frac{h_c}{2} \cdot \left(\sqrt{1 + \frac{8 \cdot q^2}{g \cdot h_c^2}} - 1 \right) = 2.69 \quad (9)$$

$$L_b = 5 \cdot h_c'' = 5 \cdot 2.69 = 13.45 \text{ m} \quad (10)$$

As downstream of the wave trap, the height of the water will be 1.00 m, resulting in the depth of the wave trap being 1.50m.

3. Conclusions

Climate and anthropogenic changes currently occurring lead to the occurrence of risk situations on hydrotechnical structures. Especially for those located in small river basins, floods become torrential must be redesigned so as to face these challenges. Following the proposed rehabilitation measures, permanent storage for recreation can be achieved and can be exploited safely. The paper evidenced the possibility to change the destination of a hydrotechnical work by reconsidering its design and arrangements.

This is the case of the The Delnita hydrotechnical structure in Suceava County was originally thought to be a tailings storage facility resulting from mining activities in the Fundu Moldovei area.

Considering that there are no mining activities in the area, the problem of changing the use of hydrotechnical structures, from sterile storage to accumulating water to a place for recreation can be solved, provided the transit capacity of the current floods is ensured.

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