Highwaters Overspilling by an Energy Dissipator Top Discharger with Increasing Macrorugosities

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Abstract: *The paperwork presents a discrete 1D numerical modeling of the local hydraulic phenomenon that takes place when water flows over a macrorugosity top discharger (steps spillway), corresponding to the transit of an accidental flood of known synthetic configuration. It describes the one-dimensional simulation procedure developed with the help of HEC-RAS v.6.0, software that can engage a new approach of numerical solving in 1D, namely based on the finite volume method. Along with the proposal to apply and go through a distinct computerized hydraulic modeling, its practical purpose is to establish the parameters of transient (unsteady) hydraulics on a concrete dam top discharger, provided with macrorugosities at the downstream face. At the same time, the work proposes thus a way of checking the accidental high waters discharge as complying with legal norms in force.*

Keywords: *Stepped spillway, hyghwater overflow, hydraulics, numerical modelling.*

1. General considerations

The performed discrete numerical modeling takes the oportunity of a highwaters defending hydrotechnical arrangement, which has in its composition an RCC dam – the Boqueron Dam (Figure 1) – provided with a special top discharger made with macrorugosities (steps) at the downstream face, and with an energy dissipator with basin and threshold, respectively [1].
The Boqueron Dam, commissioned in 1999 for Confederacion Hidrografica del Segura [2], the Albacete Province (southeast of Spain), has a maximum height of 56m. Its overflowing spillway of Creager type has one free 16m gap, designed for a 1.5m regular water blade height above the climax (the maximum water blade height rises to 4m, as corresponding to the verification situation). The top discharger goes on with a canal following the dam downstream face of 0.73:1 general slope, accomplished in steps of 1.2m height, the overflowing reaching down an energy dissipator with a 35m length and 9.12m depth basin and a 4.30m height threshold, respectively.

Fig. 1. Upstream and downstream views of the RCC Boqueron Dam [2]

Figure 2, as a hydraulic structure cross-section, brings the graphical representation of the analyzed flowing route, meaning a longitudinal cross-view through the concrete dam and the energy dissipation basin, respectively a detail for achieving the variable macrorugosity in the spillway upper area.
The discrete 1D numerical modeling developed with the help of HEC-RAS v.6.0 [3] comprises two solutions for the numerical approach: the finite differences method (as a default choice) and the finite volume method (a new approach). As for the considered model with steep falls in steps (as on the area of the downstream face provided with macrorugosities), the default method of finite differences leads to numerical instability and therefore, for this case, a numerical solution cannot be reached.

When selecting the finite volume method, the numerical instability disappears (true, not immediately, but requires further geometrical adjustments), obtaining so the convergence to a numerical solution when the distance between the defined 1D cross-sections complies with some additional conditions as presented at the execution of the numerical model.

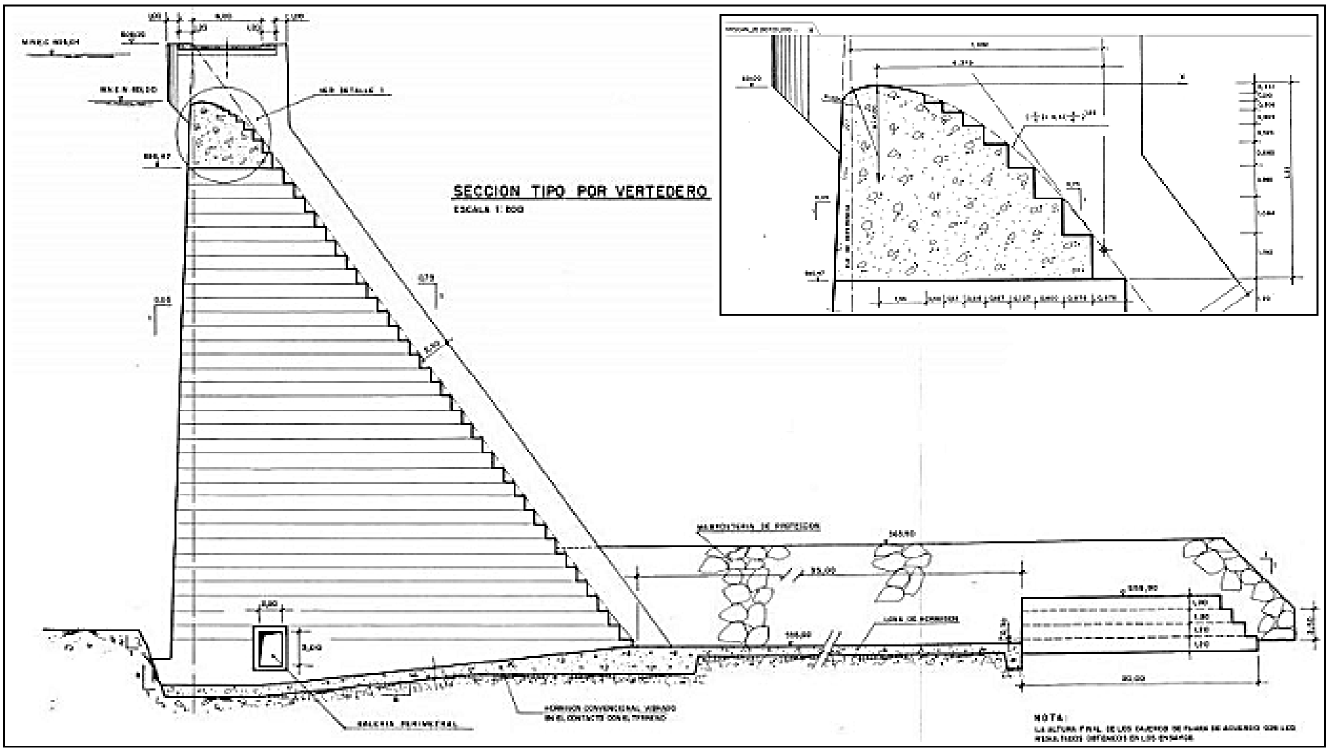

Fig. 2. Hydraulic structure cross-section: dam, macrorugosities profile top discharger, energy dissipator with basin and threshold; detail view of the variable macrorugosities accomplishment at the discharger upper part

One can see the message shown in later on Figure 5, *Notes generated when writing the computation files: Warning - XS's were found to have a reach length less than half the main channel width; **The 1D finite volume method is sensitive to the volume between XS's. Locations with very short reach lengths (small volumes) may require shorter time-steps to remain stable.
Therefore, for these reasons, if at first the developed numerical model considered the structural width corresponding to the spillway gap, a successive reduction at several values was performed. There is mentioned that there still were some cross-sections not complying with the condition, but nevertheless the numerical instability disappears.
As regarding the flow discharged by the special configurated spillway studied in the developed numerical simulation, the geometric configuration of a regular unfolding synthetic highwater was considered as seen in Figure 3 graphic representation.

2. General elements regarding the numerical model accomplishemt and the water flow simulation

The assessment of the existing situation was based on the known geometric values, materialized by a situation plan and cross-sections, specifically considered by a background image (extension file .dxf) adopted in HEC-RAS v.4.1 [4] (following the storage in a .dwg extension file). This resulted in a numeric database stored in specific files associated with the defined model, which was then loaded into HEC-RAS v.6.0. Specifically, in order to build the discrete numerical model, a linear segment was cut out of the entire flow path, geometrically representing a canal of rectangular cross-section of 16m width. The modeled segment has a length of 100.72m and has a known plane-view shape by the actual topographical values. The cross-sections of the modeled flow path segment are indicated in Figure 4 – a total of 312 sections, of which 113 are given by the given topographic values (dark color) while the other 199 sections are obtained by linear interpolation (light color and additional end notation with asterix). The elevations were graphically determined by considering the longitudinal scale profile (Autocad) and knowing the configuration of the route at the base as seen in the representation in Figure 2.

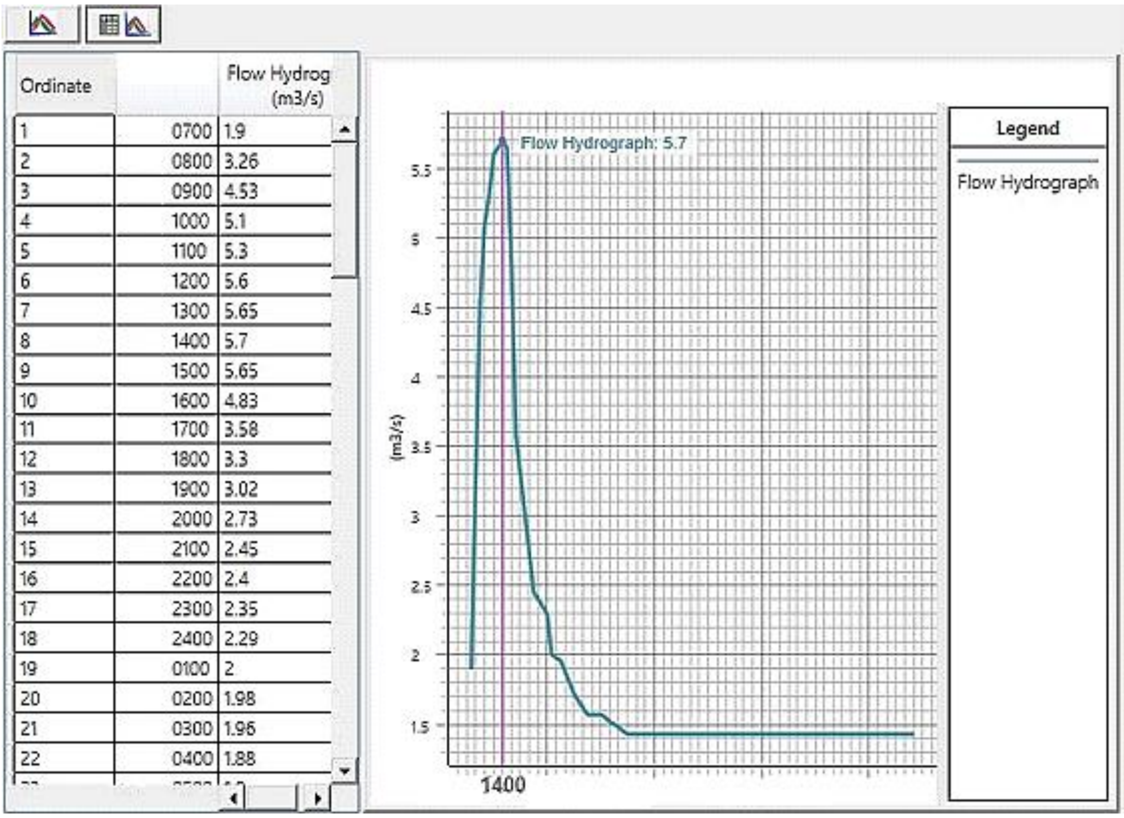

Fig. 3. Graphic representation of the employed synthetic flood hydrograph

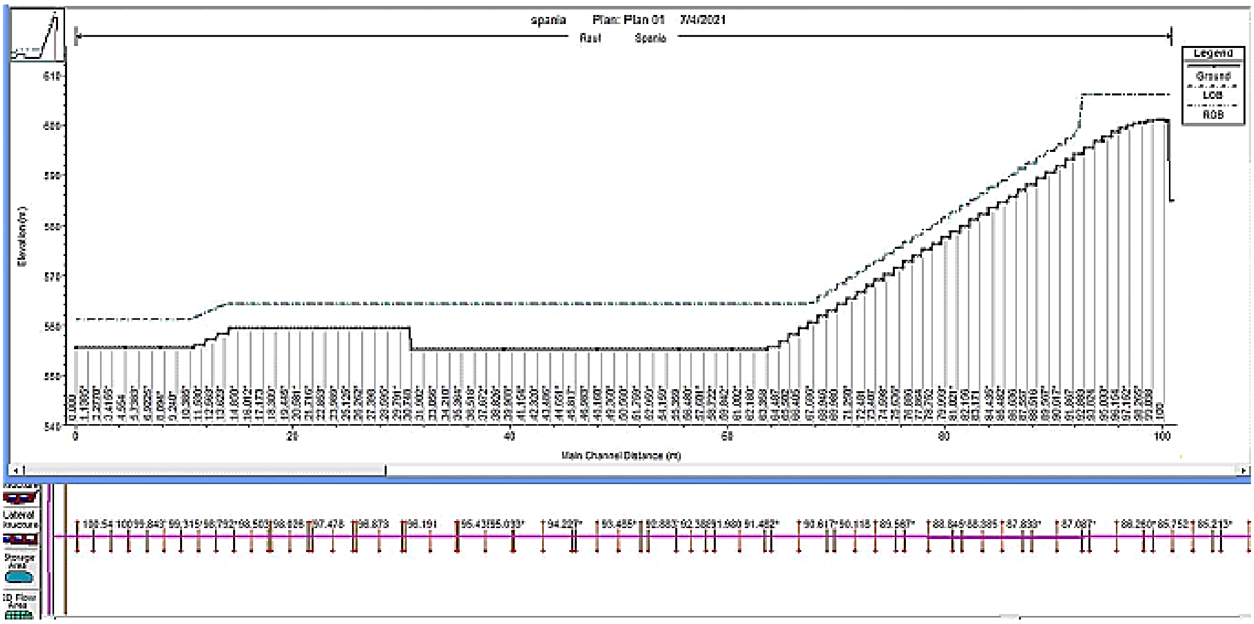

Fig. 4. Plane-view (bottom) and central longitudinal profile of discrete numerical model (1D), as corresponding to the top discharger with macrorugosities slope and energy dissipator

Further on, at the actual execution of the discrete numerical model (canal with the width corresponding to the spillway opening, B=16m) in HEC-RAS v.6.0, respectively following the employment of the finite volume method, it was found that the model is unstable. So, successive

alterations to the following canal width values were performed as required: B=1.6m, B=0.8m and finally B=0.4m. For each case, successive runs of the hydraulic model were performed in order to establish the numerical solutions (see the representation in Fig.4, the final model of B=0.4m width).
The roughness coefficient in the flow segment was assessed within the limit: n = 0.015 … 0.045, respectively, n =0.045 for the area of the downstream face provided with macrorugosities, n=0.015 for the energy dissipation basin and the downstream threshold area, and n=0.020 for the downstream connection area. The hydrodynamic gradient is known as J=0.000625 at the entrance area, and as J=0.01285 in the end zone.

3. Initial and boundary conditions

As a current approach, the boundary conditions of the modeled hydraulic path are given by: the transited discharge with a certain occurrence probability introduced as a known high-waters hydrograph, values that are assigned to the upstream area of the 1D numerical model – specifically in the section with the metric notation "100.869" – to which the hydrodynamic gradient is also associated as required for the flow rates distribution per section; the hydrodynamic gradient in the downstream zone of the model – meaning in the section with the "0.000" metric notation. In this discrete 1D numerical model, initial conditions given by entering the initial flow are required. Thus, the particular boundary conditions consider:
- a synthetic high-waters wave at which the maximum reached flow has the value Q_{max} = 5.70 m^3/s and the hydrodynamic gradient with the value J = 0.000625, as attached to the entering upstream section of "100.869" metric notation;
- the known value of the initial flow Q_{ini} = 1.90 m^3/s is considered in section "100.869", as an initial condition for starting the model run;
- the hydrodynamic gradient estimated at the value J = 0.01285, as assigned to the "0.000" metric notation section, downstream of the modeled hydraulic route.

The hydraulic phenomenon considered in the numerical modeling takes place in time for a known period, starting from 07:00 on the first day of flood wave formation and until 10:00 on its fifth day, the end of the special phenomenon. The actual execution analysis is reduced to a significant time interval for the flow, namely it was performed until 22:00 of the first day, at which moment the in-flow decreased semnificatively down to about 2.4 m^3/s. The time step of the modeling as well as the internal mapping interval were both imposed as 0.1 seconds, but the storage of the results was adopted at a time interval of 10 minutes.

4. Numerical simulation and results presentation

For a start on, the hydraulic model was run over an interval of 20 minutes, saving in a restart file the water levels obtained in all the cross sections. These values, saved in an .rst extension file, become values adopted as initial conditions for the next run considered the actual one.
Figure 5 shows some important messages along the actual hydraulic model running time interval that refer to the cross-sections that do not comply with the condition of distancing from the neighboring sections, necessary to eliminate the numerical instability. These messages mention the time moment, the 1D path, the section metric notation and the temporal elevation, as well as the value of exceeding the imposed tolerance value. This information monitors the execution of the hydraulic model in terms of satisfying some requirements regarding the confidence parameters of the results.
Following the running of the actual numerical simulation, all the constant or time dependent parameters, referring to water levels and flows, were obtained, over the entire discrete numerical model, in the situation of non-permanent water transit along the considered route. The results obtained by the common graphic post-processing operations are presented as follows:
- Longitudinal profiles visualizing the high waters hydrograph transit mode – water levels development with respect to Sea Level – at several specific time moments along the first day of the hydraulic phenomenon (Figure 6): 07:00 (entering transit flow of 1.9 m^3/s), 08:00 (3.26 m^3/s entering flow), 14:00 (5.7 m^3/s entering flow) and 22:00 (2.4 m^3/s entering flow);
- Longitudinal profiles corresponding to the maximum entering transit flow of 5.7 m^3/s - water levels (mSL) in several specific points (sections) at 14:00 on September 22nd (Figure 7);

One can notice that the flow transit mode on the modeled route takes place under different flow regimes: rapid regime at the downstream face in the macroroughness area (below the critical flow line); slow regime in the area of the energy dissipation basin; slow and fast mode on the threshold of the energy dissipation basin; slow speed in the area of connection with the natural terrain downstream of the discrete numerical model.

- Water level (mSL) and discharge (m^3/s) development for the entering ”100.869” and outgoing ”0.000” sections (Figure 8).

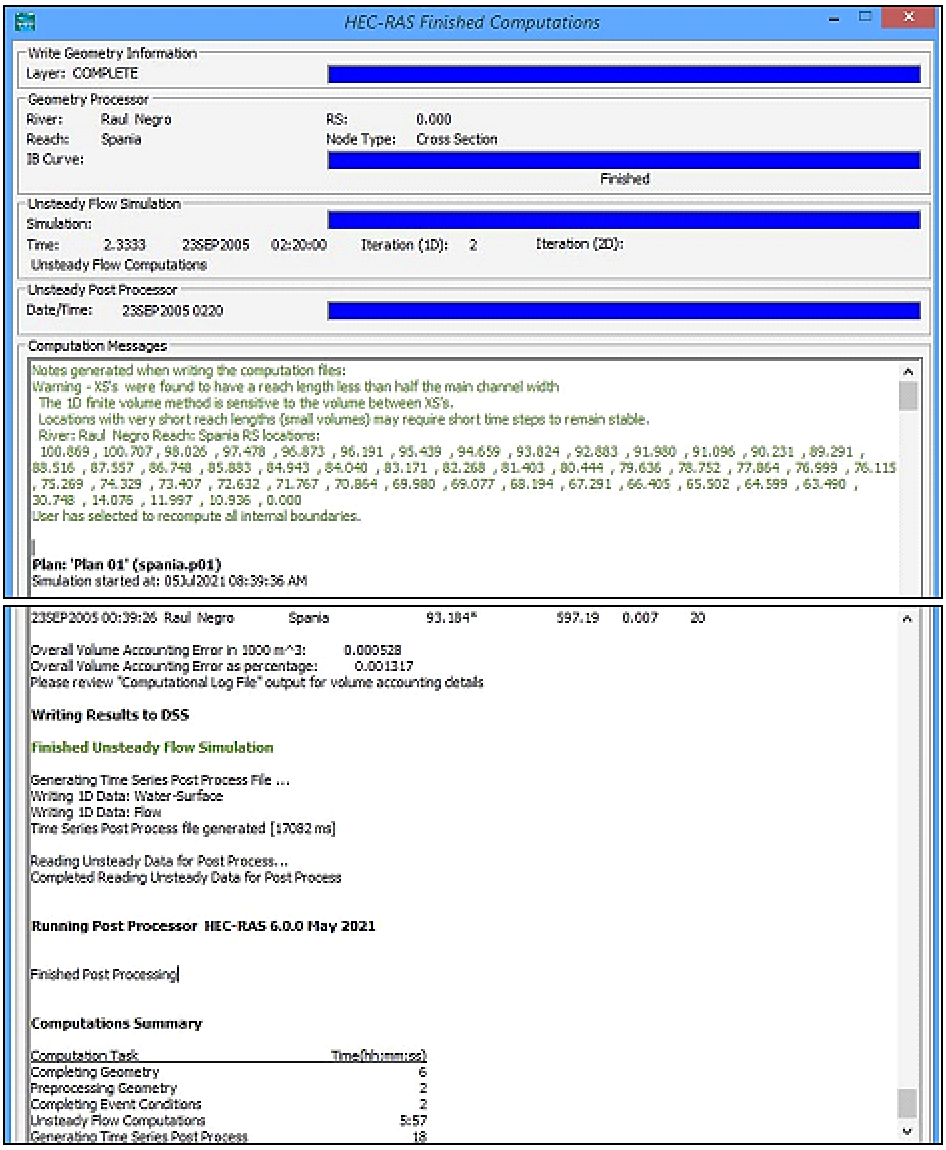

Fig. 5. Messages along the actual running of the developed model

5. Discutions and conclusions

After reviewing the results provided, it is observed that in the finite volumes method the variation of the velocities in the cross-section is no longer determined. Based on the graphical representations of the cross-sections shown in Figure 7, the average speeds in the section at the current time can be estimated (the base elevation can be considered from the profile, while the maximum level elevation is given in the cross-section). Thus:

- The section at the entrance area of notation “99.315*“(up-left in Figure 7)

$$v_{med} = \frac{Q_{max}}{B*h} = \frac{5.70}{0.4*(604.24-601.00)} = 4.398\ m/s;$$

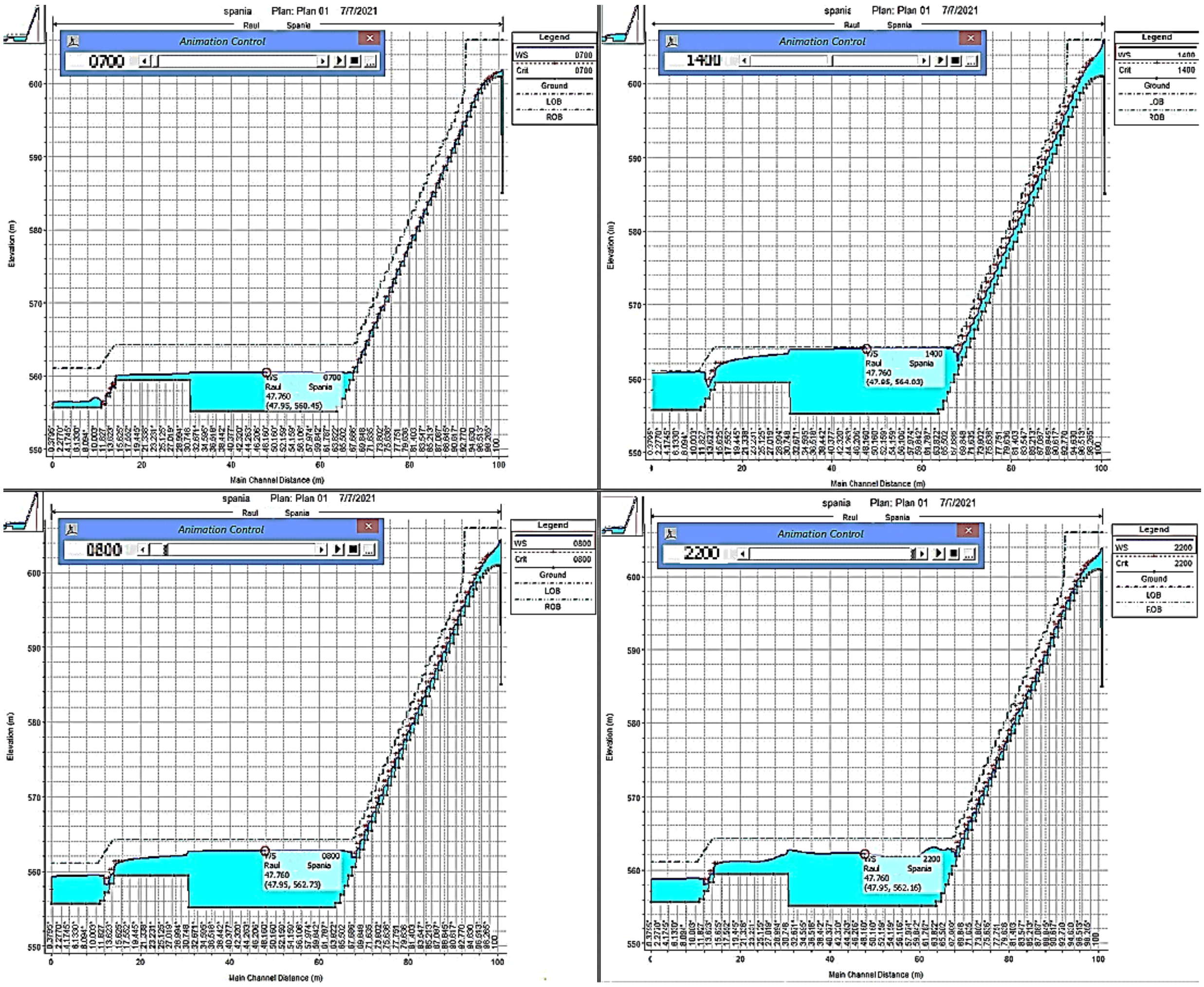

Fig. 6. Longitudinal profiles by the hydraulic model at several specific hours along the first day of the high waters transit phenomenon

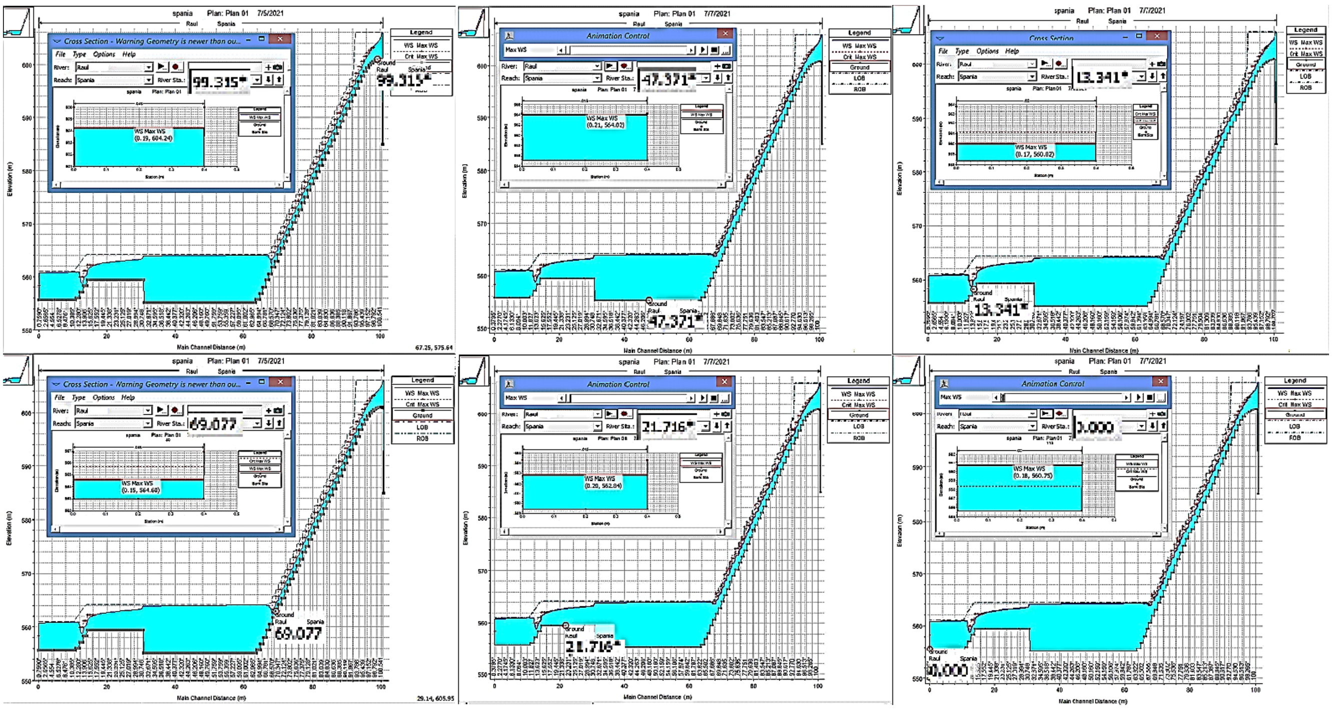

Fig. 7. Longitudinal profiles by the hydraulic model at the specific hour 14:00 of maximum entering flow detailing the water levels development in several path cross-sections

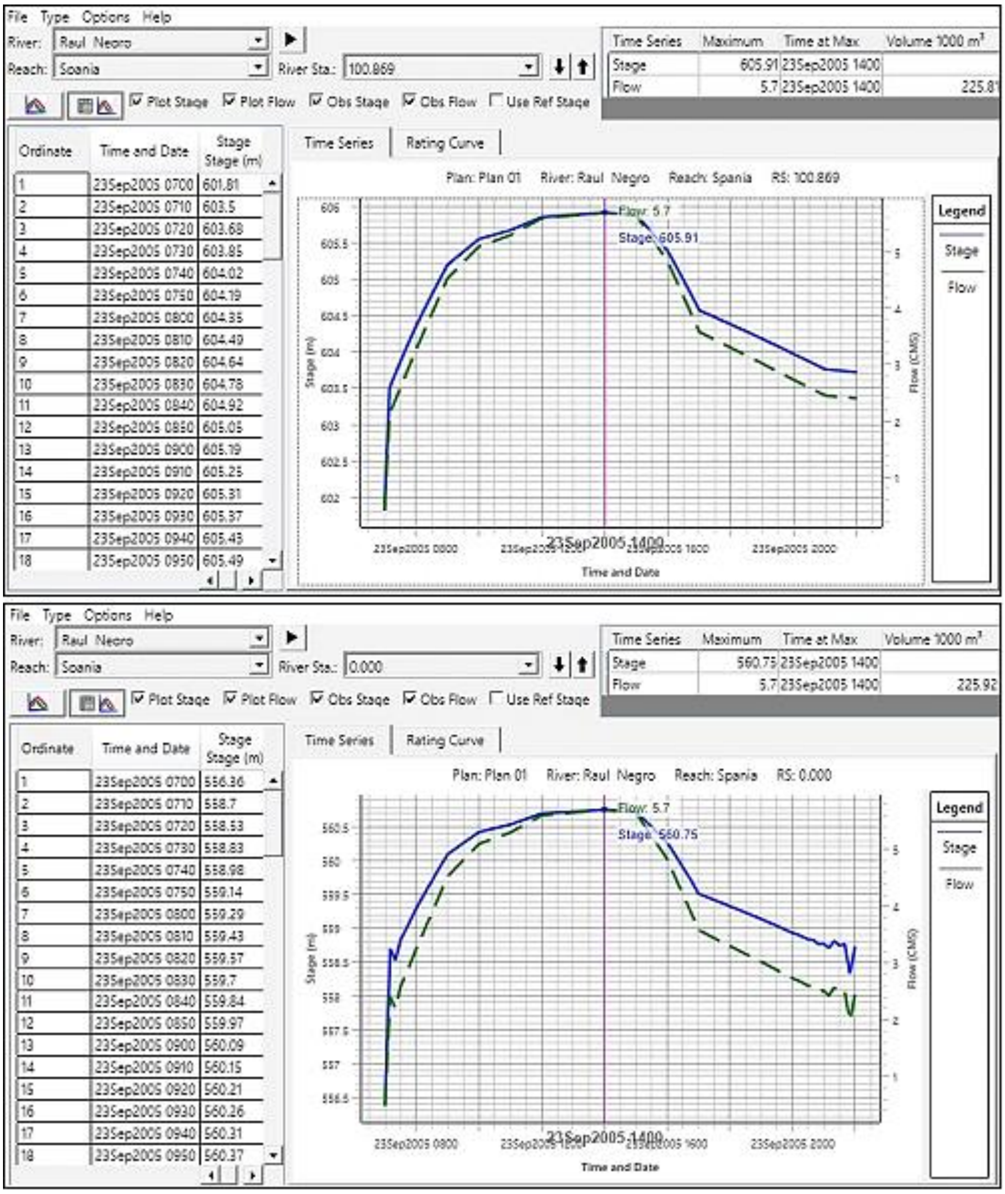

Fig. 8. Water level and discharge development for the entering (above) and outgoing sections of the model

- The section in the connection area to the energy dissipation basin, notation “69.077“(down-left in Figure 7)

$$v_{med} = \frac{Q_{max}}{B*h} = \frac{5.70}{0.4*(564.60-562.96)} = 8.689\ m/s;$$

- The energy dissipation basin area section of notation “47.371*“(middle-up in Figure 7)

$$v_{med} = \frac{Q_{max}}{B*h} = \frac{5.70}{0.4*(564.02-555.13)} = 1.603\ m/s;$$

- The section upstream of the energy dissipation threshold, notation “21.716*“(middle-down in Fig. 7)

$$v_{med} = \frac{Q_{max}}{B*h} = \frac{5.70}{0.4*(562.84-559.43)} = 4.179\ m/s;$$

- The section imediately downstream of the threshold at energy dissipater, notation “13.341*“(up-right in Figure 7)

$$v_{med} = \frac{Q_{max}}{B*h} = \frac{5.70}{0.4*(560.02-558.34)} = 8.482\ m/s;$$

- The model end section of notation “0.000“(down-right in Figure 7)

$$v_{med} = \frac{Q_{max}}{B*h} = \frac{5.70}{0.4*(560.75-555.64)} = 2.789\ m/s.$$

For a maximum head estimated as $T_0 = P + H + {v_0}^2/_{2g} = 51.234m$, by an iterative calculation (maximum 4 iterations, in this case) it is possible to determine the contracted depth in the dissipating basin (at the maximum flow rate and for the situation of a spillway considered of practical type profile $m_d = 0.49$) [5], with the help of which the average water velocity can be estimated:

$h_c = \frac{Q_{max}}{\varphi \cdot B}\sqrt{\frac{1}{2 \cdot g \cdot (T_0 - h_c)}} = \frac{5.70}{1.0 \cdot 0.4} \cdot \frac{1}{\sqrt{2 \cdot g \cdot (51.234 - h_c)}} = \cdots\ 0.452\ m$ and so $v_m = \frac{Q_{max}}{B \cdot h_c} = \frac{5.70}{0.4 \cdot 0.452} = 31.56\ m/s.$

In this situation, it can be assessed that the maximum value of the velocity reached on the analyzed route (8,689 m/s, as previously estimated for the section in the connection area to the energy dissipation basin) compared to the value determined with the approximation relation ($v_m = 31.56\ m/s$), leads to a velocity reduction coefficient on the model with macroroughness of approx. ***k_r*** = 0.275. It can be thus considered that this may represent the overall kinetic energy dissipation along the spillway downstream face due to the macroroughness system.

As to check the discharge capacity through a single spillway gap (where $B_{max} = 16m$, with respect to the width of the numerical pattern of $B = 0.4m$), i.e. the capacity of the entire spillway with two gaps, a linear scaling relationship can be considered:

$Q_{med} = \frac{B_{max}}{B} \cdot Q_{max} = \frac{16.00}{0.4} \cdot 5.70 = 228\ m^3/s\ \ \ldots\ \ Q_{dev} = 2 \cdot Q_{med} = 456\ m^3/s.$

In conclusion, the accomplishment of a discharger face with macroroughness as presented by the situation considered will lead to great kinetic energy dissipation developed at all moments of a special flood wave transit (especially obvious for the maximum flow transit).

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