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Adaptation of TELEMAC-2D to the regulation of a CNR development during floods

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Abstract — **On the Rhône River, Donzère-Mondragon is the second oldest CNR development and the most productive one with a total installed capacity of almost 2100 MW. It follows the typical CNR development scheme. It is composed of a reservoir, a barrage, a diversion channel leading to a hydropower plant and a lock. When there is no flood, water is diverted to the hydropower plant. During floods, water passes mainly through the barrage. Donzère-Mondragon specificity is its 17.3 km long headrace channel. Three specific structures were built at the diversion channel entrance to prevent water level from rising in the channel during floods and to allow navigation when there is no flood. Operating rules are established to manage water levels and hydropower plant discharges from lowest discharge to the design flood.**

A 2D-model of the whole development is built using TELEMAC system. The model is 68 km long and integrates 115 km² of flooding area. A 141 500 elements mesh has been generated and calibrated. Specific programs are implemented in TELEMAC-2D so as to represent Donzère-Mondragon development behaviour in flood period. For the hydropower plant, the regulation managing upstream water level and outflow is simulated. A specific program based on energy equation is applied for the CNR structures. The subroutine allows manual water level management. The barrage management also requires a specific implementation depending on upstream discharge. Those specific programs are gathered to perform computation during high floods in steady and unsteady mode.

Opening structures calibration is detailed. Results focus on hydropower plant and barrage discharges to check flow conservation. Reservoir and channel water levels are analysed to ensure operating rules are respected for various discharges in steady mode. In unsteady mode, program limits connected water level operating managements are identified. Computation stability problems are encountered. Part of them is fixed by adapting mesh density. Solutions found in the studies to get round of the other difficulties are mentioned. Improvements are needed to avoid instabilities due to flow calculations at a specific section.

I. INTRODUCTION

Modelling of widespread floodplains and complex structures with regulation have been carried out for a long time with mono-dimensional software. The progresses in computing performances, the densification of topographic and bathymetric data, and the capitalisation of experiences in TELEMAC-2D have made possible the bi-modelling of a complex development. The article presents the modelling of Donzère-Mondragon development.

Donzère-Mondragon is one of the eighteen developments managed by the Compagnie Nationale du Rhône (CNR). CNR holds the concession of Rhône valley from Swiss border to Mediterranean Sea. Its three main missions are electric production, navigation and irrigation. CNR engineering team develops mathematical modelling to answer operating needs, to check the concessionary obligations and for engineering as well. Modelling was mainly mono-dimensional and now very often bi-dimensional.

The bi-dimensional modelling of Donzère-Mondragon development answers the issues of dike overflows and flow propagation in Pierrelatte plain. As a first approach, the simulations are focused on flood. This article describes the methodology carried out to integrate CNR development regulation using TELEMAC-2D.

The approach is divided in four steps. Firstly, the Donzère-Mondragon development features are explained. Secondly, the TELEMAC modelling is detailed. Thirdly, the article describes programs implemented within TELEMAC-2D to integrate regulation. Finally, modelling results are analysed, limits of developed programs are listed and improvements are proposed.

II. DONZÈRE-MONDRAGON DEVELOPMENT

A. *A typical CNR development*

Donzère- Mondragon (DM) development is located in the Rhône River valley (south east of France), north of the city of Orange and south of Montelimar. It was built in 1953. The development includes Pierrelatte floodplain. Caderousse (CA) is situated downstream Donzère-Mondragon development.

Donzère-Mondragon follows the typical CNR development scheme (Fig. 1). Indeed, it is composed of:

- A hydropower plant (Usine de Bollène: USB) with a total installed capacity of around 2 100 MW. It comprises six Kaplan units with a maximum power station discharge of 1 980 m3/s. The power station units do not include sluicing operation capabilities. Thus, two surface gates and six gates were designed. Their aim is to prevent surge waves in case of full load rejection. Moreover, during big floods, around half of the diverted discharge passes through the gates.
- A barrage (BarraGe de ReTenue de Donzère: BGRT). It is composed of six gates. When upstream discharge is higher than the turbines maximal discharge, gates start opening. During big floods, the gates cannot regulate water level as they are totally opened.
- A lock for navigation purposes.
- A reservoir (retenue RE). It is 4 km long and its normal water level is 58.5 m NGF.
- A headrace channel (Canal d'amenée: CdA). It is 17.3 km long and the longest of CNR ones.
- A tailrace channel (Canal de Fuite: CF) which is 11 km long.
- The natural river course (Vieux Rhône :VR). A minimal discharge is always maintained during dry season. During floods most of the flow goes through this natural river. The famous cevenol tributary Ardèche converges into this part of the natural river course.

Figure 1. Typical low-head development scheme.

B. *Donzère-Mondragon deveplopment features*

Donzère-Mondragon uniqueness is its headrace channel which is very long. In case of flood, water levels upstream the barrage rise and this rising can be propagated in the headrace channel. Specific structures were built at the headrace entrance to minimise this effect, to reduce dykes height and to prevent solid transportation from entering the headrace channel. They are called "protecting gates".

On the right bank, the old navigable gates (Ancienne Passe Navigable: APN) were originally built for paddle boat passage. The two 45 m long gates are closed during flood. No vulnerability has been identified for flood bigger than the design one concerning this structure.

The changing characteristics of convoys and the transit difficulty through the APN required building a new gate. This new navigable gate (Nouvelle Passe Navigable: NPN) was commissioned in 1986. In case of flood; this gate is closed. In extreme conditions (discharge higher than the design flood) the NPN may break depending on upstream hydraulic head.

On the left bank, the hydropower barrage (BarraGe Usinier: BGU) was designed to limit headrace water level for high discharges.

C. *Operating rules*

So as to combine hydroelectricity optimisation, navigation and overflow prevention in headrace channel, operating rules were set. Operating rules manage the water level at a specific location (Regulating Point – Point de Réglage: PR) in the reservoir or in the headrace channel depending on the input discharge. They also assign power plant discharge vs. the total input discharge. These rules have to be observed from the lowest discharge to the design flood.

Three regulating points (Fig. 2) are used to manage the water levels of the Donzère-Mondragon development: PR1 at SNCF bridge for low discharges, PR2 upstream the barrage for middle discharges and PR3 downstream the "keeping structures" at KM 171.5 for high discharges.

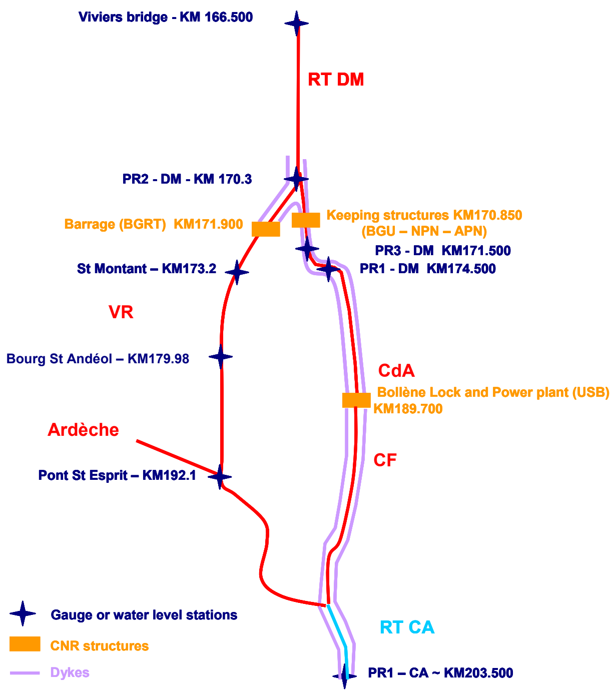

Figure 2. Location of Donzère-Mondragon structures, dykes and stations.

As the study focuses on high discharges, only rules at PR3 are considered. The water level is managed by the BGU with a 50 cm allowed variation.

III. TELEMAC MODELLING

A. Computational domain

The whole stretch of the modelling is 68 km long, integrating downstream part of the Ardèche tributary and 2 km of the Caderousse downstream reservoir. Piers of the sixteen bridges situated in the modelling footprint are represented as islands within the mesh. This modelling is considered acceptable given the study purposes. Indeed, results are not focused on local phenomenon at the immediate vicinity of bridge piers. Hard lines have been built to represent CNR and other dykes. The mesh segment size varies from 40 m close to the CNR structures, as instabilities are liable to occur, to 100 m in the low-water bed of the natural watercourse. The grid (Fig. 3) comprises more than 71,500 nodes and 141,500 elements. This mesh has been generated with Matisse. The time step is 2s.

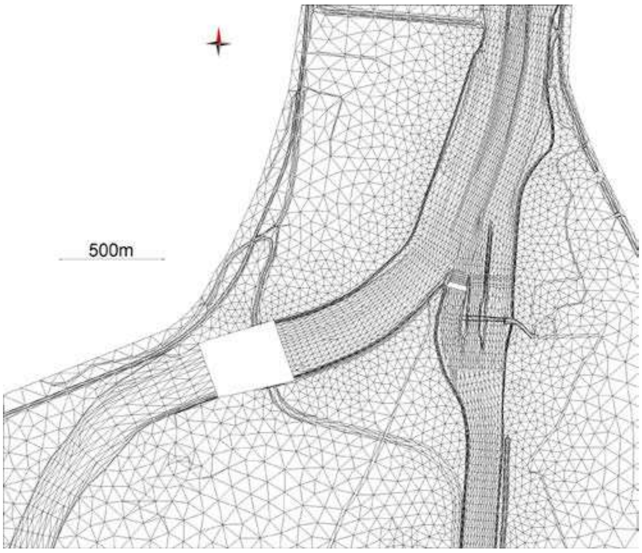

Figure 3. Bi-dimensional grid of the headrace entrance and BGRT.

B. Calibration

The modelling of turbulence is constant viscosity with an overall viscosity coefficient equal to 0.1.

Then, the calibration is focused on bottom friction coefficient, which is computed following Strickler's law [1]. A high variation in initial water levels is observed between upstream and downstream boundary conditions (more than 20 m high).This variation makes complex the model initialisation. To avoid this problem and since there is a hydraulic disconnection at BGRT and USB, it has been possible to subdivide the grid in two sub-grids. The upstream sub-model spreads from Viviers bridge to Donzère-Mondragon barrage and hydropower plant. The downstream sub-model starts from BGRT and USB to KM 203.500. The low-water bed is calibrated in steady state (Fig. 4) and the calibration of the high-water bed friction coefficient is carried out in unsteady state (Fig. 5). The low-water bed is calibrated with stationary boundary conditions for a large range of input discharges. The comparison of water level computed (lines in Fig. 4) and profiles water level measurements (points in Fig. 4) highlights an average difference lower than 10 cm. Simulations of 2002 [2] and 2003 [3] floods enable to check the calibration of low-water bed bottom frictions and to adjust high-water bed bottom frictions. The comparison of water levels calculated (lines in Fig. 5) and recorded (points in Fig. 5) exhibits a maximum difference of 15 cm at flood peak. In the floodplain, the comparison between water levels and flood marks shows an average difference of 25 cm. Consequently, the sub-models accurately represent observed water levels.

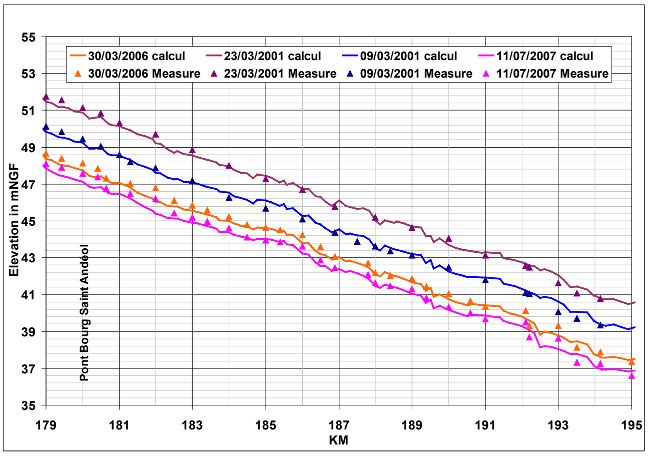

Figure 4. Calibration of low-water bed bottom friction – Downstream sub-model.

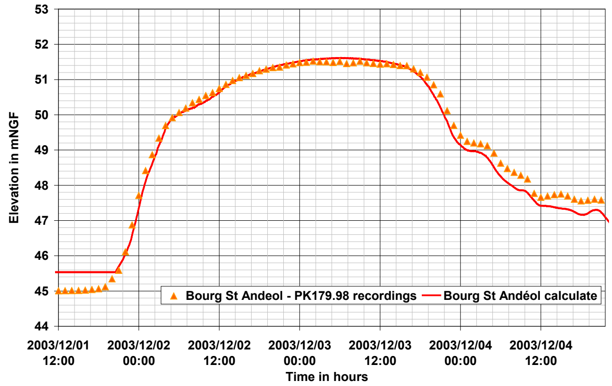

Figure 5. Checking of bottom friction low-water bed calibration for the 2003 flood at Bourg Saint Andéol hydrometric station – Downstream sub-model.

IV. SUBROUTINE

A. Equations

A Specific subroutine is carried out on TELEMAC-2D version V5P9 to calculate flow rate passing through, or water level upstream, CNR structures. The subroutine comprises three main options: weir, gate and regulation. The first two options solve equations based on energy conservation. To expose then, two variables are defined: the vertical coordinate:

$$y = z - z_c \tag{1}$$

where z is an average of water level in the section defined by the user and z_c the weir crest elevation, and the specific energy E [4]:

$$E = y + \frac{V^2}{2g} \tag{2}$$

where g is the acceleration of gravity and V is an average of scalar velocities in a section defined by the user.

The weir equations are obtained applying Bernoulli equation between upstream and downstream weir crest and considering a rectangular weir. The unsubmerged weir equation is:

$$Q = C.L.\sqrt{2g}.\sqrt{1/3}.\frac{2}{3}.\left(E_{upstream}.\right)^{\frac{3}{2}} \tag{3}$$

and the submerged weir equation is:

$$Q = C.L.y_{downstream}\sqrt{2g}.\sqrt{E_{upstream} - y_{downstream}} \tag{4}$$

where L is the weir crest width and C corresponds to the weir conveyance coefficient.

In addition, gate conveyance equations are implemented applying Bernoulli formula and considering a rectangular gate opening. The unsubmerged gate equation is:

$$Q = C_{ctr} \cdot C \cdot L \cdot \sqrt{2g} \cdot O \cdot \sqrt{E_{upstream} - O} \tag{5}$$

and the submerged one is:

$$Q = C_{ctr} \cdot C \cdot L \cdot \sqrt{2g} \cdot O \cdot \sqrt{E_{upstream} - y_{downstream}} \tag{6}$$

where L is the gate width, C_{ctr} is the coefficient of the streamline contraction, C corresponds to the gate coefficient and O is the gate opening (in meters). The gate opening can vary with time following operator instructions.

To comply gate operating rules, the regulation option calculates the flow rate to be prescribed downstream in order to respect the appropriate water level at the regulating point.

B. Description

Within the mesh, a structure is materialised as a rectangular island. The island is delimited with four boundary conditions: two liquid boundary conditions (upstream and downstream) in the mainstream direction and two solid boundary conditions corresponding to the lateral structure ends. The liquid boundary conditions (mainly prescribe discharge) are managed by Fortran programming.

C. Adaptation to Donzère-Mondragon development

Five structures have to be considered in the modelling of Donzère-Mondragon development. As the simulation starts with a high flow rate, the following configuration is chosen:

- BGRT: opened so weir equations ((3) if unsubmerged weir and (4) if submerged weir);
- BGU: gate equations with opening law to be determined;
- USB: regulation option with water level law vs. upstream discharge to be determined;
- NPN: closed so modelled by dyke that breaks in case of overflow,
- APN: closed so modelled by dyke without break possibility.

V. SIMULATIONS

A. Stationary discharge conditions

Prior simulating the extreme flood in unsteady mode, the model has to be initialised. In order to reduce water level instabilities at the immediate vicinity of the structures, a first calculation is launched with the sub-models generated in the calibration phase. The water level and flow resulting from this calculation are interpolated thanks to Fudaa PréPro software and an initial “Selafin” file of the whole model is generated.

Subsequently, the law of BGU opening vs. input discharge (at Viviers bridge station) is determined iteratively so as to respect water level at PR3 for various input discharges. It has been done in steady mode.

The USB law of water level vs. discharge, immediately upstream the power plant, is calculated for input discharges higher than the design flood. During flood, around half of the discharge (and more for extreme flood) pass through the six gates and the two surface gates. The law is determined using unsubmerged equations: (3) for surface gates and (5) for gates. The gate and weir coefficients are calibrated. The accuracy of the weir coefficient calibration is shown in Fig. 6 where calculations (blue line) and physical modelling measurements (pink squares) are compared.

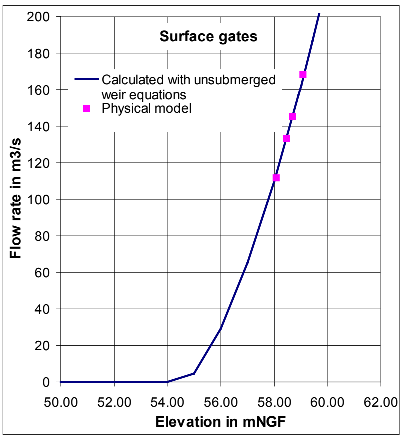

Figure 6. Calibration of weir coefficients for USB surface gates.

B. *Unsteady state*

As BGU opening law is calibrated with stationary boundary conditions, it has to be checked in unsteady mode. In Fig. 7,the PR3 water level, orange dashed line, is lower than the maximum authorised elevation, with a 50 cm toleration, as long as the input flow rate at Viviers station (dark blue line) is lower than the design flow (time is lower than t1 where t1 corresponds to the latest time when development is in operation). Thus, BGU opening is well calibrated.

Prior to flood peak and immediately upstream the NPN, the water level overreaches the maximal head tolerated by the NPN structure. Consequently a break is created [5] and launched at t2.

The discharge passing through the BGU is a relevant indicator of the development behaviour representativeness. Indeed, as long as the BGU opening is manoeuvred, its flow rate (light blue line) is stationary. Afterward, the flow rate increases following the input discharge rising (at Viviers) and since gate opening is fixed. When NPN break occurs, BGU flow rate suddenly decreases given that most of the discharge passing through the BGU is diverted into the NPN wide opening.

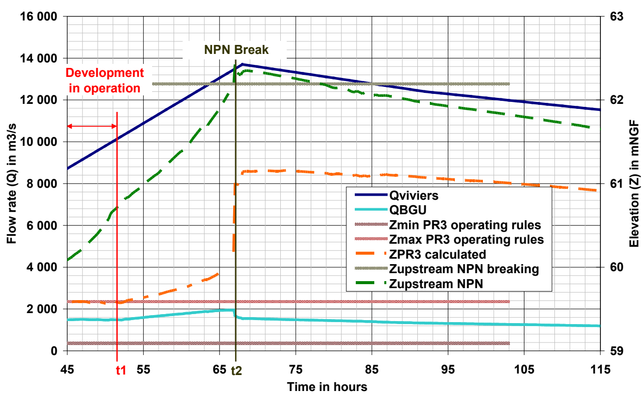

Figure 7. Flow rates and water levels evolution for the extreme flood.

In case of extreme flood, almost the whole Pierrelatte plain is flooded (Fig. 8). Water depths can reach 6 meters in gravel-pits (downstream part of the model). Inundations occur on the left bank of the headrace channel. The velocities (red arrows in Fig. 9) exhibit overflows within the headrace channel due to NPN break. They are located predominantly upstream the SNCF bridge.

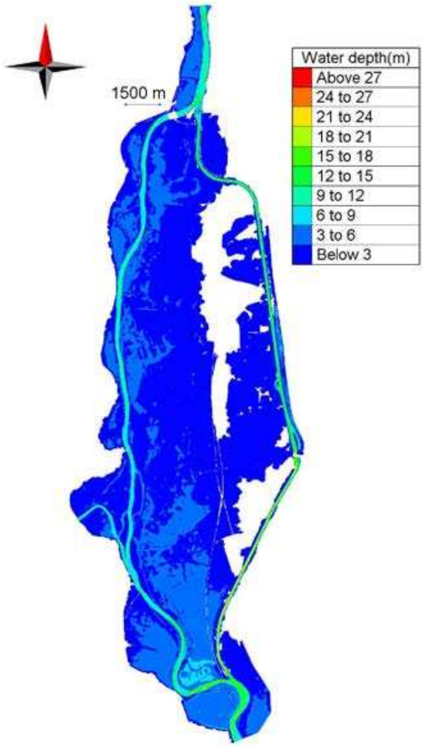

Figure 8. Map of the maximum water depths in case of extreme flood within Donzère-Mondragon developments.

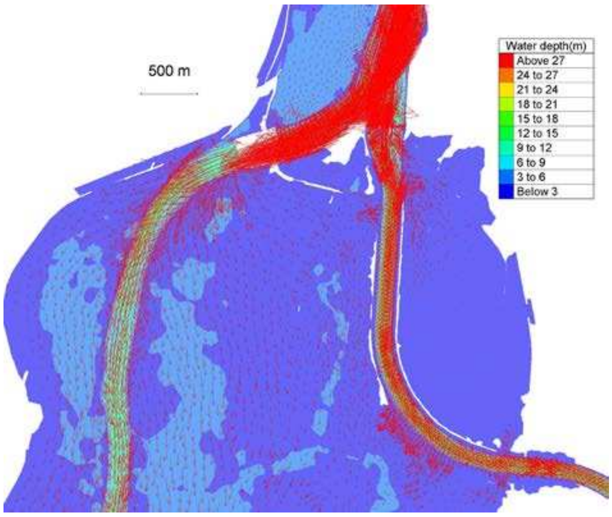

Figure 9. Map of the maximum velocites UV in case of extreme flood at the upstream part of headrace channel.

C. Technical problems and improvments

The problems encountered concern:

- mesh distortion: it is noticed that the water levels and velocities analysis is sensitive to strong distortions. A solution is to create a utility program giving every element distortion so as to modify the mesh. N.B. Distortion maps generated by Matisse don't seem to be exploitable;
- flow rate variation: a small variation in water levels can induce a high variation in flow rates, especially at the initialisation. The solution was to add a relaxation variable Q correction ;
- In TELEMAC V5P9 version, it was not possible to prescribe at a boundary condition depending on the discharge passing through a selected section. A programming was attended but didn't succeed due to discharge calculation instabilities. The upgrading of V5P9 in V6P1 could solve this problem.

VI. CONCLUSIONS

A hydrodynamic 2D modelling of Donzère-Mondragon development has been built and calibrated with TELEMAC-2D. Specific subroutines have been implemented within the model to consider CNR structures during extreme flood. The main outcomes of the article are as follows:

(1) The development behaviour is successfully represented and operating rules are complied.

(2) In case of an extreme flood, high water levels are calculated in Pierrelatte floodplain and overflows are noticed in the headrace channel.

(3) Program limitations are identified: the subroutine only answers to flood issues. Furthermore, instabilities in flow rate calculation prevent regulation from running at a specific section.

The present results consider all CNR structures are in operation. The next step of the study will be to change CNR structures configuration: USB units out of order or a BGRT gate closed for maintenance and observe water levels evolution. Considering subroutine improvements, prospective is to implement a complete regulation of each barrage gate to ensure fulfilment of the operating rules from low flow to floods.

REFERENCES

[1] J.-M. Hervouet, Hydrodynamique des écoulements à surface libre, Modélisation numérique avec la méthode des éléments finis, Presse de l'école nationale des Points et chaussées, 2003

[2] S.Raimondo, S.Reynaud, E.Divet, Vallée du Rhône, Rapport de crue de novembre 2002, CNR, unpublished

[3] S.Reynaud, Vallée du Rhône, Rapport de crue de décembre 2003, CNR and Météo France, unpublished

[4] Richard H.French, Open Channel Hydraulics, Water Resources Publications, LLC, 2007, p53-p55

[5] M. Pochat, S.Trossat, Dykes break modelling, Presentation at the 2009 Telemac User Club, CNR, unpublished