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Numerical and experimental study of Favre Waves with TELEMAC-3D and MASCARET

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***Abstract* — CNR is the first producer of 100 percent renewable energy in France, operating and managing 19 hydroelectric power plants on the Rhone River. After sudden shut-down of turbines because of mechanical or electrical incidents, wave is generated that raises the water level of the head race channel. This operation is either called disjunction, or rejection wave. Improving the knowledge of this phenomenon is essential for CNR in order to implement suitable actions both at the barrage and at the power plant.**

Following recent modelling with TELEMAC-2D & 3D, being calibrated with measurement on Chautagne head race channel [1], 3D modelling appears the most consistent one. However, reserves have to be done about the secondary wave amplitude, called Favre wave. To evaluate more accurately the relevance of TELEMAC-3D software and of MASCARET 1D as well, trials on a physical model have been conducted at CNR laboratory, few experiments being available. Tests have been performed on a rectangular channel, representing the head race channel. Slope can be modified on the physical model. The sensitivity of numerical parameters with TELEMAC-3D is highlighted by comparison with laboratory measurements.

I. INTRODUCTION

The disjunction of a hydroelectric power plant consists in the sudden turbine shutdown, which becomes necessary in the following cases:

- Decoupling between turbines and electricity transmission network. All the turbines are concerned and must be quickly stopped under threat of destruction by over-speed,
- Mechanical (part being broken, vibration) or electric malfunction, which concerns *a priori* one turbine at worst. The closure is also required.

A wave is then produced in the headrace channel. However, its amplitude can be reduced or almost cancelled, depending on available structures:

Opening the dedicated gates is any. This solution works for cases number 1 and 2,

Using the turbines: water crosses them after having moved the blades in the neutral position. This solution is hardly possible for case 2. This system is typically used for bulb turbines.

Thereafter, mitigation ways are not taken into account therefore studies concern the maximum possible wave only. The plant shutdown is usually less than one minute, generating great waves upstream, with additional amplitude along the banks, with possible risk of overflow over the dyke crest. Downstream of the plant, a negative wave forms with a risk of draught shortage for vessels, if any. The study and the predictive understanding of these waves is therefore vital for CNR, in terms of safety.

In 2010, a disjunction test at current flow (500 m³/s i.e. 71% of equipment rate) was held at the Chautagne power plant on upper Rhone River, France. This test allowed us to obtain water level measurements at numerous points along the development. In 2013, modelling studies were conducted to compare simulation results with experimental data. Numerical modelling CRUE 1D (internal code of CNR) and TELEMAC-2D, are able to represent the primary waves only. The secondary waves, called Favre waves, are also generated, due to the vertical acceleration. They are superimposed on the primary waves. These secondary waves are modelled with TELEMAC-3D. The results obtained, after a significant refinement of the mesh, are promising, nevertheless don't match completely the field observations recorded during the test [1] (see Fig. 1).

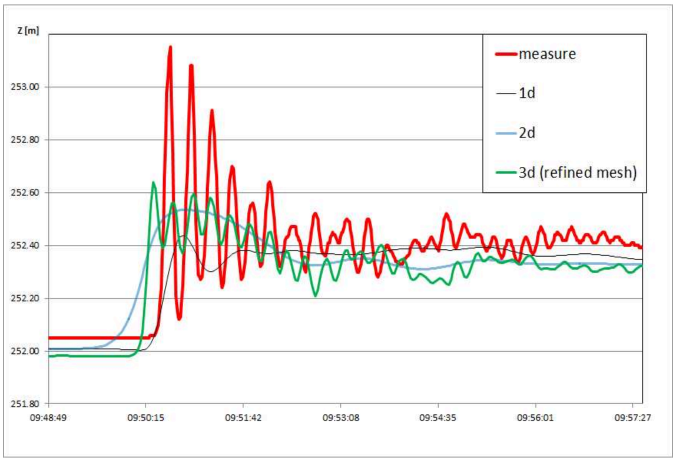

Figure 1. Measures and calculations of wave disjunction (Chautagne development [1]).

To go further ahead, it is decided to test TELEMAC-3D by considering a much simpler shape in order to get rid of the Chautagne complex geometry.

The MASCARET model is tested as well (developed by EDF), which incorporates the vertical acceleration.

This publication presents the comparison of results of these two numerical models with experimental measurements on a rectangular scale-model channel.

II. OBJECTIVES

The main objective is to replicate precisely the secondary waves with numerical models. In this paper, numerical modelling is performed in a similar way as in the laboratory one: propagation of the secondary waves in a rectangular channel with variable slope.

The advantage of a laboratory experiment is twofold: on the one hand, to avoid the superposition of complex hydraulic phenomena on an existing hydropower development (curvature of the channel, bathymetric variations, presence of lock, etc.), on the other hand to make measurements without risk for the actual development.

Objectives of this study are as follows:

- Perform measurements and initiate a database, regarding the secondary waves, in a simple rectangular channel for different slopes;
- Achieve 1D and 3D numerical simulations and compare the calculations with the laboratory measurements;
- Evaluate the software relevance for calculating rejection waves and provide recommendations to CNR operating department.

III. EXPERIMENTAL TEST CASE

A. *Experimental model*

The CNR hydraulic laboratory has a rectangular channel whose dimensions are: 6 m long, 7.5 cm wide and 16 cm high. This channel is equipped with a recirculation system with water tank and a motorized slide valve. The channel can also be tilted from 0 to 4°, which allows to test different flow configurations. A position sensor is used to calculate the valve closing speed. The shutdown is total. The channel represents a vertical slice of flow.

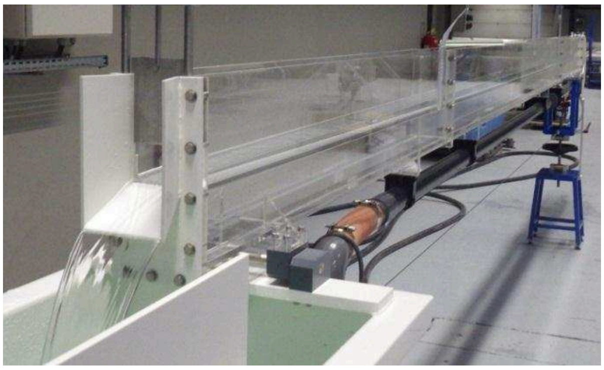

Figure 2. View from downstream of experimental channel.

B. *Scenario choice*

The scenarios are determined by the following variables: inclination of the channel, inflow and closing speed of the valve. The slope must be sufficiently low so that the Froude number is less than 1.3. Indeed, the literature [3] indicates that for a Froude number greater than 1.3, there is no secondary wave, since the primary wave breaks.

The slope must be similar to the Rhone one, to wit between 0 and 0.2°, 0.3491% being chosen for the experimental channel.

Experimental system does not allow a great accuracy (variation of two millimetres on the ruler corresponds to 0.1°).

To optimize the understanding of the physical phenomenon and to define a suitable program of tests, preliminary trials are performed first. Tests are made with different slopes: 0°, 0.1°, 0.2° and 0.6°, in order to highlight the slope effect. For these four cases, tests with six different flow rates are achieved: from 1.6 to 10 m^3/h (from 0.44 to 2.77 l/s).

The model results can be interpreted at any scale. For example at 1/100, it represents discharges between 44 and 278 m^3/s, per width of 7.5 m; depth of 16 m and for an average width of 50 m, 296-1850 m^3/s. Similar cases of the case quoted for Chautagne, would be:

- At 1/100 for 3 l/h: 16 m water depth – 50 m average width 50 m - 556 m^3/s - 1/10 scale time;
- Also at 1/50 for 8 l/h: water depth 8m - 50m average width – 524 m^3/s - 1/7 scale time.

The tests performed on the model can therefore represent a wide range of real cases.

The secondary waves hardly form for the steepest slope (0.6°), the flow being supercritical. Moreover, wave heights are similar for 0.1° and 0.2°. Finally tests are performed for zero and 0.2° slope only, confirming previous assumptions.

The discharge determines the time when the waves appear. For low discharges, the secondary waves appear early but tend to fade (especially for zero slope). Otherwise, for high discharges, the secondary waves (amplitude *h**) appear later.

TABLE I. EXPERIMENTAL TEST LIST

	Scale	
Entrance discharge	**1**	**100**
Q1	$4.44.10^{-4}$ m^3/s – 0.44 l/	44.44 m^3/s
Q2	$1.67.10^{-3}$ m^3/s – 1.67 l/s	166.67 m^3/s
Q3	$2.78.10^{-3}$ m^3/s – 2.78 l/s	277.78 m^3/s
Valve closure speed	**1**	**100**
V1	$1.5.10^{-2}$ m/s - 15 cm/s	1.5 m/s
V2	$1.0.10^{-2}$ m/s - 10 cm/s	1 m/s
V3	$0.5.10^{-2}$ m/s - 5 cm/s	0.5 m/s

The closing speed of the valve is also an important factor, which acts on the amplitude and wave form. Indeed, faster is the closure, greater the primary waves are near the valve; but smaller the secondary waves are.

C. *Laboratory measurements*

An ultrasonic level sensor is selected, adapted for measurements on small height variations on a short time.

Direct sensor accuracy is 0.025 mm to 0.18 mm, providing in association with the acquisition system, a global precision close to 1mm.

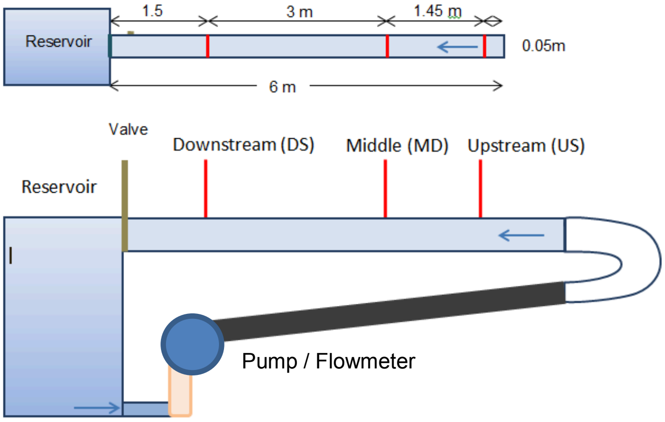

Figure 3. Scheme of the experimental system and the sensors places. Plan view and cross section.

D. *Experimental results*

After analysing experimental results, initial assessments with the Favre theory [2] can be made: higher the initial water depth is, faster the primary wave is.

Greater the discharge is, lower the frequency of the secondary wave is and it can even disappear, within the three closing speed performed of the valve. No secondary wave was observed at the downstream sensor for the speediest closing tests (V1 and V2).

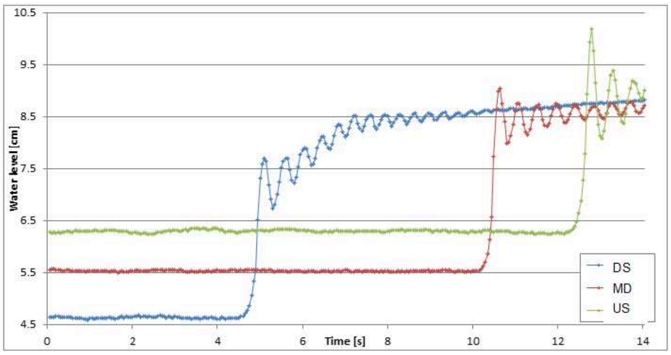

Figure 4. Measured water level at the 3 sensors.

The closing speed of the valve determines the generation of the secondary waves. The secondary wave appears later and more upstream when the closure is rapid. Experiments also revealed that the slope of the canal is also an important factor. Favre waves appear later and more upstream channel, when the slope is greater, with decrease of the celerity and of the wavelength. Consequently, following paragraphs focus on the case of a horizontal channel.

IV. 1D NUMERICAL MODELING

A. *MASCARET code*

The calculations are performed using the supercritical kernel with 7.1.7 release (Fudaa-MASCARET 3.1.9), because non-hydrostatic terms are taken into account, unlike the two previous codes (CRUE and TELEMAC-2D).

The numerical resolution of the software uses the Shallow Water Equations (SWE). As regards the specific case of the inclusion of non-hydrostatic terms, the software includes the two-dimension Euler equations.

B. *Mesh*

The channel geometry developed on the Fudaa-MASCARET is quite simple: two nodes form a reach. Absolutely vertical walls are theoretically not possible. Nevertheless the software requires one millimetre abscissa shift only at the base of the channel. In order to minimise the bias, the scale 1 to 100 is used, consequently the width of the offset is 1 mm only over a width of 7.5 m. The other reason for this choice is the packaging recovery calculation imposed by Fudaa-MASCARET, implying a limitation in the number of significant digits.

Cinematic similarities are needed to subsequently compare the results from the numerical and the physical models.

C. *Numerical settings*

Space step is 3.5 m at scale 100 and time step is 5.10^{-2} s, without forcing neither the implicitation kernel nor the friction.

Calculations take into account the friction on the walls. The non-hydrostatic option is used. Secondary waves are then present by enabling this previous option, as shown in Figure 5.

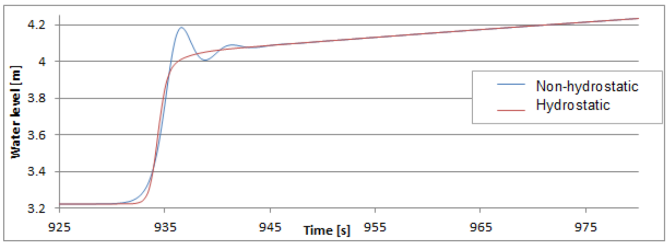

Figure 5. Secondary wave check by 1D non-hydrostatic assumption (at scale 100).

D. *Calibration*

The methodology is based on a computation, using an initial stabilized state. This calibration of the model is performed adapting the roughness coefficient on the initial hydraulic conditions before the disjunction. Taking into account the similarities of a model at scale 100, the Strickler coefficient is then 70 $m^{1/3}.s^{-1}$. Some discrepancies are observed between the calculated and measured water levels, because the friction and the roughness do not react in the same way between the numerical and the physical model. The greater losses of head on the scale model can be explained by the narrowness of the channel as well as the hydraulic conditions ahead.

Error calculations are made to determine the best coefficient for finding an average water level close to experimental tests.

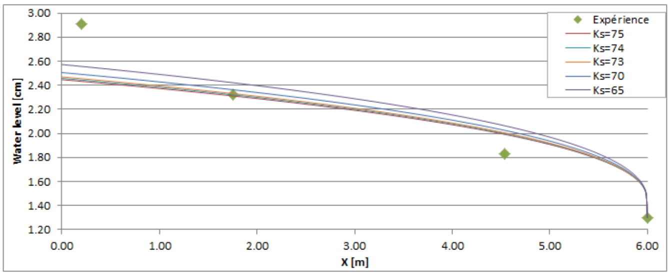

Figure 6. Water level profiles for different Strickler coefficient values, compared to measurements.

E. *Results*

Despite some setting difficulties, all disjunction tests are calculated by a recovery calculation, imposing the inflow and the outflow, decreasing to zero. For low discharges and sudden valve closures, the model provides overestimated results, compared to the observations, orders of magnitude being however respected. Results obtained with MASCARET are compared to laboratory measurements hereafter:

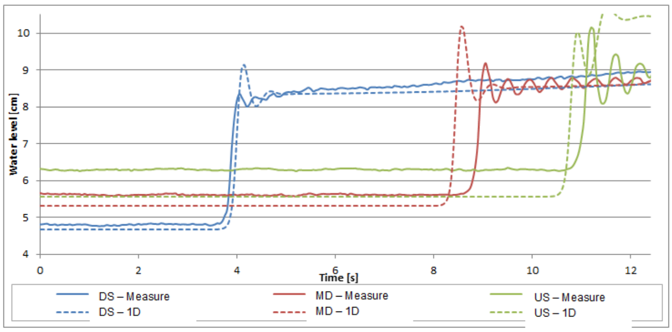

Figure 7. Water levels for scenario Q2-V2 with MASCARET.

Except downstream, the secondary waves on the model appear faster than ones calculated by MASCARET. On the model, oscillations are systematically more numerous than ones calculated: two oscillations only, when several are observed at the sensors MD and US.

The celerity of the primary wave is slower on the model, its wavelength and relative amplitude (amplitude versus baseline) as well.

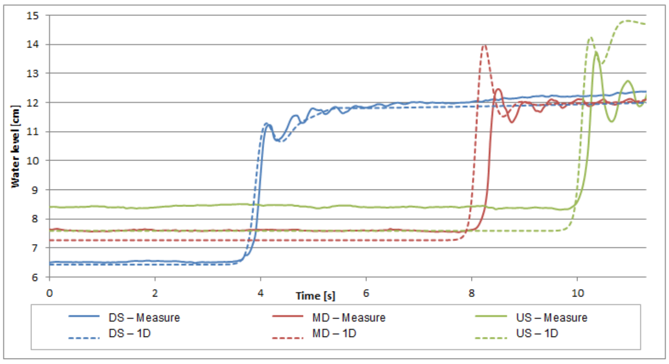

Figure 8. Water levels for scenario Q3-V2 with MASCARET.

At the downstream sensor, signal is quite consistent for the first oscillation (Figure 8). However at the other sensors, the same trends are observed, i.e. fewer oscillations with MASCARET.

Trends observed in other scenarios are identical to those presented above. In conclusion, the 1D numerical model tends to overestimate the magnitude of the wave once it is developed in the channel (upstream sensor); which is in line with the safety.

F. *Influence of valve closure*

During a valve closure, the discharge neither decreases immediately nor linearly. 2 types of valve closure are tested: linear and double slope (hydrograph by MASCARET). In summary, the type of closure has an influence on the formation of secondary waves. However, none of the proposed modelling seems the best. It would be necessary to measure the actual discharge passing under the gate to get the actual hydrograph closure, for more realistic results.

V. 3D NUMERICAL MODELLING

A. *TELEMAC-3D code*

It is chosen to perform the calculations using TELEMAC V6P2 release (parallel version, 32 cores).

Following the parameter optimization studies [1], it is chosen to use TELEMAC-3D non-hydrostatic release.

B. *Mesh*

Mesh is created using the MATISSE software, the geometry is fairly simple to generate. In contrast, representation of the side walls is complex: quick tests showed that it is not possible to consider the walls as smooth borders or taking into account the friction (Nikuradse law). Although there are limitations to the wall mesh representation (pseudo-vertical) to the consideration of friction, this solution is used by checking at least 2 wet nodes on walls in the original hydraulic condition.

Mesh consists of 11,091 nodes and 20,960 elements, the space step of about 0.01 m on the bottom of the canal and 0.005 m on the walls.

C. *Numerical settings*

The calculation is performed by 3D recovery file from previous simulations (previous hydraulic parameters calculated: water depth, horizontal and vertical velocities). Main parameters are shown in Table II.

TABLE II. TELEMAC-3D PARAMETERS

Parameter	Value
COEFFICIENT FOR VERTICAL DIFFUSION OF VELOCITIES	1.10^{-6}
COEFFICIENT FOR HORIZONTAL DIFFUSION OF VELOCITIES	5.10^{-3}
NUMBER OF HORIZONTAL LEVELS	6
TIME STEPS	5.10^{-4} s
NON-HYDROSTATIC VERSION	Yes
DYNAMIC PRESSURE IN WAVE EQUATION	Yes
VELOCITY VERTICAL PROFILES	2 (log)

The tests carried out by integrating a vertical turbulence model (mixing length) show no significant effect on the calculation of the secondary waves in the rectangular channel. On the channel with horizontal slope, with the flow rate Q1, sensitivity analyses are carried out on the number of layers, on the time step and on the difference between hydrostatic and non-hydrostatic release. This last comparison is shown in Figure 9. Favre waves are modelled by non-hydrostatic version, otherwise the primary wave only is visible.

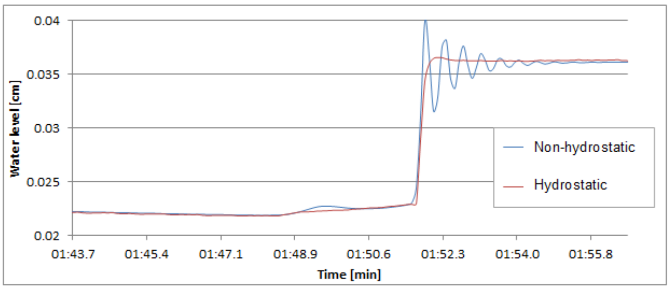

Figure 9. Secondary wave check by 3D non-hydrostatic assumption.

Then a study about the number of horizontal layers is carried out. A high number of layers causes a lower maximum water depth, close to the observations made during the scoping tests. A lag time of around 0.1s is also noticed. In regard to the volume of the model with six horizontal layers (66,546 nodes, 104,800 elements) and relative low variation of the results, it is chosen to adopt 6 layers.

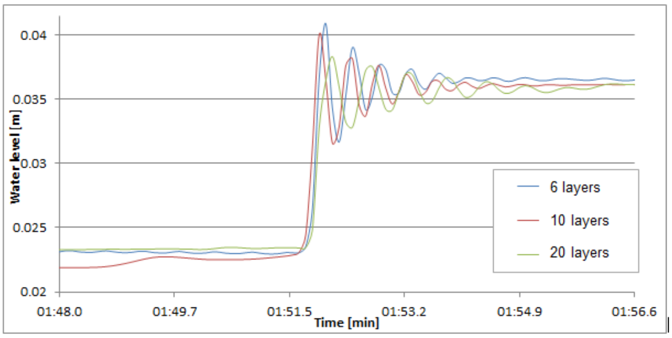

Figure 10. Number of layers impact – water levels.

D. *Calibration*

The best calibration of the initial state is obtained by imposing a flow entering and water level exiting, with real uniform roughness (Strickler coefficient 70 $m^{1/3}.s^{-1}$) and 6 layers. Figure 11 shows the calculated water levels and the data points for Q2.

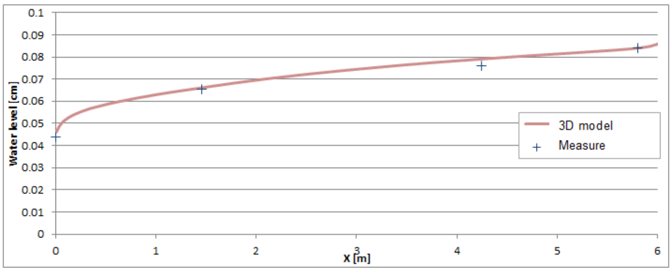

Figure 11. Water level for baseline with TELEMAC-3D for scenario Q2.

E. *Results*

The results are presented on the horizontal channel. Figure 12 shows the calculations compare to the measures in Q3-V1 scenario.

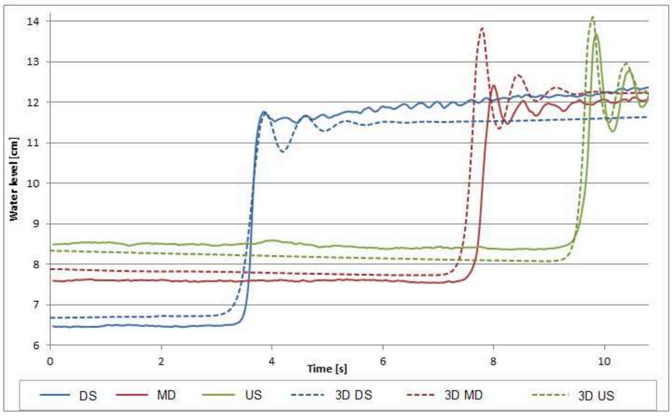

Figure 12. Water levels for scenario Q3-V1 with TELEMAC-3D.

Simulated and observed curves are relatively close, in particular for the upstream sensor. The secondary waves occur always earlier on the numerical model. Analysis confirms that wave celerity is 3% greater, its period and its wavelength as well. The relative amplitude calculated is very close to the observations at the upstream sensor (2% error), but quite different at the middle sensor. In all cases, the numerical model leads to an overestimation of the maximum amplitude reached by the waves, in line with the safety.

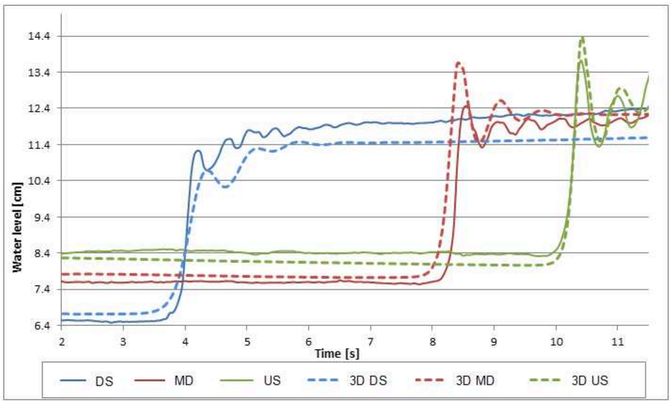

Figure 13. Water levels for scenario Q3-V2 with TELEMAC-3D.

Same trends are observed: celerity and wavelength are slightly greater in the numerical model. The upstream amplitude is well calculated and an excess response in the middle (same characteristics with Q3-V3 test). The conclusion is that TELEMAC-3D reproduces correctly the phenomenon of secondary waves, despite an overestimation tendency, lining with the safety.

F. Influence of layer number

In the upstream part, best results are obtained using 6 layers with a best relative amplitude value (wave amplitude compared to baseline). At the middle sensor, previous trend is not so obvious, the two models providing similar results. The model constructed with 20 layers conducts to the best results concerning both frequency and wavelength. For the relative amplitude and the celerity, which correspond to the reference criterion for CNR, best results are obtained with the 6 layer model. Consequently, the 6 layer model is largely favoured.

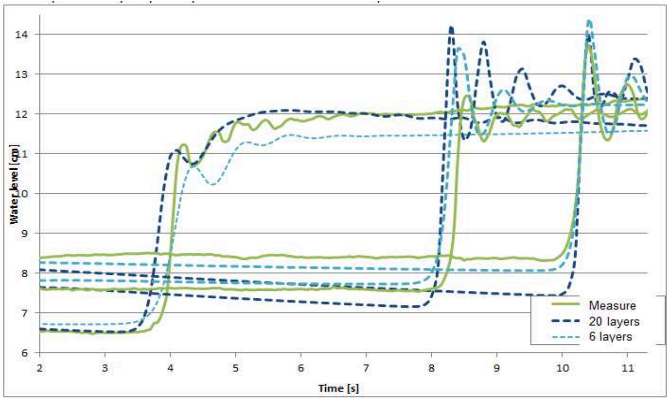

Figure 14. Number of layers impact – water levels for scenario Q3-V2.

VI. 1D-3D COMPARISON

While MASCARET seems more precise than TELEMAC-3D regarding water depth and amplitude (Figures 8 and 13), TELEMAC-3D is more precise regarding the wave celerity, the frequency of the secondary waves and the number of oscillations. In all calculations provided by MASCARET, secondary wave diminishes much faster the secondary waves than the other code. Figures 15 - 16 show results for scenarios Q3 and Q3-V2-V3.

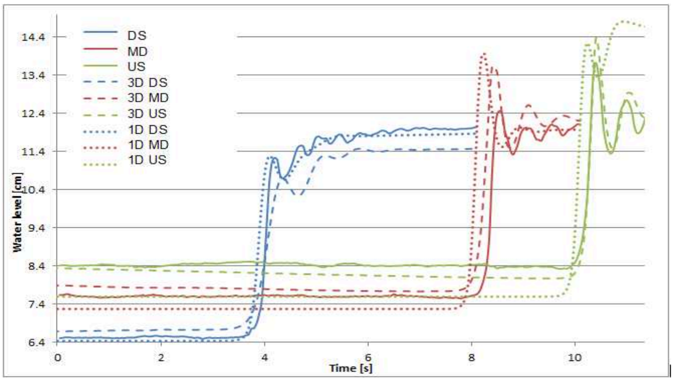

Figure 15. Water levels for scenario Q3-V2 with TELEMAC-3D and MASCARET.

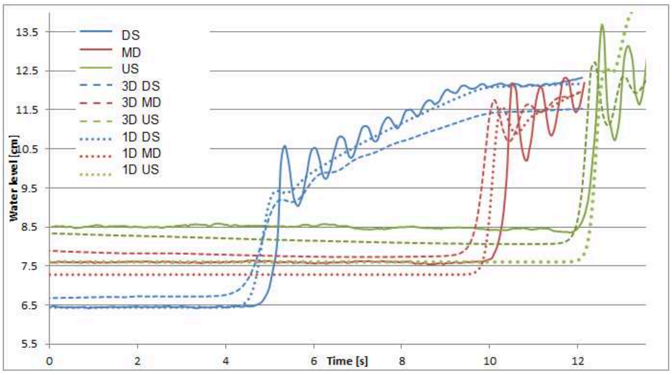

Figure 16. Water levels for scenario Q3-V3 with TELEMAC-3D and MASCARET.

VII. CONCLUSION

As a result, 1D and 3D (with vertical acceleration) numerical models don't get large differences. Compared to physical model measurements, discrepancies have the same order of magnitude.

A more pronounced attenuation of the signal is observed for the 1D modelling.

MASCARET and TELEMAC-3D overestimate the amplitude of the secondary waves. This is fundamental regarding the hydraulic safety, which leads to consider both models as operational for disjunction calculations.

Compared to 3D, 1D approach is obviously easier and faster to implement for calculations.

In this experimental case, the 3D code performance is far from having been fully used, because of the simplifying assumptions such as rectilinear uniform rectangular channel. 3D calculation would undoubtedly be efficient in curves, edge effects and abrupt changes in geometry. These accidents are common for the run of the river developments.

Headrace, channel, lock, tributaries, are also opportunities for the basic wave to split, to reflect and therefore to become more complex. Nevertheless 3D calculation time is expected to be enormous, therefore only some key cases should be tested; regarding the safety again.

At this stage, we recommend using the 1D code for a simple headrace channel, but to use TELEMAC-3D for a full development or a complex headrace channel. A comparative study would be needed on an existing Rhone River development (1D, 3D and measures) for different cases of disjunction (partial to total uncompensated).

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