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- Numerical study of impacts generated in a dam break varying depth and obstacle position
- All types of wave impacts were obtained through this set-up by adjusting the parameters
- Impact pressures one order of magnitude larger than in previous dam break studies
- Wet dam break may be quick and simple way to generate strong wave impacts

Numerical study of the wave impacts generated in a wet dam break

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Abstract

This paper focuses on the numerical analysis of the different wave impacts generated in a wet dam break by varying the downstream depth and the position of the obstacle. With this relatively simple set-up, five hundred impacts were simulated, providing a comprehensive dataset covering the four classical impact types : sloshing, flip-through, aerated and broken wave impacts. We determined the dam break parameters for which the flip-through impacts occur and showed that among all the flip-through impacts, there is one optimum generating a pressure peak larger than the others. Compared to impact pressures recorded or simulated with a dam break flow in most previous studies, the present numerical work reached values one order of magnitude larger. Therefore with a dam break on wet bottom, it seems possible to generate strong wave impacts in a short duration. This has to be verified experimentally specifically focusing on the measurements repeatability due to the obvious effect of the gate dynamics effect on the subsequent flow. When and if confirmed, this simple set-up could be used to validate numerical models without the need of a long propagation stage before impact.

Keywords: Dam break, wet bottom, wave impact, flip-through, Navier-Stokes model, Volume Of Fluid, impact pressure.

1. Introduction

Strong wave impacts may cause important damages on coastal or ocean structures such as breakwaters or marine renewable energy converters [16]. Additionally, for instance in ships carrying liquids in partially full tanks, violent sloshing phenomena can also lead to wave breaking susceptible to damage the membranes placed inside the tanks or endanger the stability of the vessel.

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7 The literature highlights four type of wave impacts (e.g., [24], [22]) depending on the
8 position of breaking point with respect to the obstacle. In the case of a broken wave, a
9 turbulent bore breaker with low vertical and uniform horizontal velocities is generated. In
10 that case, the impact is dampened by the foam and as a result, pressure reaches relatively
11 low values. When the breaking point is closer to the obstacle and in case of a plunging
12 breaker, a volume of air can be entrapped between the wave and the wall. If this volume
13 is significant, two pressure peaks can be observed. The first one corresponds to the crest
14 impact on the wall while the second one is due to the compression of the air pocket and is
15 applied on a larger zone for a longer duration. If the breaking point is located just before
16 the wall, only a small air cushion is trapped and transformed into small bubbles after the
17 impact. This kind of impact -referred to as "flip-through impact" when the air cavity is
18 as small as possible - leads to the highest pressures. A very sharp single peak is observed
19 sometimes followed by high frequency oscillations (due to the expansion and contraction
20 of the small air pocket). The flip-through generates a very particular time history of pres-
21 sures referred to as the "church roof profile" in Peregrine [48]. Finally, when the trough
22 level reaches the theoretical impact point before the crest hits the wall, an upward deflected
23 breaker is created. This kind of breaker leads to a sloshing impact for which two relatively
24 small peaks are generally observed. They correspond to the first impact and then to the
25 overturning of the upper part of the wave that was concave and becomes convex. Hull
26 and Muller [25] and Scolan [52], both showed that a smooth transition between these four
27 types of impacts could be obtained by increasing gradually the amplitude of the wave.

28 In the flip-through impact, the wave approaching the wall has a very steep (almost
29 vertical) face. A flat impact could therefore be expected. Nevertheless, the wave trough
30 is blocked by the wall and starts running-up on the wall which causes the focusing of the
31 free surface at one point on the wall. This leads to the formation of a vertical jet with very
32 high accelerations ($> 1000g$) and local pressures superior to $10\rho g(H + h)$ with h water
33 depth and H wave height [48]. Experimental and numerical studies have been carried out
34 in order to get a better understanding of the flip-through phenomenon. Lugni et al. [39]
35 obtained waves impacts through the longitudinal forced motion (sway) of a prismatic tank
36 (sloshing). Accelerations up to $1500 g$ were reported in this study. Hattori et al. [22], Hull
37 and Muller [25], Hofland et al. [23], Kimmoun et al. [31], Brosset et al. [10], Mai et al.
38 [40] generated violent impacts with focused waves in a flume. The latter are obtained by
39 forcing several different waves of which the parameters are very finely tuned to exactly
40 superimpose at the expected focal point.

41 Repeatability is a major concern when studying wave impact either experimentally or
42 numerically, the peak pressure value being largely variable for the same impact condi-
43 tions. This apparently chaotic behavior is due to the phenomenon, which mixes large scale
44 processes (wave propagation, wave breaking) and very local water/air interface physics.

45 Related to the latter matter, Lubin et al. [37] for instance clearly showed the different in-
46 stabilities which can lead to the atomization of the plunging jet in a breaking wave. As a
47 result, it is very hard to ensure that the local wave shape and dynamics just before impact
48 are exactly the same. For instance, in wave focusing, uncertainty may come from : the
49 wave paddle motion, inaccuracies in the still water level that can strongly affect the focal
50 point, small perturbations at the surface of the water due to the previous waves, instabil-
51 ities of the wave crest (Kelvin-Helmoltz instability), etc. As a consequence, a very long
52 and fine tuning of the parameters associated to a very high density of pressure sensors is
53 necessary to capture the flip-through phenomenon.

54 Numerical studies were also conducted to study the flip-through impact. As a matter
55 of fact, the first paper proving the existence of this particular impact is a numerical work
56 [12]. The same author also showed that a parabolic shape of the free surface before im-
57 pact is also a characteristics of the flip-through [13]. More recently, Bredmose et al. [9]
58 compared experimental measurements of focused wave impacts and simulations based on
59 a fully nonlinear potential-flow solver. Colicchio et al. [11] studied experimentally and
60 numerically, with a SPH model solving the Navier-Stokes equations, the sloshing inside a
61 tank subjected to horizontal oscillation. Scolan et al. [54] also compared numerical results
62 obtained by means of a desingularized technique (potential flow solver) with the results of
63 experimental sloshing of a prismatic basin. The latter study particularly underlined the im-
64 portance of finding an easier way to simulate wave impacts compared to sloshing. Because
65 of the complexity of the processes (focusing of waves and sloshing), the numerical models
66 require a long time of computation before the actual impact often leading to difficulties in
67 the results interpretation.

68 If we leave aside the problem of repeatability and now analyze the results of the studies,
69 the main points of interest were the maximum impact peak pressures reached, the contro-
70 versial vertical position of the maximum impact pressure and the presence of a vertical jet
71 rising along the wall. Very high pressure peaks (i.e., in the range $10 - 100\rho gh$) were for
72 instance obtained in Hattori et al. [22], Lugni et al. [39], Bredmose et al. [9], Hofland et al.
73 [23].

74 The position of the maximum impact pressure on the wall remains a subject of debate.
75 According to Hattori et al. [22], Minikin [43] and Goda [19], the pressure peak values
76 are obtained at the still water level (SWL). Whereas Kirkgoz [32] and Allsop [4] observed
77 peak values under the SWL. Hull and Muller [25] measured peak values at the SWL for the
78 flip-through impacts and above the SWL for the cases with entrapped air. More recently,
79 Hofland et al. [23] conducted a large scale experimental study and found the peak values
80 to be reached above SWL for all impact cases. This characteristic of the impacts is still not
81 clearly understood and needs further investigations. Another feature of the flip-through as
82 described in Cooker and Peregrine [12], is the very fast jet rapidly rising along the wall just

83 before the crest hits the wall. Hull and Muller [25] added particle tracers to the flow but
84 failed to prove the existence of the aforementioned jet. Peregrine [48]), Lugni et al. [39]
85 and Scolan [52] obtained vertical accelerations of 1000g, 1500g and 1800g, respectively,
86 the largest value likely being the 59000g reported in [13].

87 Many features of the wave impacts and in particular the flip-through impact need fur-
88 ther investigation. Unfortunately, the lack of repeatability and the complexity of the exper-
89 imental ways of generating impacts make it difficult to control accurately the wave shape.
90 In this paper, we propose to use the dam break flow on wet bottom to generate wave im-
91 pacts. This classical hydraulic flow allows to generate almost instantaneously a breaking
92 wave [28] whose shape may be possible to control. Furthermore, by placing an obstacle
93 at the right position relatively to the generated wave, one can hope to obtain an impact
94 meeting for instance the conditions of the flip-through.

95 For a long period of time, studies mainly focused on the kinematic aspects of the dam-
96 break flows (e.g., [55]). In the last 15 years, some works started including the dynamic
97 aspects. Most of them consist in dry dam-break simulations and they are often limited
98 to the validation of the numerical model used for one specific case of dam-break with no
99 further intention of analyzing the dynamics of the impacts.

100 In the case of dry dam-break flow impacting an obstacle, limited values of peak pres-
101 sures were usually measured : $4.5 \rho gh$ [36], $1.35 \rho gh$ [30], $3.25 \rho gh$ [27], $2.7 \rho gh$ [29], 2.3
102 ρgh [41], $2.4 \rho gh$ [33] and $1.35 \rho gh$ [45]. As mentioned before, one can expect pressures
103 superior to $10 \rho gh$ in the case of a flip-through impact. The magnitude of the aforemen-
104 tioned impacts stays therefore quite limited. Even so, the dry bed cases put in evidence
105 some important characteristics of the dam-break that are also true in the case of the wet
106 bottom.

107 First, the motion of the gate has a strong influence on the results. It is therefore nec-
108 essary to take it into account in both, the numerical [29] and experimental [36] studies in
109 order to obtain a good repeatability and a good agreement between the results. Then, by
110 varying the upstream water depth or by moving the obstacle away from the gate, one can
111 expect to observe interesting variations of the impact pressures on this obstacle. Aureli
112 et al. [7] conducted several experiments in which the upstream depth varied from 7 to 13
113 cm in 5 mm steps. A quadratic relation between the impact peak force and the initial water
114 depth in the tank could be highlighted. Issakhov et al. [26] moved the obstacle further from
115 the gate and observed an increase of the impact pressure that was assumed to be caused by
116 the increase of the flow velocity.

117 Very few studies specifically concentrate on the dynamics of wet dam-break induced
118 wave impacts, and most of the time, the wet-bottom condition is a consequence of the
119 remaining water in the flume after the previous test (insufficient drying time) or is due
120 to leaks at the gate. More over, the impact studied is always the one of a turbulent bore

121 (e.g., [35]). Among those kind of studies, Al-Faesly et al. [3] for instance compared the
122 pressures on a 1m high square column in the case of a dry bottom and a wet bottom dam-
123 break with an upstream depth of 55 cm and a downstream depth of a few mm (2-3 mm).
124 In the dry case, the maximal pressure, equal to $5 \rho gh$ at 2 cm above the bottom of the
125 flume was larger than in the wet case (i.e., $1.75 \rho gh$). Larger depths ratio (i.e., downstream
126 to upstream depths) were used for instance in Arnason [6] or in Hartana and Murakami
127 [21] but the maximal pressure generated on the obstacle is always less than $5 \rho gh$ which
128 is the typical values for bore impacts. More recently, Zuo et al. [59] employed a Large
129 Eddy Simulation mesh-free incompressible model to study the impact of violent plunging
130 breakers generated by large upstream to downstream water depth ratios. Relative pressure
131 values $p/\rho gH$ (with H upstream water depth) of the order of 2 to 5 were obtained. Using a
132 scaling with the downstream water depth, this would give values around 50 which is quite
133 large. Nevertheless, only a few cases were investigated, the maximum pressure on the wall
134 was not recorded, and all the impacts illustrated in the figures involved the formation of a
135 pocket, which the monophasic incompressible model used could not properly describe.

136 The present paper proposes a first comprehensive study of the wave impacts generated
137 in the wet dam break flow with a focus on the sloshing and flip-through impacts. In this
138 work, the position of the obstacle with respect to the wave and the upstream water depth
139 is varied to generate different impact types and obtain variable pressure values. This first
140 study is purely numerical and will require further experimental validation to confirm the
141 results obtained. Nevertheless, if slight variations in the results may be expected, we think
142 that the overall conclusions of the paper will remain valid.

143 In section 2, the numerical model used and the set-up of the numerical experiment
144 are presented. This set-up is based on Jánosi et al. [28], which serves as reference for
145 the model validation. In section 3, the results of more than 500 hundred impact cases are
146 presented and discussed in section 4. Finally, the main conclusions of this work are drawn
147 in section 5.

148 **2. Method**

149 *2.1. Model*

150 The model used to perform this research is the multi-fluid Navier-Stokes Volume Of
151 Fluid (VOF) model THETIS. It has been validated in numerous studies related to water
152 waves [1, 38, 2, 46, 15, 17] and more specifically to wave impact in Mokrani and Abadie
153 [45] and Martin-Medina et al. [42].

154 The system of equations, also called one-fluid model, is composed of the incompress-
155 ible Navier-Stokes (NS) equations (1) and an additional equation for the interface evolu-
156 tion (2) :

$$\left\{ \begin{array}{l} \nabla \cdot \mathbf{u} = 0 \\ \rho \left(\frac{\partial \mathbf{u}}{\partial t} + (\mathbf{u} \cdot \nabla) \mathbf{u} \right) = \\ \rho \mathbf{g} - \nabla p + \nabla \cdot [(\mu + \mu_t)(\nabla \mathbf{u} + \nabla' \mathbf{u})] \end{array} \right. \quad (1)$$

$$\frac{\partial \chi}{\partial t} + \mathbf{u} \cdot \nabla \chi = 0 \quad (2)$$

157 In which, ρ and μ , respectively density and viscosity of the fluid, are spatially varying
 158 variables and $\chi(x, y, t)$ is a phase characteristic function equal to 1 in water and 0 in air.
 159 In THETIS, a selective function has been implemented to check that the velocity field is
 160 locally turbulent. In this case, a Large Eddy Simulation model is turned on with a subgrid
 161 viscosity, μ_t , calculated with the Mixed Scale model as described in Lubin et al. [38].

162 The NS equations are discretized on a fixed Cartesian grid using a finite volume formu-
 163 lation. Following Patankar [47], the finite volumes formulation is solved using staggered
 164 mesh known as the Marker And Cells (MAC) method from Harlow and Welch [20]. For
 165 this reason, in THETIS the mesh is composed of two grids: the pressure grid and the
 166 velocity grid. The coupling between velocity and pressure is solved using the projection
 167 method [18].

168 Equation (2) is solved by introducing a color function F defined as the average of the
 169 phase characteristic function $\chi(x, y, t)$ over the mesh cell. With this definition, F indicates
 170 the volume fraction occupied by water in a mesh cell and the interface position is defined
 171 as the iso-line $F = 0.5$.

172 At the end of each time step, the local cell water volume fraction is used to recalcu-
 173 late local values of density and viscosity necessary to solve the NS equations. For cells
 174 containing a fraction of water and air, equivalent density and viscosity are calculated by
 175 interpolations based on the water and air fraction. The equivalent density is here obtained
 176 with a linear interpolation : $\rho_{eq} = \chi \rho_w + (1 - \chi) \rho_a$ (with subscript w for water and a for
 177 air), which exactly computes the amount of mass present in the cell, while the equivalent
 178 viscosity is computed with the following harmonic average : $\mu_{eq} = \frac{\mu_w \mu_a}{\chi \mu_w + (1 - \chi) \mu_a}$, allowing to
 179 retrieve the right value of the viscosity in a simple shear flow [51].

180 Different VOF methods are implemented in the model THETIS to solve equation (2).
 181 The Piecewise Linear Interface Calculation (PLIC) method [58] is employed here with a
 182 correction step called SVOF-PLIC (S for Smooth). One of the great advantages of PLIC
 183 is to keep the discontinuous nature of the interface between water and air thanks to the
 184 Lagrangian character of the transport method. To ensure the stability of the PLIC method,
 185 a sufficient condition is that the segments are not advected over more than half of a cell size
 186 during a time step [1]. The SVOF-PLIC method, used in complement of PLIC, consists

187 in slightly smoothing the volume fraction function by introducing a controlled diffusion
 188 zone around the interface [49]. The color function includes a narrow diffusion zone that
 189 represents the interface more regularly. By this way, the discontinuities which may appear
 190 with the PLIC algorithm, and may sometimes lead to the computation divergence, are
 191 smeared and the interface is more stable while being a little less accurate. The model
 192 can then describe high interface distortions. Practically, the parameters and the number of
 193 internal iterations are adjusted to limit the interface fractioning while keeping an accurate
 194 description of the interface. The reader is referred to Pianet et al. [49] for a detailed
 195 description of the algorithm and a comparison of its efficiency with other classical interface
 196 tracking methods.

197 2.2. Numerical experiment

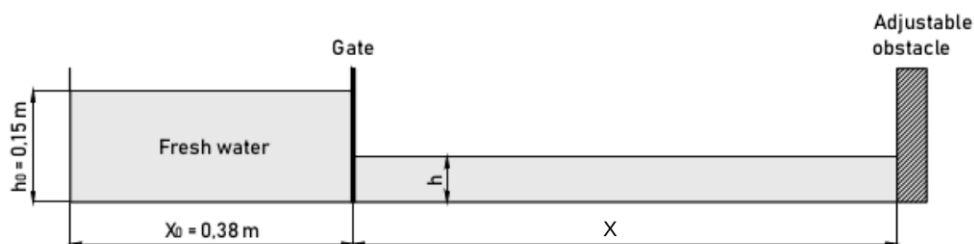


Figure 1: Sketch of the set-up studied and the associated parameters

198 In the wet dam breaking problem with an obstacle downstream (Figure 1, top panel),
 199 the problem parameters are : the upstream and downstream depths, respectively h_0 and h ,
 200 the length of the upstream tank X_0 and the position of the obstacle X with respect to the
 201 gate position.

202 In our study, we considered the same general set-up as in Jánosi et al. [28] to benefit
 203 from their experimental results. h_0 is fixed to 0.15 m. Two cases were studied in Jánosi
 204 et al. [28], involving downstream water level h respectively equal to 0.018 and 0.038m.
 205 These two cases are used here as reference to validate the model without obstacle. For
 206 these two cases, the lateral downstream limit of the model is fixed at 2m from the gate.
 207 The mesh is irregular. In the horizontal direction, the grid step is exponentially decreasing
 208 from $x=0$ to $x=0.38$ m (175 cells) down to 0.001m, equal to 0.001m from $x=0.38$ m to
 209 $x=0.78$ m (200 cells), and exponentially increasing, starting from 0.001m after $x=0.78$ m
 210 (325 cells). In the vertical direction, the grid step is constant and equal to 0.0015m (200
 211 cells between $y=0$ and $y=0.3$ m). Free slip conditions are set to each boundary.

212 As already mentioned in Crespo et al. [14], and because during the experiment, the gate
 213 velocity was of the same order as the flow horizontal velocity, it was necessary to take into

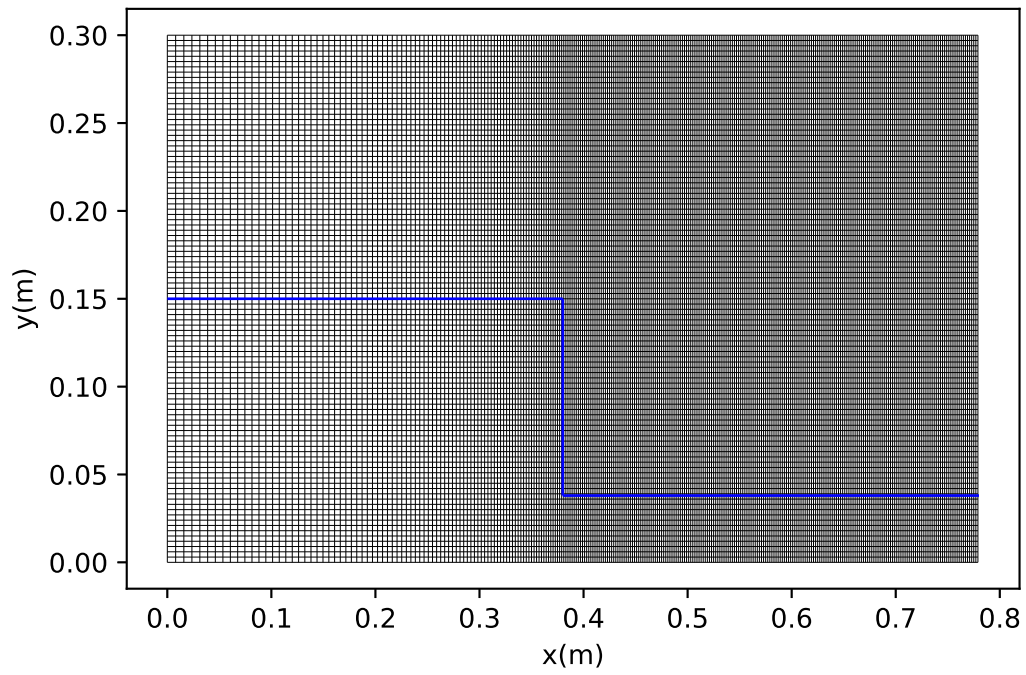


Figure 2: Representation of the mesh used for the numerical experiment here with $X = 0.40$ m, $h = 0.038$ m and position of water at $t=0$ in blue. Only one point out of four are plotted for the sake of readability.

214 account the kinematic of the dam gate to properly reproduce the experimental results. We
215 tested a constant uplift velocity as well as a constantly accelerated uplift motion. The
216 motion of the gate was stopped before it reached the free surface in order not to generate
217 unwanted effects. By doing so, we avoided the water to be dragged with the gate and the
218 generation of water droplets. For this reason, small rectangles can be observed in the next
219 figures relatively to the type of impacts.

220 For the numerical experiment, most of the simulations were carried out for values of h
221 between 0.018m and 0.038m (i.e., between the two values of the reference case for which
222 the model was compared with the experiment) with a step for h of 1 mm. The numerical
223 setting is the same as described previously, except that the downstream limit is fixed by
224 the value of X . The horizontal mesh step is constant and equal to 1 mm after the gate
225 (Figure 2) . For each value of the downstream depth h , cases were simulated starting from
226 $X = 0.12$ m, with a step of 0.01m, until reaching the first entrapped air cases. Additional
227 simulations were then carried out for values of h superior to 0.038 m in order to see if
228 it was possible to reach higher pressures and peak pressures. As peak pressures started
229 decreasing after $h = 0.045$ m, no further cases were simulated. Indeed, it is supposed
230 that, with decreasing differences between h and h_0 , the breaking point is more and more
231 delayed, up to the point where it does not occur anymore. Therefore, the impact pressure is
232 expected to decrease progressively as observed. On the lower end of the range, $h = 0.018$
233 m generates a breaking almost immediately after the gate release and the maximum impact
234 pressure (i.e., varying X) was lower than for larger h . It was therefore considered as our
235 lowest value. Conversely h_0 is not considered variable to keep the model close to the
236 experimental set-up [28], to which it has been compared.

237 Overall, about 500 simulations were carried out. For each simulation, the maximum
238 pressure and the position of this maximum in the vertical (i.e., z -axis) were calculated and
239 recorded at each time step, one cell before the rightward domain boundary, which played
240 the role of the obstacle. Additionally, to investigate the role of the interface shape before
241 impact, a snapshot of the free surface was taken at this stage, providing the shape of the
242 wave face and the associated local velocities (u the horizontal velocity and v the vertical
243 velocity). This snapshot was taken as soon as the free surface increased for more than
244 20% of the initial water level, again one cell before the wall position. The magnitude of the
245 velocity is then calculated from u and v and averaged over the free surface in a region close
246 to the impact to get a characteristic impact velocity called V_{imp} . The other characteristic
247 quantities are : the long wave celerity $c = \sqrt{gh}$ and a reference time $t_0 = h/c$.

248 **3. Results**

249 *3.1. Validation of the model*

250 The results of the numerical simulations have been compared with the snapshots ob-
251 tained by Jánosi et al. [28](figure 3). As no pressure measurement was performed in the
252 former reference, only wave shapes are compared. Nevertheless, according to Martin-
253 Medina et al. [42], which used the same model THETIS, the correct reproduction of the
254 shape and temporal evolution of the free surface guarantees similar impact pressure signals
255 to be obtained. This is also a conclusion of the work performed in Mokrani and Abadie
256 [45] using the same model.

257 Two cases were simulated with the downstream depth equal first to 18 mm and then
258 to 38 mm. Experimental free surfaces have first been digitized in order to compare with
259 the numerical simulation results. As our interest is focused on the wave just before it
260 breaks, only the first four snapshots of each case were considered and digitized. The
261 quality of the snapshots being quite low, it may be a source of imprecision in the following
262 comparisons. This imprecision is of approximately 2mm when considering the height h
263 of the downstream free surface on both pictures. These 2mm are responsible for a scale
264 imprecision of more than 10% in the 18 mm case and 5% in the 38 mm case, which will
265 affect the considered wave height and position of the wave front at a given time.

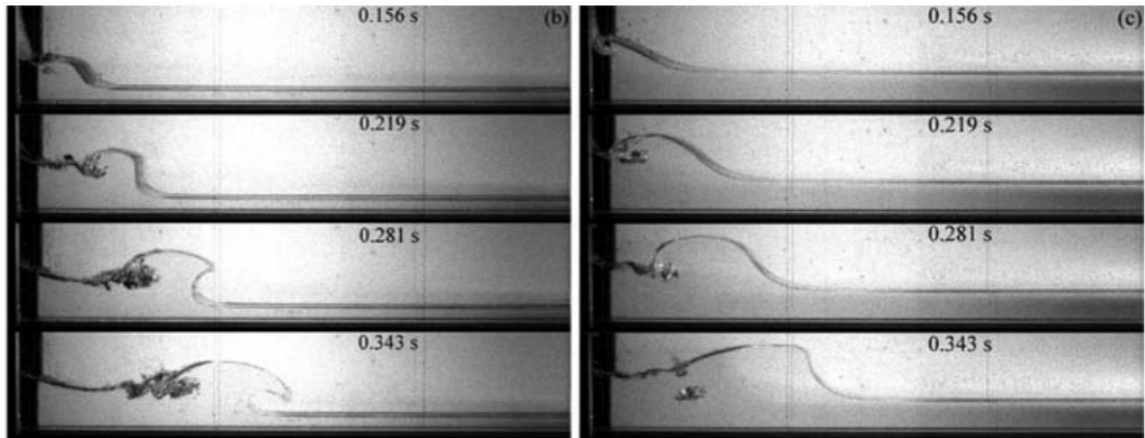


Figure 3: Snapshots from Jánosi et al. [28] with upstream water depth $h_0=15$ cm and downstream water depth, $h=18$ mm (left panel) and $h=38$ mm (right panel)

266 As can be seen on Figure 4, for a downstream depth of 18 mm, the model offers a good
267 representation of the position of the wave front and height between 0.156 s and 0.343
268 s. One of the peculiarities of the wave, as generated in the experiment, is the leftwards
269 breaking that can be observed in the two first snapshots. This feature is well represented

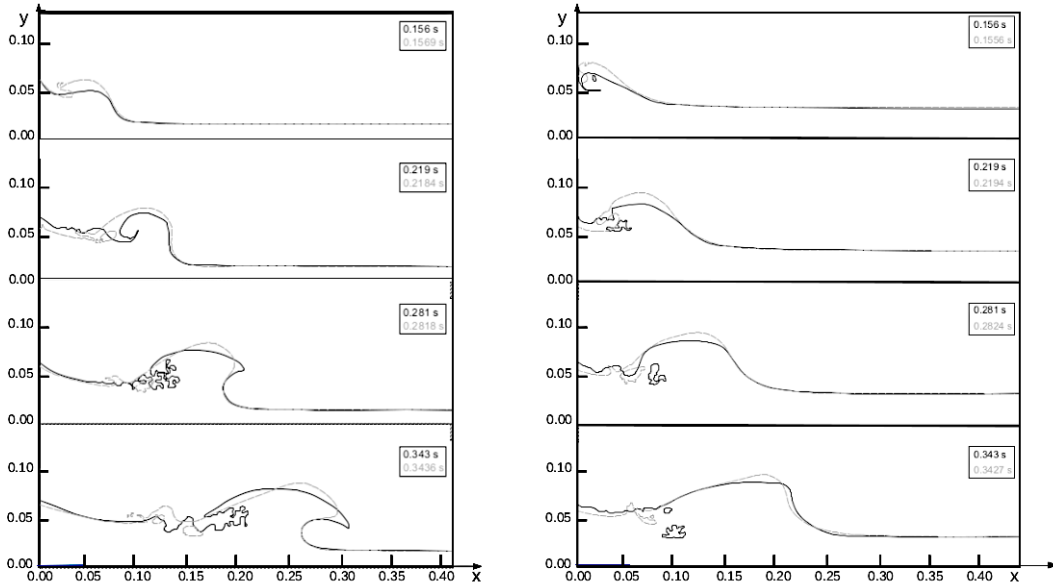


Figure 4: Comparison between experimental and numerical results for $h = 18\text{mm}$ (left) and $h = 38\text{mm}$ (right). Black curve : [28] grey curve : THETIS simulation.

270 by the numerical simulation. On the other hand, the model does not fit perfectly the wave
 271 breaking, especially the development of the breaking jet. Nevertheless, as the purpose of
 272 this work is focused on impacts without air entrapment, this late breaking process is not
 273 crucial. Nevertheless, it should be noted that the impact process being very sensitive to the
 274 slightest changes in wave geometry, the observable differences between experiment and
 275 simulation may generate non negligible differences on the peak pressures.

276 The results obtained for the 38 mm case are slightly better than those obtained for the
 277 18 mm case. One explanation could be the larger scale of the case that reduces the effects
 278 of the imprecision of the picture.

279 A comparison between the results obtained with a 1.5 m.s^{-1} constant velocity and with
 280 a uniformly accelerated gate (whose corresponding average velocity value is equal to 1.5
 281 m.s^{-1}) has been performed (figure not shown here). The progression of the wave happens
 282 faster for the uniformly accelerated gate movement. The shape of the wave is also flattened
 283 in comparison with the constant velocity case. Finally, as in Crespo et al. [14], the best
 284 results were obtained with a constant velocity of 1.5m.s^{-1} . This is the configuration used
 285 for the next numerical experiment.

286 A mesh study has also been conducted to see the effect of the resolution on the impact
 287 pressure estimated by the model (figure 5). The model set-up corresponds to the sketch

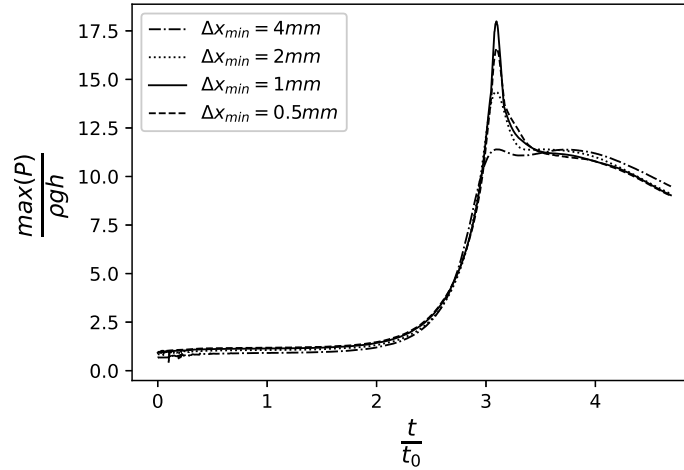


Figure 5: Maximum pressure on the wall for different mesh resolution, $h = 0.018m$ and $X = 0.1m$.

288 of figure 1 leading to a very quick impact on the obstacle due to the limited downstream
 289 depth. The variable represented is the time variation of the maximum pressure on the wall
 290 which is of interest in this paper. Figure 5 shows that the peak first increase up to a max-
 291 imum of about 17.5 with the resolution of 1mm and then decrease by 8% with the finest
 292 resolution of 0.5mm. The model convergence is therefore not perfectly achieved. This
 293 is not completely surprising owing to the combined difficulty of the gate/flow interaction
 294 with the inherent instability of the liquid impact problem. With the same model, Mokrani
 295 and Abadie [45] for instance showed the heavy resolution requirement to achieve conver-
 296 gence at the stagnation point in a dry dam break impact. Impact pressures are also shown
 297 to slightly vary with mesh resolution in the mesh-free simulations presented in Zuo et al.
 298 [59]. Methods used to track the interface are also susceptible to generates discrepancies
 299 in the peak pressure computed [34]. Furthermore, convergence is more easily achieved on
 300 the pressure impulse than on the pressure peak [5, 44]. Therefore, the choice of the 1mm
 301 resolution for the numerical experiment is a compromise between accuracy and CPU con-
 302 straints. Nevertheless, due to the aforementioned numerical limitations, the individual
 303 pressure peak values computed may not be completely accurate while the relative trend is
 304 expected to be more trustful.

305 3.2. Numerical experiment

306 Several types of impacts were obtained depending on the position of the obstacle (X)
 307 with respect with the breaking point location (itself depending on the water depth h). Two

308 processes happen simultaneously. The progression of the wave forces the wave trough to
 309 run-up quickly along the wall while the wave crest moves forward more slowly. The time
 310 history of these two phenomenons and the fact that they coincide or not is responsible for
 311 the type of impact generated on the obstacle. Hence, four types of impacts were observed
 312 : sloshing impacts (Figure 6), flip-through impacts (Figure 7), air pocket impacts (Figure
 313 8), broken wave aerated impacts (Figure 9). The flip through impact, which is the limiting
 314 case after which air starts to be entrapped, is characterized by the birth of a small vertical
 315 jet (not visible on the figure) associated to very large accelerations. As mentioned before,
 316 the presence of small rectangles in the figures is due to the fact that the motion of the gate
 317 is stopped before it interferes with the free surface.

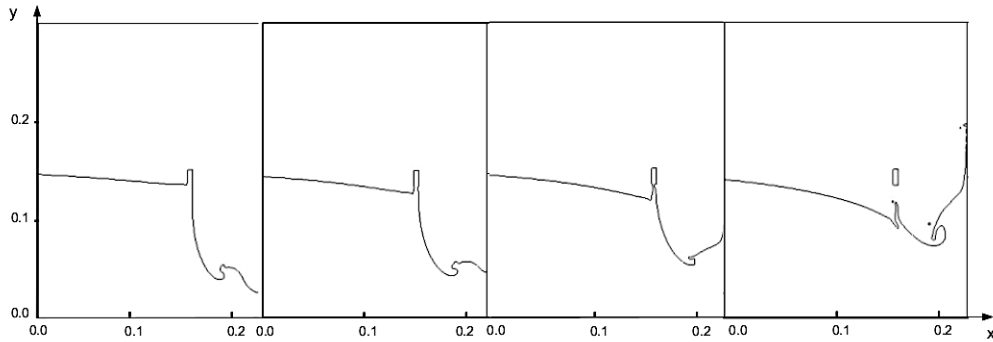


Figure 6: Sloshing impacts obtained for $h = 18\text{mm}$ and $X = 0.07\text{m}$ at $t = 0.12, 0.16, 0.25$ and 0.40s .

318 The two cases studied in János et al. [28] are described in terms of kinematic and
 319 dynamic of the impacts generated depending on the position of the obstacle (Figures 10
 320 and 11). The bottom panel shows the obstacle position for which, the first pressure peak
 321 appears on the wall. For instance for the case $h=18\text{mm}$, the critical position is $X=0.075$
 322 m. Before the latter, the largest pressure is on the bottom ($z_{p_{max}}=0$) as pressure is purely
 323 hydrostatic. After this limit, a larger pressure peak appears on the wall (i.e., no longer on
 324 the bottom part). Up to the limit case of the flip-through (marked by the ∇ symbol), we
 325 refer to these kinds of impacts as sloshing impacts (marked by the $+$ symbol). Right after
 326 the flip-through case, air entrapment starts to be observed. Even though the incompressible
 327 code is not supposed to represent properly the actual process when air entrapment
 328 occurs, results are plotted to give some information on the evolution of the parameters
 329 when passing through the flip-through limit. For the two downstream water heights stud-
 330 ied (i.e., 18mm and 38mm), and whatever the impact type, the pressure peak occurs at
 331 a very stable relative altitude $z_{p_{max}}/h \approx 2.5 - 2.75$. The value of $\frac{V_{imp}}{c}$ for the low water
 332 depth case, linearly increases from about 1.3 when X is the smallest to about 1.75 after the

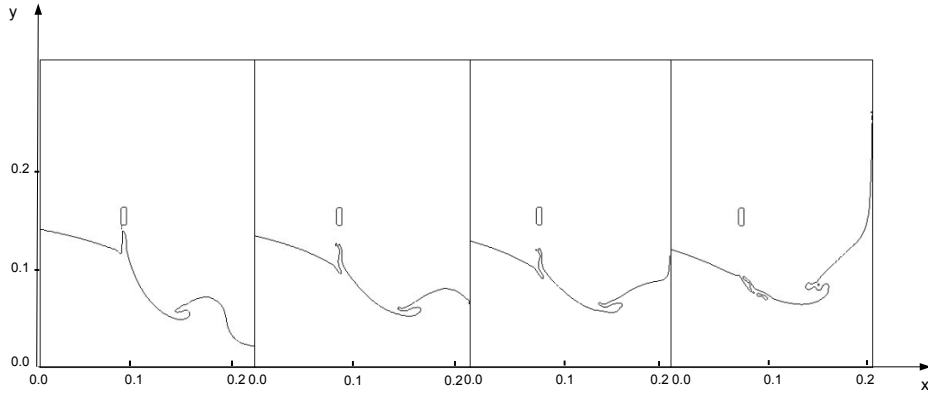


Figure 7: Flip-through impacts obtained for $h = 18\text{mm}$ and $X = 0.119\text{m}$ at $t = 0.22, 0.31, 0.38$ and 0.61s

333 flip-through position. In the larger depth case, it is very stable around 1. It is likely that the
 334 more distant obstacle, in the larger depth case, allows for a stabilization of the wave front
 335 velocity whereas in the low depth case, the position of the obstacle is closer to the gate and
 336 therefore the flow velocity is still rapidly evolving. As for the pressure peak, it is shown
 337 to gradually increase in absolute value as well as in relative value (i.e., scaled by dynamic
 338 pressure) in both cases as soon as the pressure maximum is no longer hydrostatic. Note
 339 that air entrapment does not drive any change in this evolution (the curve being perfectly
 340 continuous). The flip-through impact obtained for a downstream depth of 18mm generates
 341 a peak pressure of 6.185kPa for $X = 0.119\text{m}$ while the 38mm case gives rise to a 10.36kPa
 342 pressure peak when $X = 0.32\text{m}$. Interestingly, similar peak values are obtained when the
 343 pressure is scaled by the initial hydrostatic pressure (35 against 27.7 for $h=18$ and 38 mm
 344 respectively) whereas a larger difference is obtained with dynamic pressure as reference
 345 (12.5 against 28).

346 Figure 12 shows the evolution of pressure at z_{pmax} with time for simulations involving
 347 different values of X . These simulations can be separated into two groups. The first one,
 348 represented with black dotted lines, corresponds to cases where the pressure maximum is
 349 reached at the bottom of the water (i.e., between $X = 0.02\text{m}$ and $X = 0.074\text{m}$ for $h = 18\text{mm}$
 350 and between $X = 0.12\text{m}$ and 0.22m for $h = 38\text{mm}$). In those cases, the time evolution of
 351 pressure is relatively slow and the pressure maximum is equal to the hydrostatic pressure
 352 value corresponding to the wave run-up on the wall. The second group, plotted with

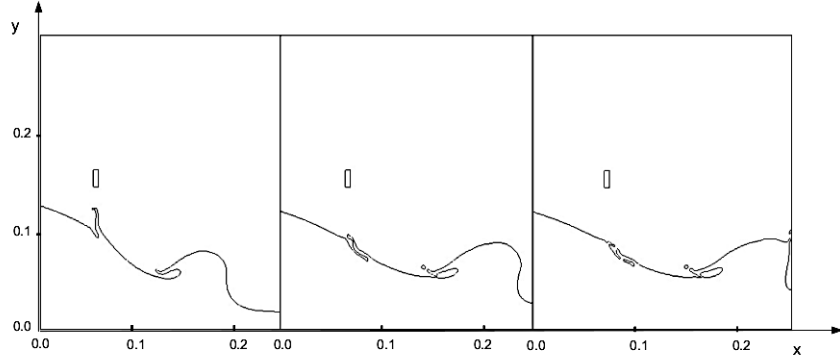


Figure 8: Impacts with entrapped air obtained for $h = 18mm$ and $X = 0.17m$ at $t = 0.28, 0.365$ and $0.435s$

353 continuous lines, corresponds to sloshing impacts. They are obtained for positions of
 354 the right wall between $X = 0.075m$ and $X = 0.118m$ for $h = 18mm$ and between $X =$
 355 $0.23m$ and $0.32m$ for $h = 38mm$. Here, the maximum of pressure is not reached at the
 356 bottom of the water but at a higher point on the obstacle. The pressure peaks are short
 357 (approximately $0.01s$) and their amplitude gets bigger and bigger as X increases. In the
 358 two cases plotted, the highest peaks are obtained for the flip-through impacts represented
 359 with red lines (further cases involving air entrapment are not considered in that plot).

360 The results obtained for all the simulations performed in this study are summarized in
 361 contour plots (figures 13, 14, 15, 16 and 17).

362

363 Figure 13 shows the distribution of the maximum pressure in the $X - h$ domain. Two
 364 grey lines approximately discriminates the three observed regimes (i.e., hydrostatic, slosh-
 365 ing and aerated). In the hydrostatic area, the pressure maximum is generated at the bottom
 366 by the water local elevation and its value is quite uniform in this domain. The gradient
 367 is much larger in the sloshing regime where an actual peak is observed in the pressure
 368 time signal. The flip-through occurs around the second grey line, crossing higher pressure
 369 contours, and which equation is : $\frac{h}{h_0} = 0.103\frac{X}{h_0} + 0.032$. The area corresponding to aerated
 370 impacts (broken waves) or impacts with entrapped air (below the grey line representing
 371 the air entrapment limit) are the ones that engender the highest pressures in this study, but
 372 these cases have to be considered with caution as compressibility is not considered in the
 373 model.

374 In Figure 14, the maximum pressure is divided by ρgh . In the hydrostatic area, the
 375 contours give an idea of the run-up generated by the impact on the vertical wall. In the
 376 sloshing area, the maximum value (i.e., about 35) of this non-dimensionalized pressure is
 377 obtained for the smaller scale case with $h = 0.018m$ and $X = 0.119m$. Along the limit of

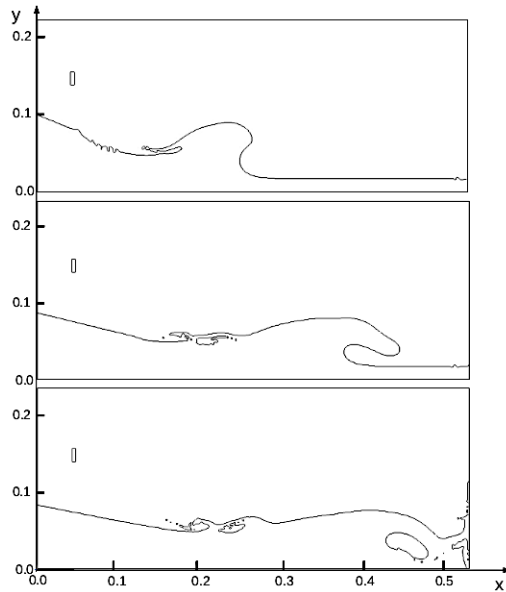


Figure 9: Impact obtained after the wave has totally broken for $h = 18\text{mm}$ and $X = 0.47\text{m}$ at $t = 0.3639$ and 0.3861s

378 air entrapment (i.e., flip-through cases), $P/\rho gh$ then decreases gradually with increasing h .

379 The maximum impact velocities V_{imp} , larger than $1.68\sqrt{gh}$, appears for X between
 380 0.12 and 0.16 m and h lower than 0.19 m (Figure 15), namely for relatively small values
 381 of h and X). The relative impact velocity then decreases concentrically and regularly with
 382 increasing X and h . For a given h and starting from the lowest X value, the impact velocity
 383 first increases toward a maximum value. After reaching its maximum, for cases with
 384 $h < 0.037$, the impact velocity then decreases when the obstacle is moved further away
 385 and air starts to be entrapped. For larger h , the impact velocity is more or less constant after
 386 the first observed increase. Nevertheless, for all the cases, the impact velocity decreases
 387 beyond the air entrapment limit.

388 When scaled relatively to the dynamic pressure, the non-dimensional pressure is the
 389 highest for the cases involving large values of X and h (Figure 16), contrasting to the
 390 behaviour observed in Figure 14. Between the two grey lines, the flip-through impacts
 391 generate the highest pressures, with maximum relative pressure starting from slightly less
 392 than $15\rho V_{imp}^2$ (bottom part of the grey line delineating air entrapment) to approximately
 393 $30\rho V_{imp}^2$ (top part of the same grey line). For a given h , peak pressures then decrease
 394 quite linearly as sloshing impacts are considered (i.e., for X decreasing). On the left of
 395 the hydrostatic limit (top grey line), the relative pressure is almost constant (around 6)
 396 reflecting the weak spatial variation of V_{imp} and P in this area.

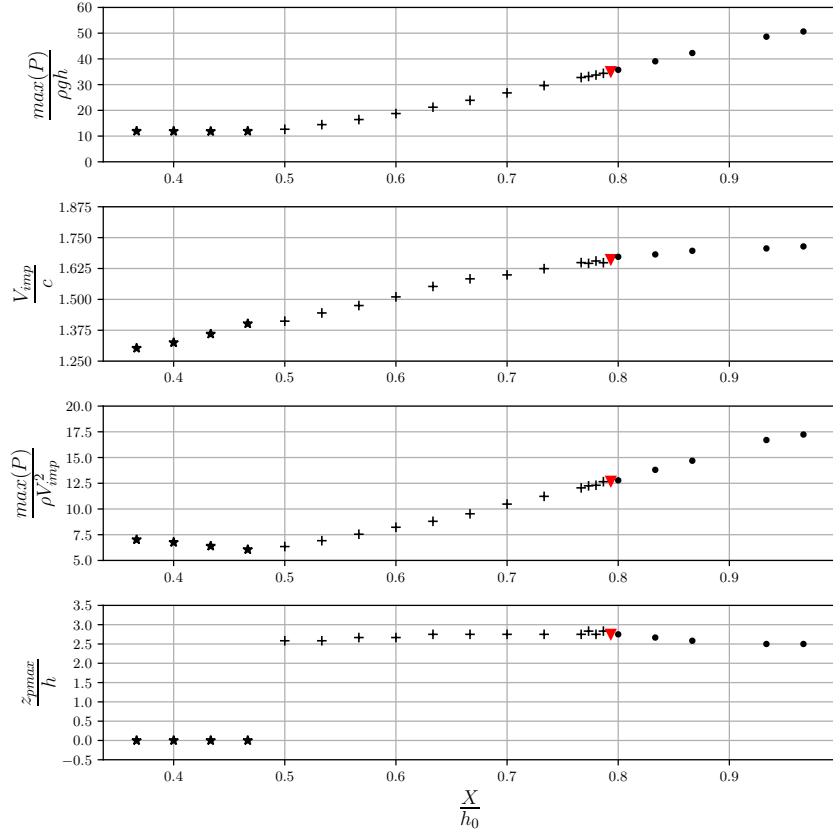


Figure 10: From top to bottom panel : maximum pressure recorded on the wall scaled by hydrostatic pressure, non dimensional impact velocity, maximum pressure recorded on the wall scaled by dynamic pressure and non dimensional height of pressure maximum versus X/h_0 for $h = 18mm$. Symbols used : * for hydrostatic process (no pressure peak), + for sloshing, ∇ for flip-through and \bullet for air entrapment.

397 Figure 17 shows the heights where the impact peak pressures are generated. Peak pres-
 398 sures appear at heights between 2 and 3 times the water depth h for the sloshing impacts,
 399 the largest values being associated to low water depths and the smallest to high water
 400 depths. Close to the flip-through impact limit, the values of z_{pmax}/h are in the range of
 401 2.3 - 2.8 and start decreasing when getting closer to the grey line. Therefore, the height of
 402 maximum pressure seems to be a good indicator of the parameters for which a flip-through
 403 impact occurs.

404 Finally, accelerations of the free surface at the wall has been calculated for several
 405 cases along the grey line approaching the flip-through limit (Figure 18). To perform this

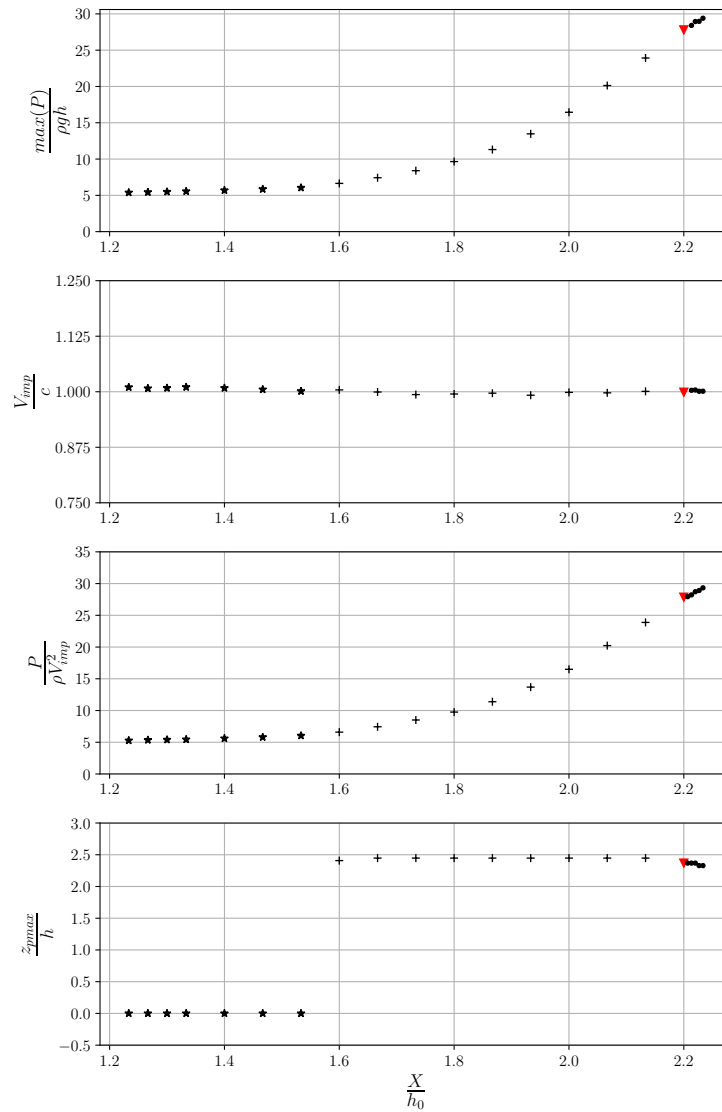


Figure 11: Same legend as previous figure with $h = 38mm$

406 computation, the free surface elevation next to the wall is first recorded and an exponential
 407 law fitted up to the visible inflection point after which acceleration decreases (i.e., black
 408 curves on Figure 18 left). The exponential is then analytically derived twice to retrieve the
 409 acceleration value (Figure 18 right). The maximum accelerations found with this method
 410 are between 76 and 142g, the largest value approximately corresponding to the area (e.g.,

411 Figure 14) where the pressure peak is the largest.

412 **4. Discussion**

413 In this work, we numerically studied wave impacts generated in the wet dam break
414 flow. Based on two parameters, about five hundred distinct cases were obtained and re-
415 sulted in different peak pressures on the obstacle. All the impacts can be grouped in four
416 types : sloshing impacts, flip-through impacts, air pocket impacts and broken waves im-
417 pacts.

418 These four types of impacts have already been described in the literature before but
419 their modes of generation were different and generally required a longer time span from
420 wave generation to impact. This longer duration adds to the inherent difficulty of the
421 phenomenon, for instance in terms of repeatability. Brosset et al. [10] stresses the problem
422 of repeatability when trying to reproduce a flip-through impact with wave focusing. In this
423 study, the flip-through was obtained only once among 140 tests and for this, preliminary
424 tests were first performed on a smaller tank to have a better control over the repeatability
425 of the flow. In Hofland et al. [23], also based on wave focusing, among the 137 measured
426 impacts, only 8 were identified as actual flip-through impacts. As underlined in this article,
427 for a same configuration (same depth and focal distance) only approximately one test out
428 of 10 gave rise to a flip-through impact. Sloshing in a prismatic tank has also be used
429 to generate violent impacts (e.g., [39]) and the problem of repeatability arises generally
430 in that case too. Impacts generated by wave shoaling in a flume (e.g., [31]) can also be
431 affected by a lack of repeatability due to the poor repeatability of the wave maker itself and
432 the difficulty to control the water depth in the wave flume. A long generation mode also
433 leads to very large data set due to the very high sampling rate required to capture the impact
434 phenomenon. It is also a supplementary difficulty for numerical simulation. Indeed, in
435 order to predict the proper impact dynamic, one has to reproduce very accurately the wave
436 shape just before impact. This implies perfectly solving the generation and propagation
437 stage, which may already be a challenge for the models. This usually requires heavy model
438 coupling techniques to be employed [53]. In the present paper, wave impact is generated
439 very quickly after removing the reservoir gate. The wave is created instantaneously by the
440 interaction of the water volume released with the downstream fluid. It is a very simple
441 configuration which can be easily used in any laboratory. Note that repeatability has still
442 to be checked experimentally. For this, the gate lift control will probably be of the utmost
443 importance.

444 In figure 12, in both reference cases, a very sharp and high peak of pressure was ob-
445 served close to the air entrapment limit. This shape, often referred to as a "church roof
446 shape" [48], is a consequence of the large vertical acceleration due to the presence of the

447 obstacle. In the critical flip-through cases (e.g., Figure 7), this acceleration reaches a max-
 448 imum due to the existence of a more or less ideal focal point toward which, a large amount
 449 of the fluid particles converges [39]. In the present study, the vertical accelerations of the
 450 free surface close to the obstacle are approximately in the range 50 – 150g (Figure 18).
 451 These large values are nevertheless lower than the extreme values usually reported in the
 452 literature (see section 1) likely due to the reduced dimension of the set-up studied. The
 453 maximum pressures obtained with this dam break configuration are of the order of 10kPa
 454 and relatively to the hydrostatic pressure, of the order of 30. This is clearly in the range
 455 10-100 ρgh associated to the strongest impacts reported in the literature [39] and one order
 456 of magnitude larger than the values so far reported in the impacts generated in dam break
 457 flows. Along the grey line delineating sloshing and aerated impacts (Figure 13), given by
 458 the equation $\frac{h}{h_0} = 0.103 \frac{X}{h_0} + 0.032$, starting from the lower water depths, the peak pressure
 459 first increases to reach a maximum around $h \approx 0.036$ (or $h/h_0 \approx 0.24$) and then decreases.
 460 The existence of this maximum was expected. Indeed, one may expect the flow to reach
 461 the ideal condition to generate the highest pressure peak between the very low downstream
 462 water depths, for which the wave shape may not be optimal and the high downstream wa-
 463 ter depths, which may attenuate the wave dynamic and even prevent its breaking for the
 464 largest values. Therefore, critical impacts (i.e., flip-through) exists for each h but there are
 465 some slight variations among them, dependent on the wave shape and the local velocity
 466 field generated.

467 When scaled relatively to the dynamic pressure, the maximal values are again around
 468 30 (Figure 16), which also corresponds to large peak pressure values reported in the liq-
 469 uid impact literature [57]. Note that the variable $\text{Max}(P)/\rho V_{imp}^2$ is more relevant than
 470 $\text{Max}(P)/\rho gh$ to characterize the efficiency of the impact process to transform the wave
 471 momentum into a pressure peak. As mentioned in Mokrani and Abadie [45] and Martin-
 472 Medina et al. [42], this efficiency strongly depends on the flow horizontal velocity, hence
 473 the introduction of V_{imp} , but also, and to a large extent, to the water-air interface local
 474 angle. Figure 16 shows that this efficiency is maximum approximately where pressure is
 475 also maximum.

476 In the present work, the pressure peaks associated to sloshing impacts are located sig-
 477 nificantly above the water height h . For the flip-through impacts, this height is equal to
 478 $2.3h$ to $2.8h$. This result stresses the difference between regular (or irregular) wave impact
 479 and the present case. In the former cases, the free surface involves a trough and a crest
 480 and the pressure peak, generated by the breaking wave impact, is, as already pointed out
 481 in the introduction, usually located near the average water level [19]. In the present study,
 482 the process significantly differs due to the water volume added in the domain after $t = 0$
 483 and therefore, results are not comparable. Similarly, the waves dynamics also differ. A
 484 regular wave is free and the breaking is generated most of the time by the water depth

485 reduction. In the present case, the wave is forced by the pressure gradient due to the dam
486 break. Nevertheless, to that respect, Figure 15 shows that, except for very small h values,
487 the flow velocity associated to the breaking wave is close to the free shallow water wave
488 celerity, which is what may be expected for a free wave breaking.

489 The main limitation of this study is the purely numerical approach proposed. Regard-
490 ing this concern, the model has been validated in former studies related to wave impacts
491 [45, 42]. From these studies, it is known that the pressure field accuracy depends on the
492 velocity and local interface proper resolution at impact. Here, even if this model has been
493 carefully compared to experimental results [28], uncertainties remain. The gate velocity
494 law may not be exactly the same as in the experiment. It is not possible to verify this point
495 as no reference was made in the associated article on that aspect. There is also a slight shift
496 between the experimental [28] and simulated free surfaces. As the pictures quality is quite
497 low, it is hard to know which part to attribute to the digitization process or to the actual
498 weaknesses of the model. We also showed that the pressure peak is particularly sensitive
499 to the mesh resolution (Figure 5).

500 Finally, the choice of an incompressible model to study water wave impacts may be
501 argued. It is indeed well known that compressible effects plays a role in this phenomenon,
502 especially when air is entrapped. In this case, the air pocket may oscillate and this trans-
503 lates to the pressure on the wall [8]. Additionally, the presence of air bubbles in water
504 may lead to water compressibility allowing the generation of pressure waves [50]. Those
505 effects are susceptible to really appear if the wave impact occurs after the breaking point.
506 If this is not the case, then the flow stays mainly incompressible during the impact pro-
507 cess. Very large pressure can anyway be generated [56]. From a numerical point of view,
508 it is difficult to study incompressible flows with a compressible model and vice-versa. The
509 point of view defended in this paper is to concentrate first on the incompressible part of
510 the problem before focusing later on the compressible aspects. Therefore, the results given
511 after the flip-through limit, where air is supposed to be entrapped, should obviously be in-
512 terpreted with the highest caution. Those limitations given, the numerical results should
513 not depart too much from the real phenomenon and provide a valuable guidance for a
514 further and necessary experimental confirmation.

515 **5. Conclusions**

516 In this paper, a Navier-Stokes VOF model was used to study the wave impacts gener-
517 ated in a wet dam break flow when varying the position of the downstream wall and water
518 depth. The following conclusions can be drawn from this work :

- 519 • first, to reproduce accurately the shape of the wave observed in our experimental
520 reference [28], a gate motion had to be taken into account in the simulations,

- 521 • in the range of parameters tested, for a downstream water depth h and increasing
522 progressively the impacted wall position, the following phenomena are observed :
523 hydrostatic run-up, sloshing impacts, flip-trough impacts, air-pocket impacts and
524 broken wave interaction with the wall,
- 525 • flip-trough impacts are defined as the limit case of the sloshing impacts just before
526 air entrapment could be observed. For the wet dam break flow, we show that this
527 condition is met for $\frac{h}{h_0} \approx 0.103 \frac{X}{h_0} + 0.032$. Along this line, the free surface vertical
528 acceleration is in the range 50 to 150g. We also showed that among all the possible
529 flip-through impacts, there is one optimum generating a pressure peak larger than the
530 others,
- 531 • compared to impact pressures recorded or simulated with a dam break flow in previ-
532 ous studies (except the recent work of Zuo et al. [59] already discussed), the present
533 numerical work reached values one order of magnitude larger (i.e. $P_{max} \approx 35\rho gh$ or
534 $P_{max} \approx 30\rho V_{imp}^2$). Those values are in line with usual extreme pressures reported in
535 wave impacts [39],

536 Therefore, wet dam break flow may be used to quickly generate a given wave impact ad-
537 justing the parameter of the experiment. Nevertheless, as this study is purely numerical,
538 an experimental confirmation would be the logical perspective of this work. The future
539 experiments should focus on measurements repeatability, as slight changes in the gate dy-
540 namics could modify the subsequent flow enough to make the pressure peaks unrepeatable.
541 Additionally, by modifying the equilibrium between the different forces at stake, the ex-
542 periment scale could also play a role on the shape of the wave for a given set of parameters
543 and completely change the previous conclusions. Finally, the compressible impacts when
544 air is entrapped have still to be tackled numerically. This is also a perspective of the present
545 work.

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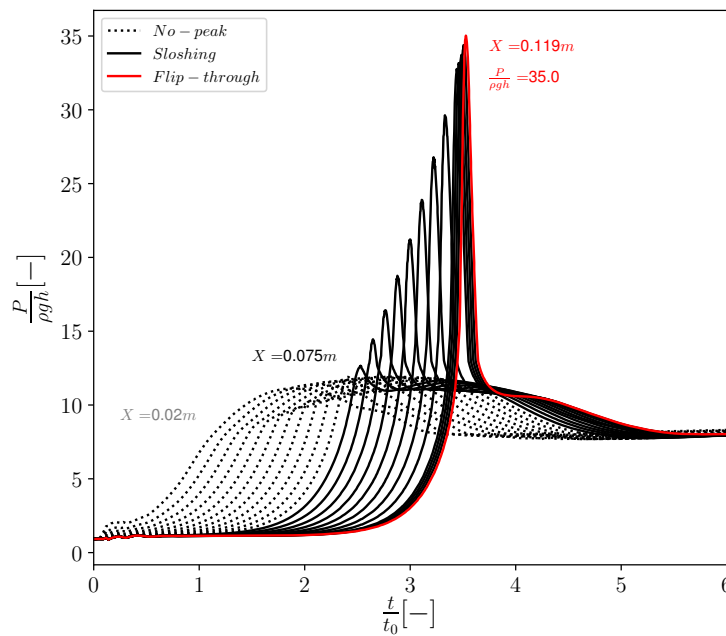
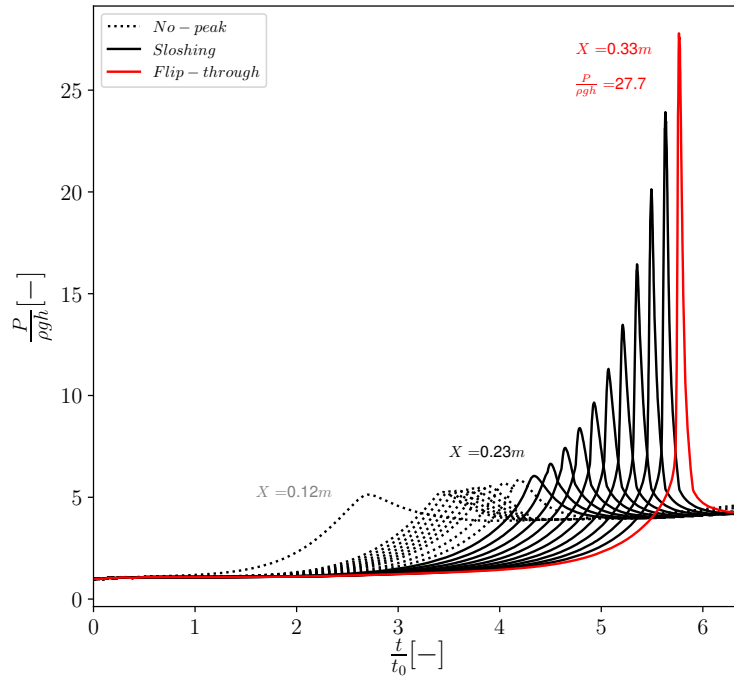


Figure 12: Evolution of the normalized pressure with relative time t/t_0 with $t_0 = h/c$, at elevation z_{pmax} for the reference cases $h = 18mm$ and $38mm$ (respective peak pressures are $6.185kPa$ and $10.36kPa$), gradually moving the obstacle to the right (i.e., varying X).

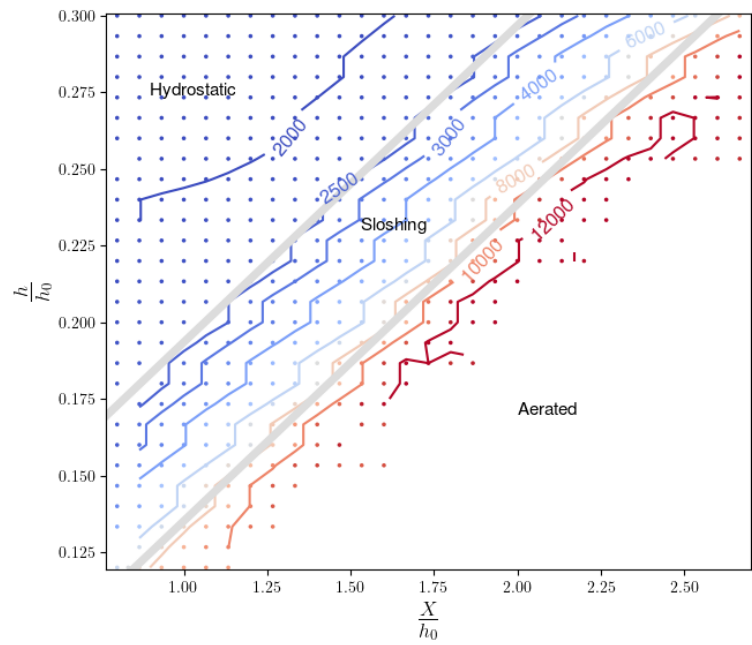


Figure 13: Contours of $\text{Max}(P)$ in the scaled $X-h$ domain considering all the cases simulated

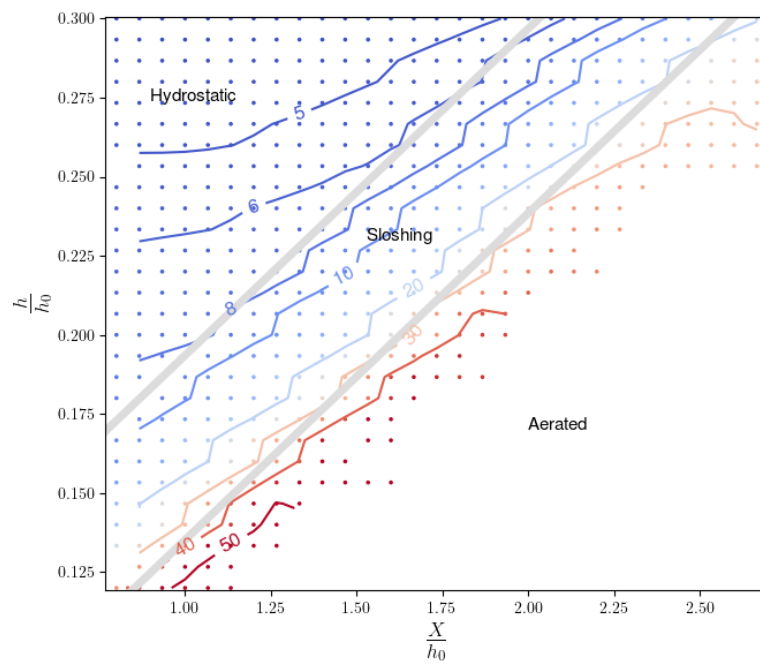


Figure 14: Contours of $\text{Max}(P)/\rho gh$ in the scaled $X - h$ domain

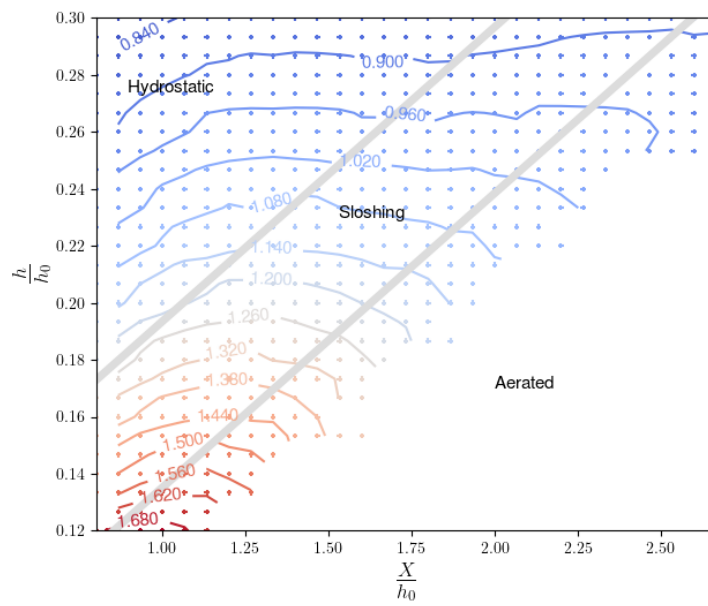


Figure 15: Contours of V_{imp}/\sqrt{gh} in the scaled $X - h$ domain

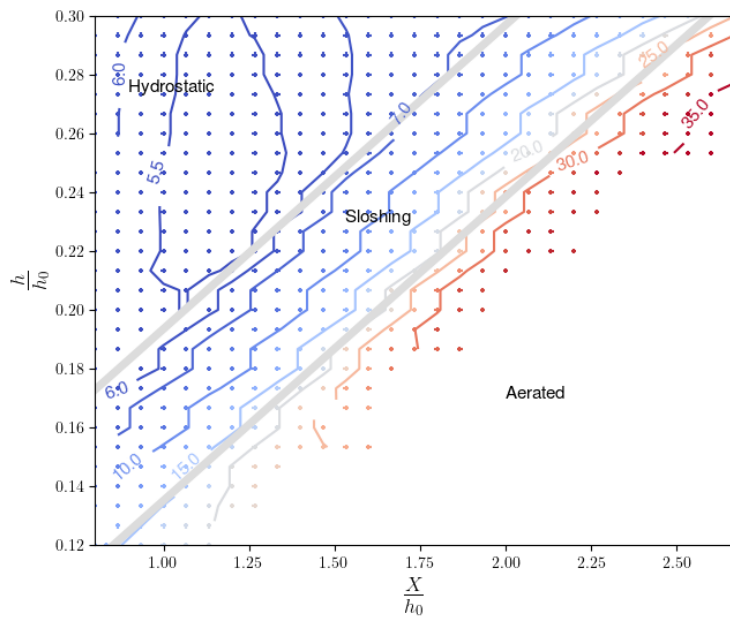


Figure 16: Contours of $\text{Max}(P)/\rho V_{imp}^2$ in the scaled $X - h$ domain

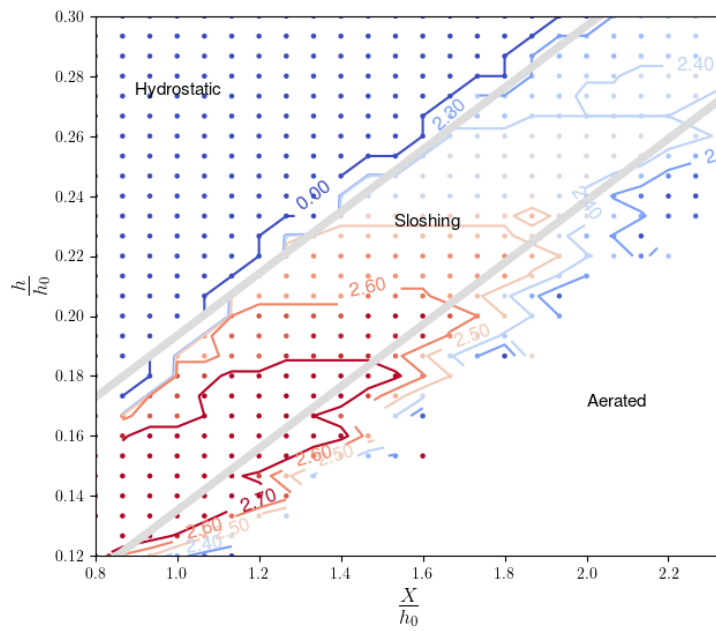


Figure 17: Contours of $z_{P_{max}}/h$ in the scaled $X - h$ domain

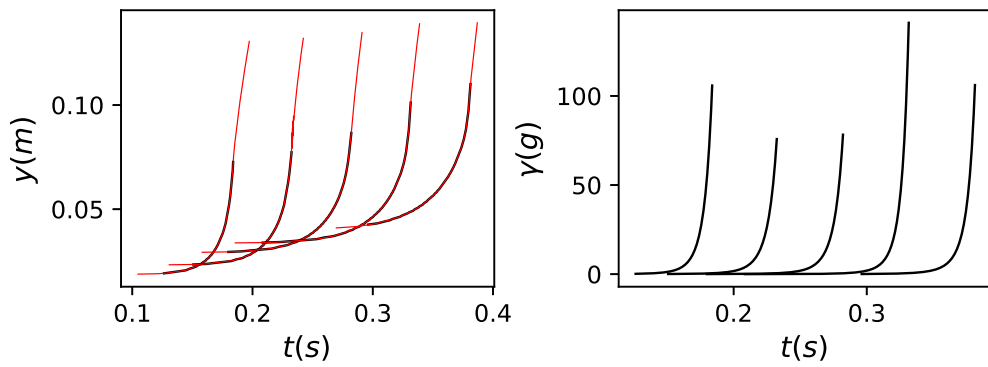


Figure 18: Left panel : free surface time evolution (red) next to the wall during the impact for $X/h_0 = 1$ (first curve on the left and so on), 1.33, 1.66, 2, 2.33 and $h/h_0 = 0.135, 0.17, 0.205, 0.24, 0.275$ respectively. Right panel : corresponding accelerations in g calculated by fitting an exponential law on the black curves shown in the left panel and deriving twice.