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EXPERIMENTAL STUDY OF THE EARLY STAGES OF A DAM-BREAK FLOW OVER FIXED AND MOBILE BEDS

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Thesis presented for the degree of Doctor in Engineering Sciences

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To my mother,
who gave me wings.

To my father
who taught me how to read a compass.

To my *Pequenita*,
shelter of my soul.

*All life is an experiment.
The more experiments you make the better.*
Ralph Waldo Emerson

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*If you want to travel fast, go alone;
if you want to go far, go together
African proverb*

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List of Symbols

Roman Characters

- c : wave celerity $c = \sqrt{gh}$.
- c_0 : dam-break wave celerity $c_0 = \sqrt{gh_0}$.
- c_s : sound speed in water.
- c_{sM} : sound speed in a specific medium M.
- C_1 : positive characteristic curve.
- C_2 : negative characteristic curve.
- C_{fw} : Friction coefficient between the water and sediment layers.
- C_{fs} : Friction coefficient between the sediment layer and fixed bed.
- C_b : fixed bed concentration.
- C_s : sediment layer concentration.
- dt : time interval between two consecutive PIV images.
- $d\mathbf{x}$: Correlation peak displacement vector.
- e_b : bed erosion/deposition rate.
- e_s : sediment layer erosion/deposition rate.

f	: lens diaphragm aperture or measured number of events per bin.
f_c	: cut frequency.
f_D	: Doppler frequency.
$f_{D\max}$	: maximum measurable Doppler frequency.
f_i	: observed frequency in bin i .
f_o	: UVP transducer emitting frequency.
F	: Expected number of events per bin
F_i	: expected frequency in bin i .
F_{pr}	: Pulse repetition frequency.
g	: gravity acceleration.
h	: flow height.
h_0	: initial water height in the dam's reservoir horizontal.
$h_{\max}$	: UVP maximum analysed depth.
h_s	: sediments layer height in the two-layer model.
h_u	: UVP control volume height.
h_w	: water layer height in the two-layer model.
$H_{\max}$	: UVP maximum analysed non-dimensional depth.
$I1, I2$	: two consecutive PIV images.
k_s	: bed rugosity.
K	: number of bins.

l_i	: distance between Voronoï stars' extremities.
m	: ratio between non-dimensional coordinate and non dimensional time, $m = X/T$.
n	: number of runs of a dam-break made degrees of freedom4.
n_s	: number of cycles in a sound burst.
N_r	: number of repetitions.
p	: number of successful runs.
P_n	: detected particle n .
Q	: don-dimensional unitary flow rate.
$\mathbf{r}$	: position vector.
R_{uw}	: correlation coefficient.
Re_{δ^*}	: displacement thickness Reynolds number, $\text{Re}_{\delta^*} = U_o \delta^* / \nu$
Re_{Θ}	: momentum thickness Reynolds number, $\text{Re}_{\Theta} = U_o \Theta / \nu$
s	: ratio between sediment and water density $s = \rho_s / \rho_w$.
S	: search area.
t	: time coordinate.
t_0	: natural time-scale of the dam-break flow $t_0 = \sqrt{h_0/g}$.
t_m	: Mean time in the PTV measurements.
t_r	: removal time of the gate.
T	: non dimensional time coordinate $T = t/t_0$.
T_r	: non dimensional removal time of the gate $T_r = t_r/t_0$.

u	: horizontal component of velocity.
$\bar{u}$	: time-averaged value of the horizontal component of velocity.
u'	: fluctuation of the horizontal component of velocity.
$\hat{u}$	: depth-averaged velocity.
u_f	: horizontal component of the fall velocity of a seeding particle in water.
u_s	: sediment layer velocity in the two-layer model.
u_w	: water layer velocity in the two-layer model.
U	: non-dimensional horizontal component of velocity.
U_o	: Potential flow velocity.
U_f	: horizontal component of the non-dimensional fall velocity of a seeding particle in water $U_f = u_f/c_0$.
$\overline{u'w'}$	: velocity covariance.
$\mathbf{v}$	: velocity vector.
V_i	: area enclosed by a Voronoï Cell.
V_{max}	: UVP maximum measured velocity.
V_r	: Velocity range.
V_{1p}	: volume of cross correlation maximum peak
V_{2p}	: volume of the cross correlation second highest peak.
w	: vertical component of velocity.

w_f	: vertical component of the fall velocity of a seeding particle in water.
$\bar{w}$	: time-averaged value of the vertical component of velocity.
w'	: fluctuation of the vertical component of velocity.
W	: non-dimensional vertical component of velocity $W = w/c_0$.
W_f	: vertical component of the non-dimensional fall velocity of a seeding particle in water $W_f = w_f/c_0$.
x	: horizontal component.
x_0	: lagrangian particle initial horizontal coordinate.
X^*	: non-dimensional horizontal component, $X = x/(c_0 t)$
X_0	: non-dimensional horizontal gate coordinate, $X_0 = x_0/h_0$
z	: vertical coordinate, water height variable.
z_0	: lagrangian particle initial vertical coordinate.
z_b	: fixed bed level.
Z	: non dimensional water height $Z = z/h_0$.
Z_M	: acoustic impedance of a medium M .

Greek characters

α	: UVP incidence angle.
β	: resting angle.
δ	: boundary-layer height.
δ_z	: Uncertainty associated to the channel's bottom location.

Δt	: time averaging interval.
Δ_i	: Difference between the original time series and its Hamming filtered value.
η	: square root of the non dimensional water height elevation $\eta = \sqrt{z/h_0}$.
Θ_u	: UVP constant.
κ	: Von Kármán's constant.
λ	: sound wavelength.
μ_b	: average of binned variable.
ρ_b	: fixed bed layer density.
ρ_M	: Medium M density.
ρ_p	: seeding particle's density.
ρ'_s	: transport layer density.
ρ_w	: water density.
σ_{bj}	: standard deviation of binned variable j (u or w).
σ_u	: standard deviation of u .
σ_w	: standard deviation of w .
ϖ	: ratio between the height's difference and the initial height $\varpi = (z_0 - z_p)/h_0$.
τ_b	: bed shear stress.
τ_{ws}	: shear stress exerted by the water layer on the sediment layer.

Acronyms

ASCE	: American Society of Civil Engineers.
CFL	: Courant, Friedrichs and Lewy.
DNS	: Direct Numerical Simulation.
HLL	: Harten, Lax and von Leer.
LEMSC	: Laboratoire Essais Mécaniques, Structures et Génie Civil.
iMMC	: Institut of Mechanics, Materials and Civil Engineering.
MEMS	: Micro Electro Mechanical Systems.
PIV	: Particle Image Velocimetry.
PTV	: Particle Tracking Velocimetry.
RAM	: Random Access Memory.
SNR	: Signal-to-Noise ratio.
TR-PIV	: Time-Resolved PIV.
UVP	: Ultrasonic Velocity Profiler.

Abstract

The dam-break flow is an extreme example of a severe transient flow, often associated with catastrophic flood events. In a real case it is a very complex flow, involving not only many different time and space scales but also a complex fluid-bed interaction that will affect the propagating wave celerity. The need for studies on this kind of flow arises mainly from the risk assessment needed for existing, or to-be built, dams. The definition of the flooded areas and their evolution with time constitutes a valuable information for the authorities' evacuation plans conception.

Numerical models are often used to obtain the required information about the dam-break flow propagation. However for an improved modelling it is important to know the physics of the flow. This can be acquired by means of small scale models in the laboratory, where the effect of the different variables can be isolated and analysed. The information thus obtained can be used to gain a better comprehension of the dam-break flow, but also to obtain an experimental results data base that can be used to calibrate and validate existing numerical and analytical models.

The present study is devoted to the experimental analysis of the dam-break flow, with special emphasis on the initial stages of the dam-break flow. Rather than focus only in the macro-scale variables, i.e., water level evolution and wave celerity, the present study deals with the point-wise velocity measurements within the dam-break flow.

For velocity measurements different techniques are used. Two of them based on imaging techniques (PIV and PTV), and a third one based on the acoustic Doppler Effect (UVP). These techniques allowed for a space and time description of the dam-break flow velocity field. The measurements were carried considering different flow heights and bed types. The dam-break flow over beds of sediments is here analysed and the velocity field of both water layer and sediment layer measured simultaneously. A two-layer numerical model is then used and tested against the obtained experimental results.

The transient character of the dam-break flow motivated a statistical analysis of the dam-break flow. It consisted in a) repeating several times, for the same initial conditions, the dam-break experiments; b) obtaining images of the flow with an high acquisition rate (1000 Hz). An ensemble average procedure is presented and the results discussed.

For fixed smooth beds, the obtained spatial and time resolved velocity profiles made it possible, under some conditions, to determine the shear stress in the bed.

Finally the different measurement techniques are compared and discussed.

keywords: Dam-break flows. Sediment bed. Velocity field. Imaging techniques. Particle Image Velocimetry. Particle Tracking Velocimetry. Acoustic Doppler Velocimeter.

Resumo

O escoamento consecutivo a uma ruptura de barragem é um exemplo de um escoamento transiente severo, o qual é usualmente associado a eventos de inundações catastróficas. Em casos reais trata-se de um escoamento altamente complexo, envolvendo não só diferentes escalas espaço-temporais mas também uma complexa interação fluido-leito que condicionará a celeridade da onda de inundação. A necessidade de estudos sobre este tipo de escoamentos reside, entre outros, na correcta avaliação do risco colocado por barragens existentes ou a construir. A definição de zonas de inundação e a sua evolução com o tempo constitui uma informação importante para o estabelecimento, pelas autoridades, de planos de evacuação.

Os modelos numéricos são muitas vezes utilizados com esta finalidade. Contudo para uma melhor modelação numérica é importante conhecer a física do escoamento. Esta pode ser conhecida a partir de experiências realizadas em modelos à escala, em laboratório, onde as diferentes variáveis podem ser isoladas e analisada. A informação assim obtida pode ser usada não só para obter uma melhor compreensão do escoamento, mas também como uma base de dados útil para a calibração e validação de modelos numéricos.

O presente estudo tem como objectivo estudar experimentalmente o escoamento consecutivo a uma ruptura de barragens, com ênfase nos instantes iniciais. Sem se focar exclusivamente nas macrovariáveis, isto é, na evolução do nível de água e celeridade da onda, o presente estudo reporta medições pontuais de velocidade.

Para as medições de velocidade diferentes técnicas foram usadas. duas delas baseadas em aquisição de imagens (PIV e PTV) e uma terceira baseada no efeito Doppler acústico (UVP). Estas técnicas permitiram uma descrição temporal e espacial da evolução do escoamento de ruptura de barragem. As medições foram realizadas considerando diferentes alturas iniciais de água, assim como diferentes tipos de leito. O escoamento de ruptura de barragem sobre um leito de sedimentos móveis é analisado e os campos de velocidades na camada de água e na camada de sedimentos são medidos simultaneamente. Um modelo numérico de duas camadas é igualmente utilizado e os seus resultados comparados com os resultados experimentais.

O carácter transiente do escoamento de ruptura de barragem motivou uma análise estatística do mesmo. Esta consistiu em: a) na repetição da mesma experiência, nas mesmas condições; b) na medição do escoamento de ruptura de barragem com uma elevada frequência de aquisição (1000 Hz). Um método de médias de *ensemble* é aplicado e discutido.

Para o caso de leitos lisos fixos, os perfis de velocidade obtidos com suficiente resolução espacial e temporal permitiram, em certas condições, a determinação da tensão de arrastamento no leito.

Finalmente, as diferentes técnicas de medição são comparadas e discutidas.

Palavras-chave: Escoamentos de ruptura de barragem. Leito de sedimentos. Campo de velocidades. Técnicas de aquisição e processamento de image. Velocimetria por Imagens de Partículas. Velocimetria por Seguimento de Partículas. Velocimetria por Efeito Doppler Acústico.

Out of the trunk, the branches grow; out of them, the twigs.
So, in productive subjects, grow the chapters.
Melville (1851)

Chapter 1

Introduction

Let's start at the very beginning

A very good place to start

Rodgers and Hammerstein (1959)

1.1 Importance of Dam-Break Studies

Dams have been used by Man since ancient times to control the flow of river, supply water and since the late 19th century, produce hydro-electrical energy. In the present, due to the global warming problematic, dams are seen as an ecological way to produce electrical energy with low carbon dioxide emissions.

One question that arises when building a dam, usually associated with its environmental impact, is to know what will happen if the dam collapses and all the stored water is flushed downstream. This is a fact that has happened before in the past, either by natural causes, induced by Man or structural failure. A detailed review of dam break events is presented in Chanson (2006a).

In Figure 1.1 a first example of a dam-break event is illustrated. It is the collapse of the St. Francis Dam, in California, United States, which was built between 1924 and 1926, and collapsed on

12th March 1928 (Rogers, 2007). This collapse is still considered to be one of the biggest engineering failures in the history of the United States of America and remains the second most deadly catastrophe to occur in California. Two main factors were identified as the reason for the collapse: the soil over which the dam was built and the poor construction materials used (Rogers, 2007).



(a)

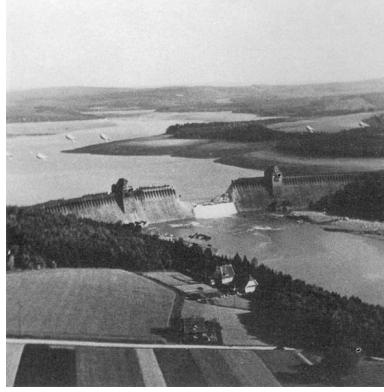


(b)

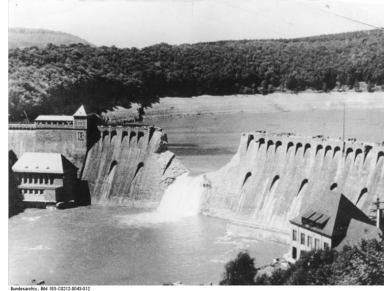
Figure 1.1: (a) California, USA, St. Francis Dam in 1926 (b) St. Francis Dam after the collapse (1928) and inspection by William Mulholland, the engineer responsible for the dam's construction (Rogers, 2007).

A second example is a case of a dam-break induced by Man. This dam-break event occurred during the 2nd World War, in the sequence of the operation Chastise, led by the British Royal Air Force in 1943 (Figure 1.2). Its targets were the German dams of Mohne, Eder, Sorpe and Ennepe, located in the Ruhr Valley, an important German industrial site. The objective was the destruction of the dams and the consequent inundation of the valley, hoping that way to slow down Germany's war effort. The operation

Chastise was not a full military success, but it showed that it was possible to attack the heart of the Third Reich (Dambusters, 2007).



(a)



(b)

Figure 1.2: The operation Chastise was a military operation whose objective was the destruction of the German dams in the industrial Ruhr valley to slow down Germany's war effort. (a) Mohne Dam (1943) and (d) Eder Dam (1943) after bombing (Dambusters, 2007).

Dam-break events occur everywhere around the world and the Table 1.1, lists, without being exhaustive, some of the main events, dating from the 19th century to the present. As it can be seen, at least once per decade there is a major dam-break event (Chanson, 2006a), thus showing the pertinence of performing dam-break studies.

It is thus important to understand the physics of the flow in order to be able to predict how the water level will evolve with time, and what is the time needed for the dam-break inundation wave to reach different regions. Research on the dam-break flow, from its initial instants to the end, is therefore useful to obtain a deeper insight on the dam-break flow physics. A better comprehension of the dam-break flow will lead to better models resulting in accurate predictions on the dam-break flow propagation.

From the different topics associated to the dam-break flow, the characterisation of the velocity field is of major importance for ac-

Table 1.1: Small list of dam-break events from 1889 to 2011 from different sources (Aleixo, 2012; Safety, 2012; Chanson, 2006a).

Year	Dam	Country	Losses (estimate)
1889	Southfork dam	USA	2200
1900	McDonald dam	USA	N/A
1916	Desná Dam	Czech Republic	N/A
1923	Gleno Dam	Italy	356
1943	Eder Mohne	Germany	1300
1959	Malpasset	France	421
1963	Vajont Dam	Italy	2500
1975	Banqiao and Shimantan	China	171.000
1982	Tous Dam	Spain	8
1985	Val di Stava dam	Italy	268
1996	Meadow Pond dam	USA	1
2004	Camará dam	Brazil	3
2011	Fujinuma	Japan	4-8

curate predictions on the dam-break propagation, and for which, unfortunately, not many studies exist.

1.2 Objectives of the Thesis

This thesis is focused in the experimental analysis of the dam-break flow field. Its first objective is the analysis of the initial stages (to be defined in section 1.5.2) of a dam-break flow. The analysis will focus not only on the evolution of the water surface but special emphasis is given to the determination of the velocity field. For this purpose imaging techniques are used and their application is tested in the case of a severe transient flow.

The second objective is to investigate the hypotheses on the velocity behaviour that are in the formulation of the two-layer model as presented in section 1.5.3. These hypotheses are:

1. Constant velocity profile in the water layer.
2. Constant velocity profile in the sediment layer.

Simultaneous measurements of both water and sediment layers velocity fields are performed.

The third objective is to lay down a methodology for a statistical analysis of the dam-break flow by two ways: a) using fast acquisition techniques, and b) by means of repeating the same experiment n -times in order to obtain a significant data set, where the ensemble-statistics tools can be used.

The fourth objective is to apply additional techniques to the velocity measurement and to test their capacity to comply with the dam-break flow severe transient requirements. In addition to the imaging techniques, which are the main techniques used, a Doppler based technique capable of measuring velocity profiles was used in an attempt to obtain a better spatial resolution near the bed region.

1.3 Thesis Organisation

Chapter 1 Introduction: The next sections of the present chapter are devoted to a detailed description of the existing bibliography about the dam-break problem (section 1.4), and in section 1.5 some of the existing theoretical and numerical models are presented:

1. The Ritter (1892) solution is presented in subsection 1.5.1.
2. Subsection 1.5.2 is devoted to the solution for the initial stages of the dam-break flow proposed by Pohle (1952).
3. And finally subsection 1.5.3 presents the main features of the two-layer model, originally proposed by Capart and Young (2002).

Chapter 2 Experimental Methodology: This chapter presents the experimental installation used for the experiments. A detailed analysis is made to the gate removal mechanism and its influence in the flow. In this chapter the measurement techniques that will be used to measure the dam-break flow are presented and discussed. Three different velocity measurement techniques were applied to the dam-break flow. Two techniques based on image processing and acquisition were used while the third was based on the Doppler effect. Their fundamentals are presented and the post-processing tools are described. A list of the different measuring devices and instrumentation used is presented together with a list of all the experiments made.

Chapter 3 Dam-Break Experiments: This chapter is devoted to the experimental results obtained for different types of the dam-break flow. The Hydrodynamic dam-break flow experiments are presented and the velocity field evolution with time is presented. Two cases are considered corresponding to different initial water heights. The obtained results are compared with Pohle's (1952) solution. For the Sedimentologic Dam-Break flow two types of sediments are used and the experiments made are also presented along with the obtained results. A procedure for interface identification is proposed and the velocity field for both liquid and sediment layer is presented. A comparison between the different measurement techniques used is also made.

Chapter 4 Statistical Analysis of the Dam-Break Flow: By repeating several times the dam-break flow measurements, it was possible to make a statistical analysis of this transient flow. This chapter is, therefore, devoted to the statistical analysis of the dam-break flow. The concept of ensemble

statistics is introduced in this chapter. Time averages within the flow duration are also considered. The limitations and requirements for such analysis are presented and discussed.

Chapter 5 Conclusions: The conclusions of this thesis are presented here and detailed by chapter. Proposals for future research directions are also detailed within this chapter.

1.4 State of the Art and Literature Review

This section presents the State of the Art and the Literature Review. This review is organized according to the following main research fields:

1. Theoretical.
2. Experimental.
3. Numerical.

1.4.1 Literature Review on the Dam-Break Flow: Theoretical Investigations

This subsection presents the important milestones in the dam-break flow theoretical investigations. The first known theoretical analysis of the dam-break flow was proposed by Ritter (1892). According to Chanson (2004), it was the collapse of the Southfork dam in the United States in 1889 that triggered Ritter's investigations about the dam-break flow. Ritter (1892) derived an expression for both wave height and wave velocity evolution in the framework of shallow-water equations. His guiding hypotheses were the following: the dam break was instantaneous, the flow was processed

in a prismatic semi-infinite channel and the fluid considered to be ideal. The channel bed was considered to be smooth and the dam's downstream reach had no initial water level, here referred as initial dry bed conditions. It was only sixty five years later that a generalisation of Ritter solution for an initial water level in the dam's downstream reach (wet bed conditions) was presented by Stoker (1957). As in the case of Ritter solution, Stoker (1957) solution was derived in the shallow-water equations framework considering a semi-infinite prismatic smooth channel, an ideal fluid and an instantaneous dam break.

The ideal fluid hypothesis used by Ritter and Stoker implied that no friction effect was considered, thus limiting the application of their solutions. The analysis of the friction effect in the dam-break flow was performed initially by Dressler (1952, 1954) and independently by Whitham (1955). Both considered that the friction effect was limited to the wave tip region. In that region a Chézy-type friction term was introduced by Dressler. The subject of the friction effect was again analysed by Chanson and Aoki (2001) and later by Hogg and Pritchard (2004).

Experiments have shown that in the initial instants the dam-break flow most diverges from the Ritter Solution. A theoretical solution for the initial instants was proposed by Pohle (1952). Assumed irrotational flow and that initially the fluid behaves as a free falling body under gravity. Pohle's solution predicted a wave front celerity lower and more realistic than the one proposed by Ritter (1892). The initial stages were also analysed by Korobkin and Yilmaz (2009) who obtained the leading-order asymptotic solution of the two-dimensional dam-break problem.

The inherent complexity of the dam-break flow led Hunt (1982, 1984) to propose an asymptotic analysis for the dam-break flow analysis. Hunt obtained a complete kinematic wave asymptotic solution for the case of a dam-break flow down a sloping, prismatic,

two-dimensional channel. In Lauber and Hager (1998a) a solution for the shallow- water equations with friction slope in the source term was presented. An analytical solution for the dam-break flow water profile and celerity over sloping channels was presented by Mangeney et al. (2000). Chanson (2006b) proposed a solution for the water profile and wave celerity for the laminar dam-break flow. This kind of flow, laminar dam-break, was also analysed by Aguirre-Pe et al. (1995) and Nsom (2002). Besides of the theoretical analysis, Aguirre-Pe et al. (1995) and Nsom (2002) presented also experimentally results for the viscous dam-break flow.

Albeit the progresses made the obtained theoretical solutions were quite limited. One of the major limitations was the absence of the sediments and their effects in the dam-break flow propagation. The works made by Fracarollo and Armanini (1999), Fracarollo and Capart (2002) and Pritchard and Hogg (2002) are milestones in the theoretical analysis of the dam-break flow over sediment bed.

The dam-break equations were also applied in the Maritime Hydraulics to the study of tsunami surges by Chanson (2005a).

1.4.2 Literature Review on the Dam-Break Flow: Experimental Investigations

This subsection presents some of the relevant works on the experimental analysis of the dam-break flow. The first experiments on the dam-break were made considering a flat fixed bed. The pioneer experimental works were made by Schoklitsch (1917) and Eguiazaroff (1935) and their goal was to describe water elevation and wave celerity by means of water level gages and flow photographs.

The data collection on dam-break flows collected by the United States Army Corps of Engineers (USACE, 1960, 1961) deserves also to be quoted. Bell et al. (1992) made laboratory experiments of the dam-break flow using straight and curved channels, measuring in a

non-intrusive way the water height evolution in time. Bellos et al. (1992) also made water level measurements for the case of the dam-break flow but considering a converging-diverging flume.

Lauber and Hager (1998a,b) also analysed the dam-break wave propagation over smooth flat bed for horizontal and sloping bed conditions. They proposed a criterion to consider the gate removal as instantaneous and most important, provided a valuable set data of water wave celerity and depth that is still used to calibrate numerical models as reported in Tan and Chu (2009). Soares-Frazão (2007) made experiments of a dam-break over a triangular bottom sill and Soares-Frazão and Zech (2008) studied the dam-break flow propagation over an ideal city.

The dam-break over a bed of loose sediments has also been studied experimentally. Leal et al. (2001), Leal et al. (2002) and Leal et al. (2006) conducted measurements of the dam-break flow over a sediment bed as well as Spinewine (2005) that presents an extended data set for the dam-break flow over sediment bed. A part of that work can be found in Spinewine and Zech (2007). Soares-Frazão et al. (2007) analysed the effects of the dam-break induced morphological changes in an erodible channel.

Estrade (1967) studied experimentally the initial stages and proposed a criterion for the validity of Pohle's solution. The initial stages were also analysed later by Stansby et al. (1998) and Bukreev and Gusev (2005). The work by Stansby et al. (1998) is one of the most cited works about the initial stages of the dam-break flow. In their experiments Stansby et al. (1998) identify several features seen during the initial stages like the formation of an horizontal jet-like flow. And Bukreev and Gusev (2005) listed the limitations of the shallow-water approach in the first instants of the dam-break flow.

Experimental techniques have also been put forward and applied to the experimental study of the dam-break flow. Initially

imaging techniques were used only for visualisation purpose. With the development of imaging techniques, namely PIV by Meynart (1982, 1983), Keane and Adrian (1992) and others (see a detailed revue in Raffel et al. (2007)) it was possible to measure not only the velocity of the dam-break wave front, but also the point-wise velocity. Results of the velocity field measurements of the dam-break flow have been presented by Aleixo et al. (2010) and Aleixo et al. (2011). Eaket et al. (2005) applied stereoscopy for three-dimensional dam-break flow water level measurements.

Aureli et al. (2008, 2011) proposed a method of measuring the water depth of a dam-break wave as a function of the camera's grey scale. Cochard and Ancey (2008) used a fringe pattern projection on the dam-break wave surface to extract the information about the dam-break wave water surface profile in three dimensions.

Spinewine et al. (2011b) applied the concentration measurement technique proposed by Spinewine et al. (2011) to study the sediment concentration profile in the dam-break transport layer. Experimental dam-break flow studies have also been made considering non-newtonian fluids. The works made by Chanson et al. (2006), Matson and Hogg (2007) and Ancey and Crochard (2009) are some of them. Chanson et al. (2006) used a thixotropic fluid and Matson and Hogg (2007) and Ancey and Crochard (2009) a Herschel-Bulkley fluid.

1.4.3 Literature Review on the Dam-Break Flow: Numerical Investigations

This subsection presents some of the main references in the numerical analysis of the dam-break flow. The limitations of the developed theoretical solutions presented in the previous subsection and the need of tools capable of predicting the dam-break flow evolution pushed for the development of numerical models. With the devel-

opment of computers and numerical methods the numerically study of the dam-break flow was made possible.

In modelling shallow-water equations the works by Roe (1981) and Harten et al. (1983) have to be mentioned: proposing approximate Riemann solvers that have proved to be highly accurate when analysing different kinds of flows; namely dam-break flows.

Initially the numerical models were essentially based in one dimensional shallow-water equations but more recently two and three-dimensional shallow-water models have been proposed. In the shallow-water equations framework, Fennema and Chaudry (1987) made simulations of one-dimensional dam-break flow and Elliot and Chaudry (1992) proposed a numerical model for two-dimensional dam-break flows. Fracarollo and Toro (1995) also used the shallow-water equations to study the two-dimensional dam-break flow, and compared the results with experimental data obtained by the same authors. Fagherazzi et al. (2004) proposed a model based in the discontinuous Galerkin method to solve the shallow-water equations for both one- and two-dimensional dam-break flows.

The developments in numerical modelling allowed also for the inclusion of the sediment transport. Capart and Young (2002) proposed a two-layer numerical model that was later developed by Spinewine (2005) and by Savary (2007). In this model the flow is divided in layers, one for the liquid phase (water layer) and the other for the sediment motion, the sediment layer. The water and sediment layers are described by the respective equations for mass and momentum conservation.

The development of CFD made possible the analysis of dam-break flow by means of the Navier-Stokes equation. Abdolmaleki et al. (2004) used a Navier-Stokes solver to study the dam-break flow, being the free surface described by the Volume of Fluid method (Hirt and Nicols, 1981). By means of a Reynolds Average Navier-Stokes equations model initially developed by Lin and Liu (1998)

the initial stages of the dam-break flow were studied by Shigematsu et al. (2004). Their numerical results were compared against the experimental data of Stansby et al. (1998). More recently Oertel and Bung (2012) used also a Navier-Stokes model to describe the initial instants of the dam-break flow. Using velocity data obtained by means of PIV those authors validated the numerical simulations made with VOF method applied to hydrodynamic dam-break flow conditions.

Mesh-free particle (Lagrangian) methods have also been used to study the dam-break flow. Vacondio et al. (2012) used Smooth Particle Hydrodynamics (SPH) to simulate flood events and applied it to a dyke break. Using the same approach (SPH) Canelas et al. (2012) analysed the velocity field of a dam-break flow over flat smooth bed. Shakibaeinia and Jin (2011) used a variation of the Moving Particle Semi-Implicit (MPS) method to study the dam-break flow over mobile bed, in this article, Shakibaeinia and Lin discuss also the effect of the gate opening in the consecutive flow. The development of numerical models devoted to the dam-break flow made it possible to study *a posteriori* some real scale dam-break events. Valliani et al. (2002) studied the Malpasset dam (France) break by means of a two-dimensional numerical model. Begnudelli and Sanders (2007) analysed the St. Francis dam (USA) collapse, Alcrudo and Mulet (2007) made a study on the Tous dam (Spain) collapse and the Gleno (Italy) dam break analysis was made by Pilotti et al. (2011).

1.5 Dam-Break Flow Models

This section presents some of the existing models related to the dam-break flow and that will be used later in this work. First the Ritter (1892) solution is derived and its limits of application

are discussed. Secondly, the Pohle (1952) model is presented along with Estrade (1967) results for the initial stages. A model developed at UCL capable of describing the dam-break flow is presented and emphasis is given to the model's hypotheses, whose validation is one of the motivations for the present work.

1.5.1 The Ritter Solution

Inspired by the collapse of the South Fork dam, Ritter (1892) presented the solution for a dam-break wave of an ideal fluid over a semi-infinite dry channel. Ritter solution can be obtained from the St.-Venant equations and its demonstration is here made. Considering a uniform rectangular channel where friction is neglected, the St.-Venant equations for continuity and momentum can be written as (Mahmood and Yevjevich, 1975; Graf and Altinakar, 2001):

$$\frac{\partial h}{\partial t} + \frac{\partial}{\partial x} (h\hat{u}) = 0 \quad (1.1)$$

$$\frac{\partial}{\partial t} (\hat{u}h) + \frac{\partial}{\partial x} \left(h\hat{u}^2 + \frac{1}{2}gh^2 \right) = 0 \quad (1.2)$$

where $h = h(x, t)$ is the flow depth, g is the gravity acceleration, and $\hat{u}$ is the depth-averaged velocity given by:

$$\hat{u} = \frac{1}{h} \int_0^h u dz \quad (1.3)$$

From equations (1.1) and (1.2) the Euler equation can be derived (Mahmood and Yevjevich, 1975):

$$\frac{\partial \hat{u}}{\partial t} + \hat{u} \frac{\partial \hat{u}}{\partial x} + g \frac{\partial h}{\partial x} = 0 \quad (1.4)$$

Multiplying equation (1.1) by g , using the wave celerity $c = \sqrt{gh}$ in equation (1.1) for the continuity equation:

$$2\frac{\partial c}{\partial t} + 2\hat{u}\frac{\partial c}{\partial x} + c\frac{\partial \hat{u}}{\partial x} = 0 \quad (1.5)$$

And for the Euler equation (1.4)

$$\frac{\partial \hat{u}}{\partial t} + \hat{u}\frac{\partial \hat{u}}{\partial x} + 2c\frac{\partial c}{\partial x} = 0 \quad (1.6)$$

Adding and subtracting equations (1.5) and (1.6) one obtains:

$$\frac{\partial}{\partial t}(\hat{u} + 2c) + (\hat{u} + c)\frac{\partial}{\partial x}(\hat{u} + 2c) = 0 \quad (1.7)$$

$$\frac{\partial}{\partial t}(\hat{u} - 2c) + (\hat{u} - c)\frac{\partial}{\partial x}(\hat{u} - 2c) = 0 \quad (1.8)$$

Introducing the differential operators

$$\left. \frac{D}{Dt} \right|_+ = \left\{ \frac{\partial}{\partial t} + (\hat{u} + c)\frac{\partial}{\partial x} \right\}$$

and

$$D/Dt|_- = \left\{ \frac{\partial}{\partial t} + (\hat{u} - c)\frac{\partial}{\partial x} \right\}$$

equations (1.7) and (1.8) can be written as:

$$\left. \frac{D}{Dt} \right|_+ (\hat{u} + 2c) = 0 \quad (1.9)$$

$$\left. \frac{D}{Dt} \right|_- (\hat{u} - 2c) = 0 \quad (1.10)$$

The interpretation of equations (1.9) and (1.10) is straightforward. Equation (1.9) states that $(u + 2c)$ is constant for a point moving through the fluid with velocity $(u + c)$. A similar reasoning

can be made for equation (1.10). Another way to interpret equations (1.9) and (1.10) is to state that they are valid for a point whose motion is characterized by characteristic curves C_1 and C_2 which are the solution of the following ordinary differential equations (Stoker, 1957):

$$\begin{cases} C_1 : \frac{dx}{dt} = \hat{u} + c \\ C_2 : \frac{dx}{dt} = \hat{u} - c \end{cases} \quad (1.11)$$

From equations (1.9) - (1.11) one can write (Stoker, 1957):

$$\begin{cases} \hat{u} + 2c = \alpha_1 = \text{constant along } C_1 \\ \hat{u} - 2c = \alpha_2 = \text{constant along } C_2 \end{cases} \quad (1.12)$$

where α_1 and α_2 are constants. C_1 and C_2 are referred as forward and backward characteristic respectively. The instantaneous dam break creates a negative wave propagating upstream into a fluid at rest with known water depth, h_0 and a positive wave propagating downstream as depicted in Figure 1.3.

It is possible to obtain an analytical solution for the dam-break flow (e.g. Ritter (1892); Stoker (1957)). The following demonstration is adapted from Chanson (2005b). Let one consider that initially the water in the dam is at rest. The dam instantaneous collapse occurs at $t = 0$. It is known that in the initial instants of the dam-break flow there exists a strong vertical acceleration, but for the application of the method of the characteristics, that vertical acceleration will be here disregarded. Furthermore, the pressure distribution will be considered hydrostatic.

In order to apply the method of the characteristics let one consider the scheme of Figure 1.3 where the dam-break removal and a

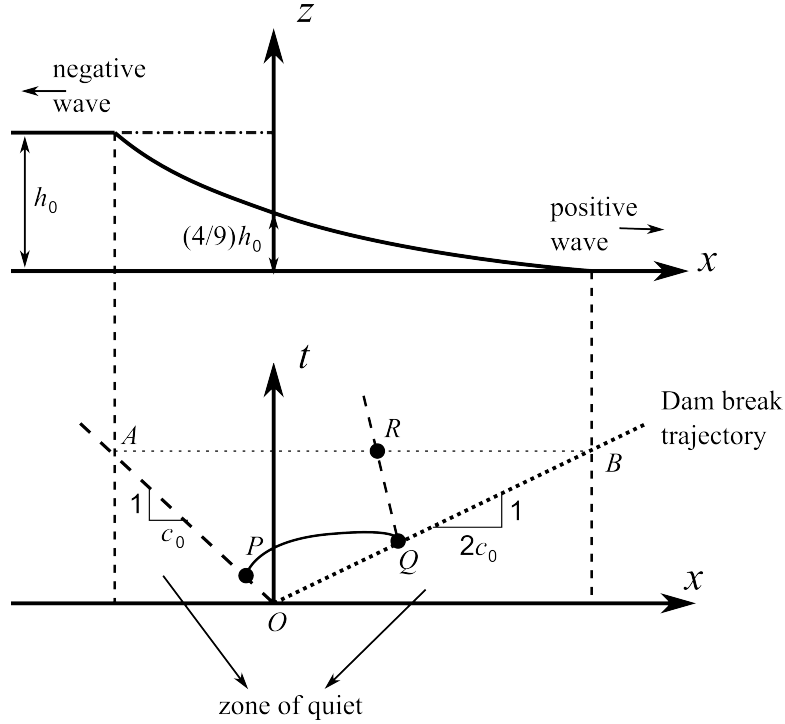


Figure 1.3: Dam break flow scheme and application of the method of characteristics. The segment $\overline{AO}$ defines the initial backward characteristic; $\overline{OB}$ describes the dam-break trajectory and segment $\overline{QR}$ is the backward characteristic. Finally $\overline{PQ}$ is the forward characteristic. The dot-dash line represents the initial ($t = 0$) water level height.

sketch of the characteristics are depicted.

Considering the (x, t) plane of Figure 1.3, the slope of the initial negative characteristic wave, $(\overline{AO})$, is given by (Chanson, 2005b):

$$\frac{dt}{dx} = -\frac{1}{c_0} \quad (1.13)$$

where, for a rectangular channel with and h_0 is the initial water height (Figure 1.3) and

$$c_0 = \sqrt{gh_0} \quad (1.14)$$

For $t > 0$ it is possible to draw the forward characteristics (trajectory $\overline{PQ}$) issuing from the initial backward characteristic (point P) and intersecting the trajectory of the leading edge (point Q). Since at point P , $\hat{u} = 0$ and $c = c_0$ the forward characteristic satisfies the following relation:

$$\hat{u} + 2c = 2c_0 \quad (1.15)$$

At the leading edge one has $h = 0$, and therefore $c = \sqrt{gh} = 0$. And so,

$$\hat{u} = 2c_0 \quad (1.16)$$

Let one considers now any backward characteristics issuing from the dam-break wave front (Figure 1.3, segment $\overline{QR}$). According to Henderson (1966) the C_2 characteristics are straight lines because the initial characteristic is a straight line. The inverse slope of each backward characteristic is a constant and, according to the same author, by using equation (1.15) one has:

$$\frac{dx}{dt} = \hat{u} - c = 2c_0 - 3c \quad (1.17)$$

The integration of the inverse slope gives the water surface profile at the intersection of the C_2 characteristic with a horizontal line $t = \text{constant}$. Following Chanson (2005b) one has:

$$\frac{x}{c_0 t} = 2 - 3\sqrt{\frac{z}{h_0}} \quad \text{and} \quad -1 \leq \frac{x}{c_0 t} \leq 2 \quad (1.18)$$

meaning the the water surface is a parabola. Along the parabolic free surface profile the velocity is given by Henderson (1966)

$$\frac{\hat{u}}{c_0} = \frac{2}{3} \left(1 + \frac{x}{c_0 t} \right) \quad (1.19)$$

Finally the Ritter solution reads (Ritter, 1892):

$$\frac{z}{h_0} = \left(\frac{1}{3} \left(2 - \frac{x}{c_0 t} \right) \right)^2 \quad (1.20)$$

$$\frac{\hat{u}}{c_0} = \frac{2}{3} \left(1 + \frac{x}{c_0 t} \right) \quad (1.21)$$

Let $Z = z/h_0$, $U = \hat{u}/c_0$ is the non-dimensional vertical averaged velocity and $X^* = x/c_0 t$, then equations (1.20) and (1.21) can be written as

$$Z = \left(\frac{1}{3} (2 - X^*) \right)^2 \quad (1.22)$$

$$U = \frac{2}{3} (1 + X^*) \quad (1.23)$$

It is useful sometimes, especially when dealing with dam-break flow images, to normalize the horizontal coordinate by h_0 instead of $c_0 t$, as $X = x/h_0$. With this new variable equations (1.20) and (1.21) can be written as (e.g. Ritter (1892), Henderson (1966), Peregrine and Williams (2001)):

$$Z = \left(\frac{1}{3} \left(2 - \frac{X}{T} \right) \right)^2 \quad (1.24)$$

$$U = \frac{2}{3} \left(1 + \frac{X}{T} \right) \quad (1.25)$$

where $T = t/t_0$, and

$$t_0 = \sqrt{\frac{h_0}{g}}, \quad (1.26)$$

that can be interpreted as the natural time-scale of the dam-break flow. Some particular results for the water level, depth average

velocity and unitary flow rate can be found at the dam section, $X = 0$:

$$U(X = 0) = \frac{2}{3} \quad (1.27)$$

$$Z(X = 0) = \frac{4}{9} \quad (1.28)$$

And for the unitary flow rate:

$$Q(X = 0) = U(X = 0)Z(X = 0) = \frac{8}{27} \quad (1.29)$$

One must note that writing the equations (1.27) to (1.29) in this order shows a geometric progression.

The elegance of the Ritter solution however, is not enough to make it useful when predicting the propagating time of a dam-break wave, especially due to the lack of a friction term.

1.5.2 The Initial Stages of the Dam-Break Flow

This section is dedicated to the initial stages of the dam-break flow. To the author's knowledge, as indicated in section 1.4, there aren't many studies dedicated to the initial stages of the dam-break flow. Pohle (1952) was probably the first to make a study focused on this subject and this section starts by revisiting his work. Unfortunately, according to Estrade (1967), Pohle did not publish many of his results. Pohle's hypotheses were the following:

1. Irrotational flow and viscous dissipation is neglected.
2. Instantaneous dam breaking.
3. Gravity is the only force acting.

Pohle developed his reasoning using the Euler equations written in Lagrangian coordinates with the constraints mentioned above. Let the coordinates of a particle located at position from the dam section be $(x, z) = (x_0, z_0)$ and $(x_0 < 0, z_0 > 0)$ for $t < 0$. The particle coordinates, (x, y) will change with time and can be written as (Pohle, 1952; Vischer and Hager, 1998):

$$x(t) = x_0 + x^{(1)}(x_0, z_0)t + x^{(2)}(x_0, z_0)t^2 + \dots \quad (1.30)$$

$$z(t) = z_0 + z^{(1)}(x_0, z_0)t + z^{(2)}(x_0, z_0)t^2 + \dots \quad (1.31)$$

where the functions $x^{(1)}$ and $z^{(1)}$ represent the velocity components for each direction and $x^{(2)}$, $z^{(2)}$ are the particle's accelerations. Before the dam removal the water is at rest and so: $x^{(1)} = z^{(1)} = 0$. By means of conformal mapping, Pohle (1952) demonstrated that:

$$x^{(2)}(x_0, z_0) = \frac{g}{2\pi} \ln \left(\frac{\cos^2 \left(\frac{\pi z_0}{4h_0} \right) + \sinh^2 \left(\frac{\pi x_0}{4h_0} \right)}{\sin^2 \left(\frac{\pi z_0}{4h_0} \right) + \sinh^2 \left(\frac{\pi x_0}{4h_0} \right)} \right) \quad (1.32)$$

$$z^{(2)}(x_0, z_0) = -\frac{g}{\pi} \arctan \left(\frac{\sin \left(\frac{\pi z_0}{2h_0} \right)}{\sinh \left(\frac{\pi x_0}{2h_0} \right)} \right) \quad (1.33)$$

According to Pohle (1952) and Vischer and Hager (1998) equations (1.30) to (1.33) define the free surface profile for the very initial instants of the dam-break flow, as seen in Figure 1.4. From equation (1.32), for $z_0 = h_0$, one has $x^{(2)} = 0$ at the free surface, after the dam removal, the particles will move in the vertical direction. Vischer and Hager (1998), following Pohle (1952) proposed for the water elevation upstream and downstream of the dam the

following expressions, respectively:

$$Z = 1 - \frac{T^2}{\pi} \arctan \left(\left(\sinh \frac{-\pi X}{2} \right)^{-1} \right), \quad X \leq 0 \quad (1.34)$$

$$Z = \frac{4}{\pi} \left(\operatorname{arc cot} \left(\exp \left(\frac{\pi X}{T^2} \right) \right) \right) - \frac{T^2}{2}, \quad X > 0 \quad (1.35)$$

where $Z = z/h_0$, $X = x/h_0$ and $T = t/t_0$ as defined in section 1.5.1.

At the dam section, $X = 0$, the following expression holds (Vischer and Hager, 1998):

$$Z = 1 - \frac{T^2}{2} \quad (1.36)$$

From equation (1.36) it is clear that a particle located between the reservoir surface and the dam section moves like a free falling body. With time the free surface in the reservoir tends to curve, with the centre of curvature below the free surface, and vice versa for the downstream region (Vischer and Hager, 1998).

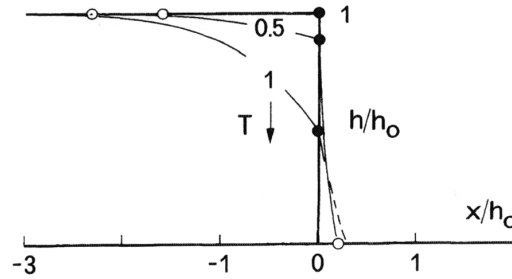


Figure 1.4: Solution of Pohle (1952), for the initial dam-break stages according to Martin (1990) and Vischer and Hager (1998).

For small values of X and T Vischer and Hager (1998) propose for equations (1.34) and (1.35) the following expressions:

$$Z = 1 - \frac{T^2}{2} (1 + \tanh(X)), \quad X \leq 0 \quad (1.37)$$

$$Z = 1 - \tanh\left(\frac{2X}{T^2}\right) - \frac{T^2}{2}, \quad X > 0 \quad (1.38)$$

Estrade (1967) studied experimentally the initial instants of the dam-break flow and proposed a criterion for which the Pohle solution was valid. According to Estrade (1967) Pohle's solution is valid for:

$$t_{PE} < 0.7 t_0 \quad (1.39)$$

where t_{PE} is here referred as Pohle-Estrade time, and $t_0 = \sqrt{h_0/g}$ is the natural time-scale of the dam-break flow. Therefore equations (1.37) and (1.38) are valid if $T < 0.7$.

Estrade (1967) mentions also the importance of the gate mechanism in the first stages of the dam-break flow behaviour. The gate influence in the initial stages of the dam-break flow is also mentioned by Liem and Kongeter (1999) and Shigematsu et al. (2004) and here it will be analysed in section 2.1.1.

The work by Stansby et al. (1998) is a milestone in the analysis of the initial stages of the dam-break flow for hydrodynamic conditions (no sediments), and considering different ratios between water height upstream and downstream of the dam. They departed from the experimental observations and, in their experiments in a flume where the dam break was simulated by a upward moving gate, they observed the formation of a jet in the initial times (Figure 1.5). For the *small times* of the initial release, ($t \leq 0.085$ s) these authors considered a non-linear potential flow approach that

was able to qualitatively predict the observed jet formation. For later times ($t > 0.12$ s in dry bed conditions) the authors compared their observations with the Stoker solution (Stoker, 1957) for which the comparison showed a close agreement.

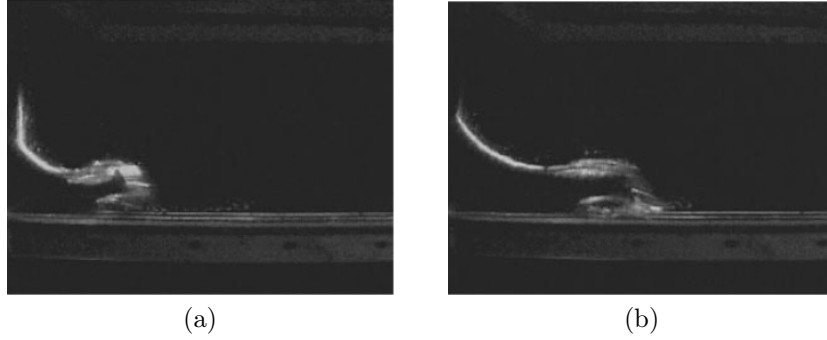


Figure 1.5: Experimental results of Stansby et al. (1998) for the initial times: (a) $T = 0.84$ and (b) $T = 1.25$. The gate is, for each image in the left side.

The numerical studies carried out by Shigematsu et al. (2004) using a Navier-Stokes equations-based model, showed that after $T > 3$ the agreement between the computed water surface and the measurements of Stansby et al. (1998) was quite good. From the simulations made by Shigematsu et al. (2004) it is clear that the gate movement is important in the initial stages, and simulations that include the gate movement have better agreement with the experimental results.

In their analysis of the instants times Bukreev and Gusev (2005) found that only after a time $T = 2.4$ the non-dimensional water depth at the dam section was $4/9$.

In this work initial stages will be referred to dam-break evolution in the time interval between the dam removal and the moment when the shallow-water equations hypotheses are valid.

1.5.3 The Two-Layer Model

Dam-break flows over sediment beds are usually simulated by means of a numerical models. One of the models that can be used is the two-layer model that is here briefly presented. The two-layer model is a numerical, shallow-water equations based that was proposed by Capart and Young (2002) and later developed by Spinewine (2005) and Savary (2007). A synthesis of this model can be found in Zech et al. (2009).

This is a model that is capable of handling sediment transport and its strategy consists in assigning a layer to each phase of the flow, solid and liquid. This idea is backed by dam-break flow laboratory experiments (Spinewine, 2005) where it is possible to see, as in Figure 1.6, that the sediments move close to the bed in the so called transport layer (Raudkivi, 1998). In Figure 1.6 it is also possible to observe a fixed bed, a moving sediment layer, and an evolving water layer.

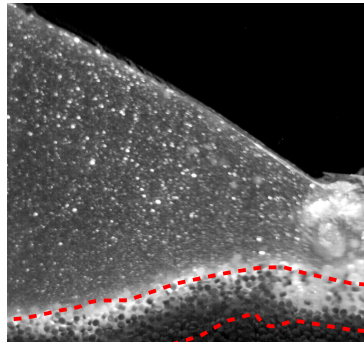


Figure 1.6: Dam-break flow image taken at $T = 0.824$ in which the region where the moving sediments can be found is roughly indicated by the dashed red line. The sediments motion in a defined region near the fixed bed inspired the two-layer model proposed by Capart and Young (2002).

In Figure 1.7 a scheme for the two-layer model is presented. A layer is used to described each part of the flow: one for the liquid

phase and another for the mixture between water and sediments. The variables for the liquid phase layer (referred as water layer) are h_w , u_w and τ_{ws} for water height, water velocity, and shear stress between the water layer and the sediment layer, respectively. The sediment layer, but also here referred as transport layer, is described by the variables h_s , for the sediment layer height, u_s for the sediment layer velocity, by the shear stress exerted on the water layer τ_{sw} and by the shear stress exerted over the fixed bed τ_s . Finally the fixed bed is described by a level, z_b and a shear stress τ_b . The following hypotheses are assumed (Spinewine, 2005; Savary, 2007; Zech et al., 2009): first the water and sediment layers velocity is constant, but can be different, i.e.:

$$u_w = A_1 \quad u_w = A_2 \quad A_1 \neq A_2 \quad (1.40)$$

where A_1 and A_2 are constants. This concept is also illustrated in Figure 1.7. The sediment layer will be a mixture of sediments and water whose density ρ'_s will be a function of the sediment's concentration in the transport layer.

$$\rho'_s = C_s \rho_s \quad (1.41)$$

where C_s is the sediment concentration in the transport layer, which is smaller than the fixed bed concentration, C_b and ρ_s is the density of the sediments. Another hypothesis made in this two-layer model is that the sediment concentration C_s is constant through out the transport layer.

For each layer a continuity equation and a momentum conservation is derived (Capart and Young, 2002; Spinewine, 2005; Zech et al., 2009). For the water layer the continuity and momentum

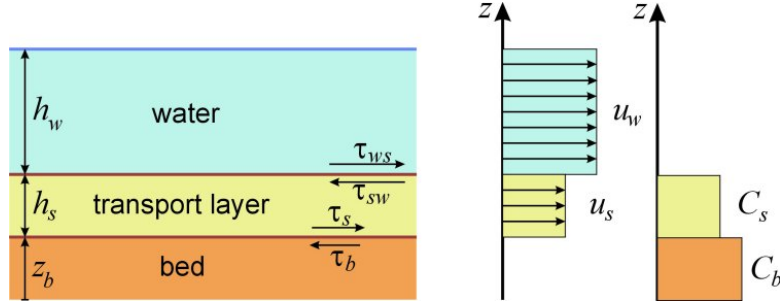


Figure 1.7: Scheme of the Two-Layer model proposed by Capart and Young (2002). Over the fixed bed, the moving sediments can be found in the transport layer and the liquid phase is represented by water. h_w is the water height, h_s denotes the transport layer height, and z_b is the fixed bed level. τ_{ws} is the shear stress caused by the water layer in the transport layer and τ_{sw} is the transport layer shear stress applied on the water layer. τ_s and τ_b represent the shear stress of the sediment layer over the fixed bed and the shear stress of the bed on the sediment layer. u_w is the velocity profiles on the water layer and u_s and c_s the velocity and concentration profiles in the sediment layer, assumed to be constant. C_s and C_b are assumed to be constant (Spinewine, 2005; Zech et al., 2009).

equations are given respectively by:

$$\frac{\partial h_w}{\partial t} + \frac{\partial}{\partial x} (h_w u_w) = e_s \quad (1.42)$$

where $e_s = -\partial z_s / \partial t$ is the rate of change of the sediment layer level and z_s is the sediment layer level.

$$\begin{aligned} \frac{\partial}{\partial t} (h_w u_w) + \frac{\partial}{\partial x} \left(h_w u_w^2 + \frac{1}{2} g h_w^2 \right) \\ + g h_w \frac{\partial}{\partial x} (z_b + h_s) = u_w e_s - \frac{\tau_{ws}}{\rho_w} \end{aligned} \quad (1.43)$$

where h_w is the water height and u_w is the water velocity. The

term $h_w u_w^2$ accounts for the kinetic energy and the term $1/2gh_w^2$ with the hydrostatic pressure. The pressure due to the bottom slope of the layer is given by $gh_w \partial(z_b + h_s)/\partial x$. To account for momentum transfer due to erosion/deposition the term $u_w e_s$ is used and finally the dissipation term is given by τ_{ws}/ρ_w . τ_{ws} is the shear stress of the water layer over the transport layer.

For the sediment layer mass and momentum conservation are given respectively by (Spinewine, 2005; Savary, 2007; Zech et al., 2009):

$$\frac{\partial h_s}{\partial t} + \frac{\partial}{\partial x} (h_s u_s) = e_b - e_s \quad (1.44)$$

$$\begin{aligned} \frac{\partial}{\partial t} (h_s u_s) + \frac{\partial}{\partial x} \left(h_s u_s^2 + \frac{1}{2} g h_s^2 \right) + g h_s \frac{\partial}{\partial x} \left(z_b + \frac{\rho_w}{\rho'_s} h_s \right) = \\ = -\frac{\rho_w}{\rho'_s} u_w e_s + \frac{1}{\rho'_s} (\tau_{ws} - \tau_b) \end{aligned} \quad (1.45)$$

where h_s is the sediment layer thickness and u_s is the sediment velocity, ρ'_s is the transport layer density. The term $h_s u_s^2$ accounts for the kinetic energy and the term $gh_s^2/2$ can be taken as an *hydrostatic* pressure. The pressure due to the bottom slope of the layer is now given by $gh_s \partial(z_b + (\rho_w/\rho'_s)h_s)/\partial x$. To account for the momentum transfer due to deposition/erosion a correction term $(\rho_w/\rho'_s)u_w e_s$ is used and for the sediment layer the dissipation term is given by $(\tau_{ws} - \tau_b)/\rho'_s$. Finally τ_b is the shear stress over the fixed bed.

Also important for the model are their closure relations. Erosion, i.e., the process that removes material from the fixed bed into the transport layer is given by, e_b (e.g. Zech et al. (2009)):

$$e_b = \frac{1}{\rho_b |u_s|} (\tau_s - \tau_b) \quad (1.46)$$

where τ_s is the shear stress caused by the movement of the sediment layer over the fixed bed. The continuity of the vertical mass exchanges leads to a relation between both rates, e_b and e_s , given by (e.g. Zech et al. (2009)):

$$e_s = -\frac{C_b - C_s}{C_s} e_b \quad (1.47)$$

where C_s and C_b are the sediment layer and fixed bed concentrations.

For the shear stresses a Chézy like-relationship is used (e.g. Zech et al. (2009)). For the shear stress of the water layer on the sediment layer one has:

$$\tau_{ws} = \rho_w C_{fw} (u_w - u_s)^2 \quad (1.48)$$

where C_{fw} is the friction coefficient that has to be calibrated. The shear stress exerted by the sediment layer on the fixed bed, τ_s is given by:

$$\tau_s = \rho_s C_{fs} u_s^2 \quad (1.49)$$

where C_{fs} is a friction coefficient, and similarly to C_{fw} has to be calibrated. Finally the bed shear stress, τ_b is given by a Coulomb type relation:

$$\tau_b = f(C_b, \beta) \quad (1.50)$$

where C_b is the concentration of the bed and β the angle of repose.

Two of the hypotheses of the above model concerning the velocity and concentration behaviour are:

1. Constant velocity profiles in water and in sediment layers.
2. Constant concentration profile in the sediment layer.

Regarding the concentration profile, recent studies made by Spinewine et al. (2011a,b) have shown that the concentration profile in the transport layer is not constant, but rather, it is a linear function of the level, z . The hypothesis concerning the velocity profile will be investigated in the next chapters of this thesis as already stated in section 1.2, and constitutes the main core of this work.

Chapter 2

Experimental Methodology

*"The time has come", the walrus said,
"to talk of many things"
Carroll (1865)*

This chapter is dedicated to the description of the experimental installation, the velocity measurement techniques, the required instrumentation and to list and identify the sets of experiments made. The section Experimental Installation describes the used flume and its main features. Special attention is given to the gate mechanism used to simulate the dam-break flow. In the section Velocity Measurement Techniques, three different methods are used to determine the velocity field/profiles of the dam-break. The section Instrumentation, presents the different types of measurement equipments used during the experiments, and the final section, List of Experiments, presents all the experiments made and their characteristics.

2.1 Experimental Installation

The measurements were carried in the dam-break channel of the Laboratoire Essais Mécaniques Structures et génie Civil (LEMSC), Institute of Mechanics, Materials and Civil Engineering (iMMC),

Université catholique de Louvain. The dam-break channel has a length of 6 m, and a rectangular cross section defined by a width of 0.25 m and a height of 0.5 m. The walls are made of glass, allowing optical access to the flow. The bottom of the channel is usually made of impervious polished wood, yet it can be changed and modified to fit the research needs. Figure 2.1 depicts a scheme of the experiment and Figure 2.2 (a) and 2.2 (b) depicts 3D views of the channel and its structure respectively. Finally Figure 2.3 shows a panorama of the channel before an experiment.

The channel is divided in two equal reaches by a downward moving gate used to simulate the *instantaneous* removal of a dam. The upstream reach works as a water reservoir and remains closed, while the downstream reach is used as test section and remains open. A complete description of the used flume can be found in Spinewine and Zech (2007).

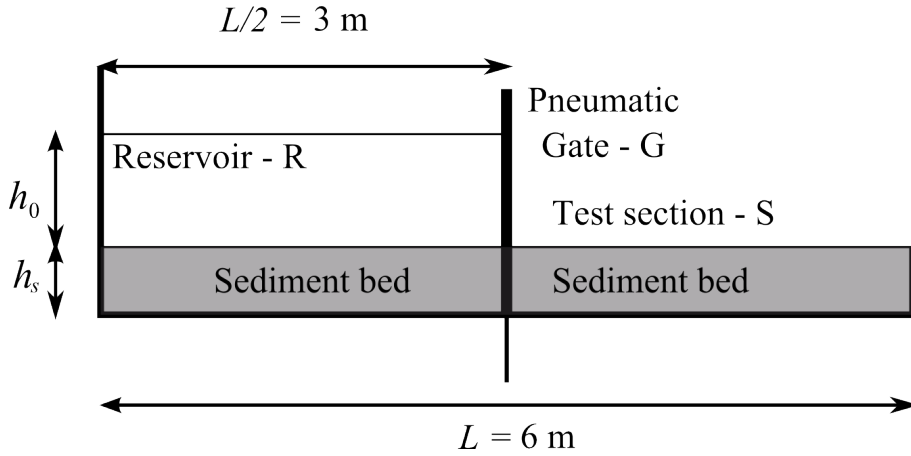
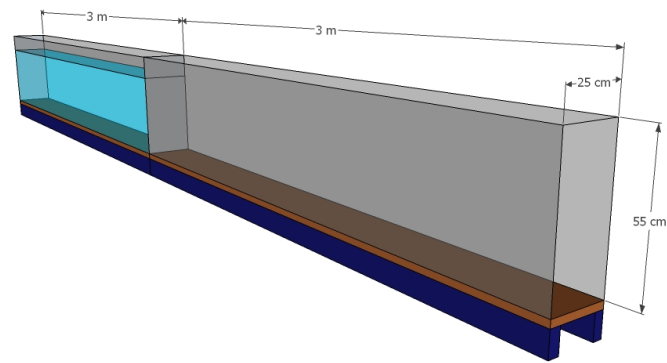
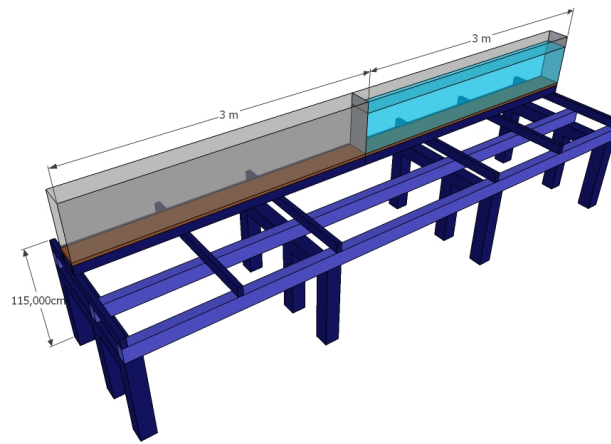


Figure 2.1: General scheme of the channel representing the test section, S, the reservoir, R, the pneumatic gate G, and the possibility of having a sediment bed with an initial height h_s .



(a)



(b)

Figure 2.2: A three-dimensional perspective of the channel is illustrated. (a) the channel. (b) the channel and its structure. Courtesy of Mr. Egli Vorfi (Vorfi, 2011).

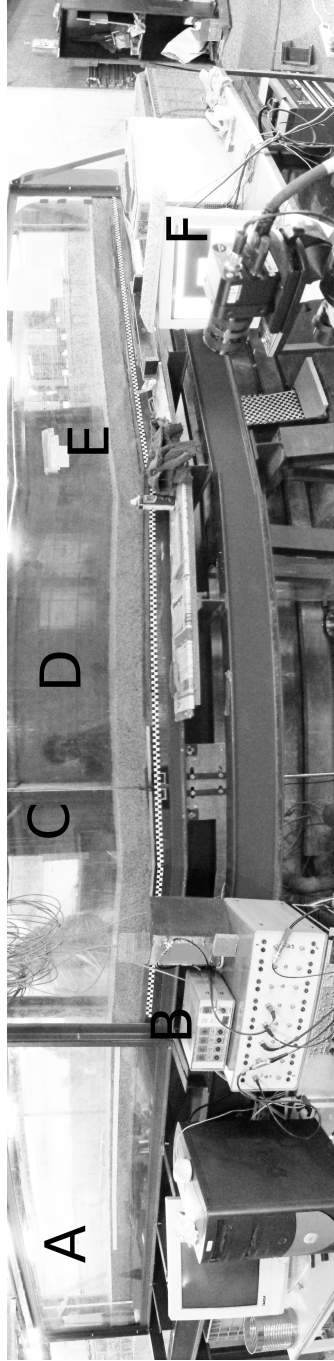


Figure 2.3: A panorama of the channel is presented giving an overview of measurement apparatus. A: upstream section used as reservoir. B: acquisition system (signal conditioner and ADC board) for the acoustic level probes. C: Gate. D: downstream section. E: sediment (sand) bed. F: Dalsa camera acquisition system.

2.1.1 The Downward Moving Gate as Dam Break Model

The downward moving gate, is one of the key features of the dam-break channel where the experiments were performed. This gate consists in a rectangular plate of aluminium with 0.245 m width, 0.865 m height and 0.007 m thick. The lateral sides of the gate have silicone joints which are lubricated allowing both the waterproofing of the gate and reducing friction effects between the silicone joints and the glass. Figure 2.4 (a) depicts the gate as it was installed in the dam-break channel. The gate is connected to a pneumatic jack, placed under the channel, which is used to remove the gate. The gap between the gate and the channel bed is filled with plasticine in order to minimize the water leakage during the experiments.

The downward movement of the gate is one of the key features of the dam-break flume. Usually, in the literature, an upward moving gate is considered (e.g Lauber and Hager (1998b), Stansby et al. (1998), Bukreev and Gusev (2005)). It is usually easier to set-up and it allows a better waterproofing of the channel; however, when considering a sediment bed, a upward moving gate will artificially entrain sediments into the water flow thus disturbing the initial conditions. This reason led Spinewine (2005) to propose a downward moving gate and is the same used in the present study. More technical details on the channel and its moving gate can be found in Bailly and Reybroeck (2003) and Spinewine (2005).

The gate opening is of fundamental importance for the dam-break studies since the initial stages will be strongly affected by the gate's removal time, as mentioned by Lauber and Hager (1998a), Liem and Kongeter (1999). Using a Dalsa Camera (specifications can be found in section 2.4.2) the removal time, position and velocity of the gate tip were determined for two different initial water levels. Figure 2.4 depicts 4 instants of the gate removal, here $t = 0$

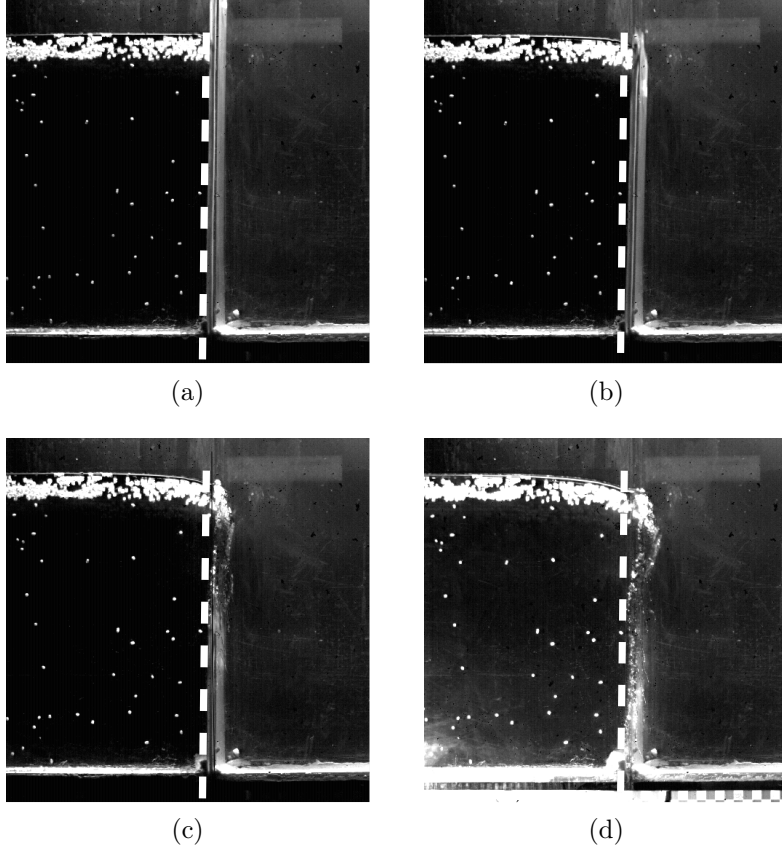


Figure 2.4: Four instants of the gate removal process for an initial water height of $h_0 = 0.325$ m. Gate position at (a) $t = -0.11$ s. (b) $t = -0.07$ s. (c) $t = 0.03$ s. (d) $t = 0.00$ s (gate completely removed). The white dashed line is parallel to the gate initial position.

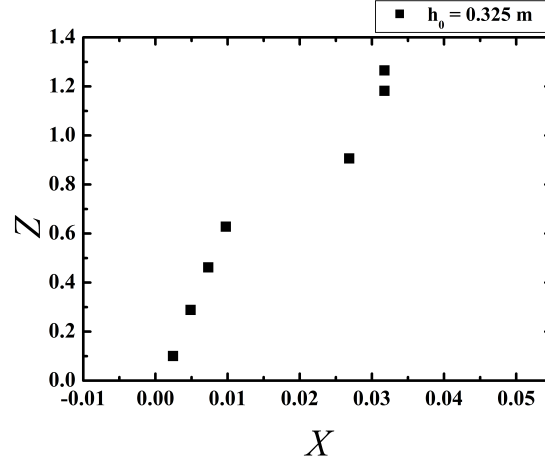
is defined as the instant for which the gate is completely removed. Figure 2.5 (a) and (b) shows the trajectory of the gate tip and Figure 2.6 (a) and (b) the gate tip velocity for the two considered water heights $h_0 = 0.325$ m and $h_0 = 0.40$ m respectively. Table 2.1 indicates the average removal times determined for each initial height. It is to note that a 23% change in the initial water level leads to an increase of the removal time of 171%. Different factors contribute to this: i) the gate movement is uniformly accelerated (constant acceleration) as shown in Figures 2.6 (a) and (b); ii) the gate deflection due to the hydrostatic pressure; iii) the friction between the moving gate and water column.

Table 2.1: Measured average removal times, t_r , of the gate for the two considered water heights and corresponding non-dimensional values.

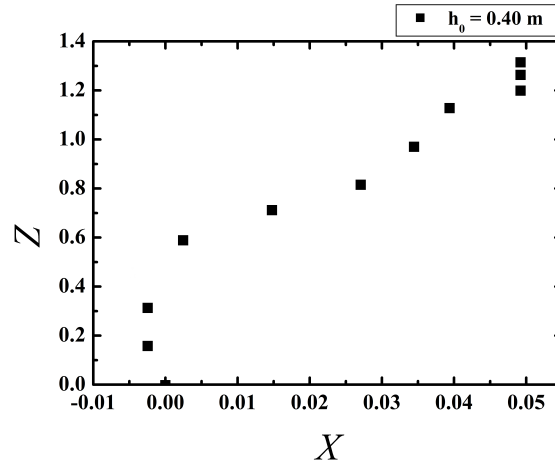
Initial water height (h_0) (m)	t_r (s)	$T_r = t_r/t_0$
0.325	0.07	0.385
0.40	0.12	0.594

To reduce the deflection of the gate due to the hydrostatic pressure, a stop plate is used in the top of the channel. This implies that during the motion the gate will not be exactly perpendicular to the bed. However it will be deflected as seen in Figure 2.5 (a) and (b), where the gate trajectory is depicted, for two different initial water levels, $h_0 = 0.325$ m and $h_0 = 0.40$ m respectively. It was verified that the maximum deflection occurred for $h_0 = 0.40$ m and it was about 0.02 m.

A criterion to determine which removal times of the gate can be considered as instantaneous as function of the initial water height was proposed by Lauber and Hager (1998b). These authors made their experiments in a dam-break channel with an upward moving gate. According to those authors, to be considered as an instantaneous dam-break, the removal time of the gate, t_r , should be less



(a)



(b)

Figure 2.5: Non-dimensional trajectory $(X, Z) = (x/h_0, z/h_0)$ of the gate tip for: (a) $h_0 = 0.325$ m. and (b) $h_0 = 0.40$ m.

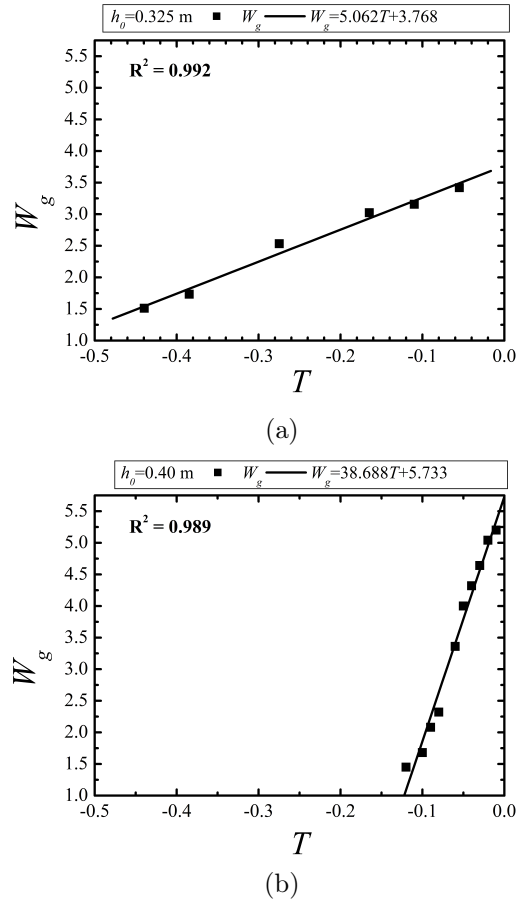


Figure 2.6: Non-dimensional velocity of the gate tip, $W_g = w_g/c_0$, as function of the non-dimensional time ($T = t/t_0$) for (a) $h_0 = 0.325$ m, and (b) $h_0 = 0.40$ m. ($c_0 = \sqrt{gh_0}$ and $t_0 = \sqrt{h_0/g}$).

than:

$$t_r < \sqrt{2\frac{h_0}{g}} \quad (2.1)$$

For the two considered initial water heights, $h_0 = 0.325$ m and $h_0 = 0.40$ m one has respectively:

$$t_r < 0.257 \text{ s} \quad \text{for } h_0 = 0.325 \text{ m} \quad (2.2)$$

$$t_r < 0.285 \text{ s} \quad \text{for } h_0 = 0.40 \text{ m} \quad (2.3)$$

According to Table 2.1 the removal time for $h_0 = 0.325$ m was found to be $t_r = 0.07$ s and for $h_0 = 0.40$ m was $t_r = 0.12$ s. Both values satisfy relations (2.2) and (2.3). With the acquired flow images it is possible to investigate the water surface configuration at the gate removal instant, and check if it is comparable with an undisturbed water surface. Two cases are presented in Figure 2.7, corresponding to an initial water height of 0.325 m (a) and 0.40 m (b). It is possible to see that, while in Figure reffg:gateout (a) ($h_0 = 0.325$, m) the water surface is almost undisturbed, for the Figure 2.7 (b) ($h_0 = 0.40$ m) the water surface is similar the beginning of a weir flow. Figure 2.7 (c) depicts the comparison of the water surface values for the two above mentioned cases.

This shows that the Lauber and Hager (1998b) criterion (eq. 2.1) is not a sufficient criterion to classify a downward gate motion as instantaneous and that the gate removal for the initial condition $h_0 = 0.40$ m can not be considered instantaneous. In Annex A an attempt is made to derive a criterion more stringent than the one proposed by Lauber and Hager (1998b).

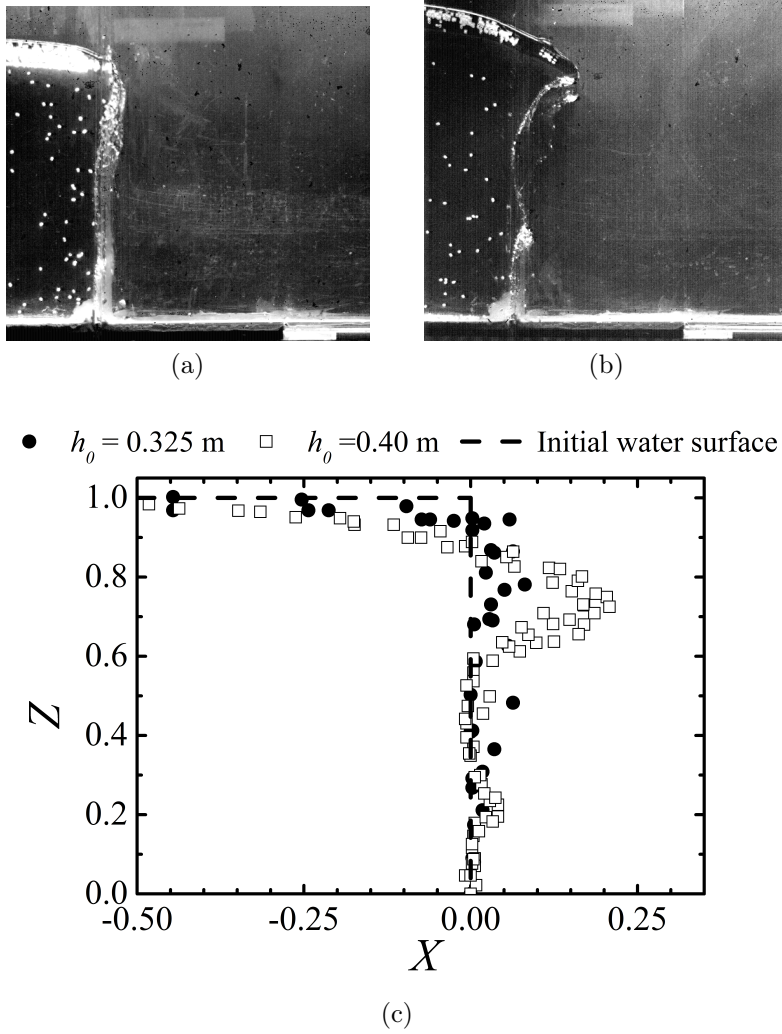


Figure 2.7: The water column surface at the moment the gate is completely removed for two different initial heights (a) $h_0 = 0.325$ m and (b) $h_0 = 0.40$ m. (c) the non-dimensional water surface profile for different runs with different water height values: $h_0 = 0.325$ m (white squares) and $h_0 = 0.40$ m (black circles) is compared with the initial water surface (dashed line).

2.1.2 Sediment Bed Preparation

The sedimentologic dam-break flow requires a bed of sediments and two types of sediments were used: sand and cylinder-shaped PVC pellets. Their characteristics are described in Tables 2.2 and 2.3 .

Table 2.2: Physical properties of the sand used in the experiments (Becker and Kaiser, 2008).

d_{50} (m)	1.72×10^{-3}
Density (kgm^{-3})	2.682×10^{-3}
Mean packing (%)	58
Cohesion (kNm^{-2})	0
Friction angle ($^{\circ}$)	38

Table 2.3: Physical properties of the PVC Pellets used in the experiments (Delvaux, 2011).

d_{50} (m)	3.85×10^{-3}
Density (kgm^{-3})	1.634×10^{-3}
Mean Porosity ϵ_0 (%)	39.22
Mean packing (%)	60.80
Permeability (cm/s)	1.25
Cohesion (kNm^{-2})	0
Friction angle ($^{\circ}$)	30

To ensure that the results are repeatable care must be given to the sediment bed preparation. The two sediments types used are different and require different preparation tasks. To compact the sand sediments a squared based pestle of 14 kg was used. Since the pestle base section was of $0.25 \times 0.25 \text{ m}^2$ the compacting was made by reaches of the same area. The compacting process revealed to affect the mass of sediments transported by the flow. Two methods were considered: method #1 consisting in dropping three times the pestle from a height of about 0.20 m, and method #2 consists in pressing firmly the sand. Three runs where made to analyse the

compacting process and to check its repeatability. The dam-break was processed over the bed, and the transported sediments were collected at the channel outlet. The collected sediments were dried in open air and weighed on a digital balance with a resolution of 1×10^{-4} kg. Table 2.4 shows the results obtained by each process. It can be seen that the collected sediments for each compacting process gave quite similar results. However it was observed that method #2 was easier to implement and it was used during the measurement campaign.

It has to be mention that for both methods the sediments were water saturated upstream and downstream. Water saturation allowed for repeat experiments without having to remove all the sediments after each run. The water saturation of the downstream reach was achieved by means of a water diffuser. This quiescent water supply prevented the artificial erosion of the sediment bed while assuring the water saturation.

Table 2.4: Comparison of the dam-break flow transported sand as function of the method used for compacting the bed.

Run	Method #1 (kg)	Method #2 (kg)
1	4.6838	3.8553
2	4.4801	3.6809
3	4.5008	3.6436
Average (kg)	4.5590	3.7266
Standard-Deviation (kg)	0.112	0.113

For the PVC pellets bed no compaction was used. However after 3 runs the sediment bed was remixed to destroy the pellets arrangements that were formed in the bed after those runs, and again levelled to meet the required configuration.

2.2 Velocity Measurement Techniques

This section presents the different techniques used to measure flow velocities and the velocity profiles of the dam-break flow.

Due to its characteristics, measuring the velocity field of a dam-break flow is not a trivial task to perform. That is why the first experimental measurements about the dam-break were focused in the so called macro-scale variables of the dam-break flow: water level and wave celerity. The measurement techniques available up to the 1980s, although sophisticated like the Laser Doppler Anemometer (Yeh and Cummings, 1964; Durst et al., 1981), were mainly point-wise techniques, which are not practical to apply when measuring a transient flow like the dam-break flow.

The development of image-based techniques to determine the velocity field shed a new light on the dam-break flow. Image-based techniques consist in taking a sequence of photographs of seeded particles in the flow, and then processing those images in order to determine the particles' displacement. To determine the particles' displacements two main strategies exist: the Particle Tracking Velocimetry, hereafter referred as PTV and the Particle Image Velocimetry, hereafter referred as PIV, and both of them were used in this work and are here described.

2.2.1 Particle Tracking Velocimetry (PTV)

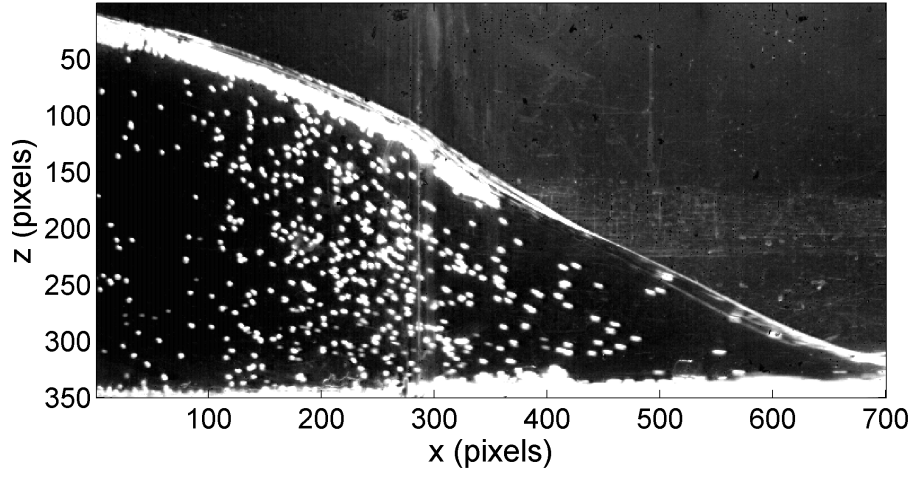
As the name suggests the principle of the PTV lies in tracking each detected particle from frame to frame. In order to do so, the PTV algorithm consists of three main steps. The first step is related to the particle detection and identification in each image of a sequence film. This is made by a succession of filtering processes that lead to the determination of each particle centre position. In the second step each detected particle is tracked in the sequence

of images. In the third step, data filtering and processing leads to the reconstruction of the velocity field (Capart et al., 2002; Aleixo et al., 2010; Aleixo, 2011).

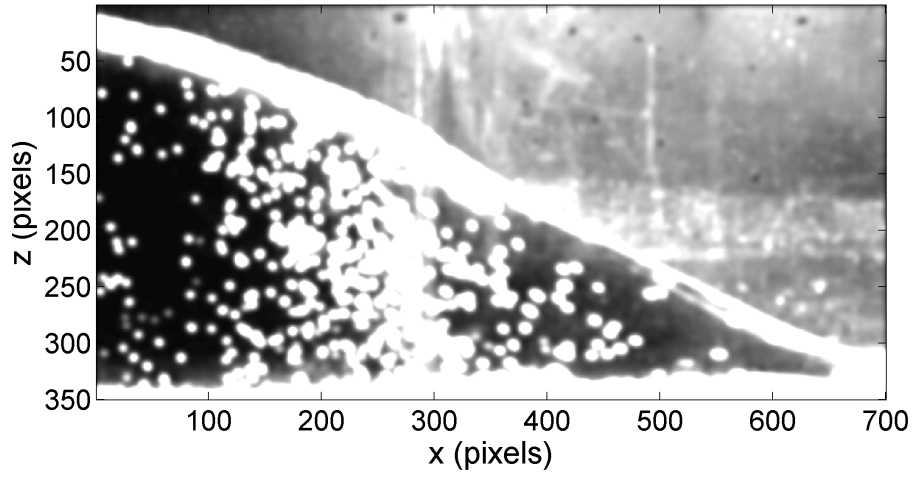
2.2.1.1 Particle Detection Scheme

To be efficient and sufficiently accurate, the detection scheme should be able to provide the particles' coordinates up to a small fraction of their diameters. The seeding particles are assumed to be white and spherical. The illumination conditions are provided so that the particles contrast well with the surrounding fluid and the images are recorded in a grey level map. An example of a raw image of dam-break wave is given in Figure 2.8 (a).

A low-pass filter is firstly applied to the images under the form of a Mexican-hat function. The purpose of this filter is to smooth out the irregularities of the images and parasitic reflections that could otherwise be erroneously identified as particles. For a more detailed description of this step the following references should be consulted: Jähne (1995). After the low-pass filter, a Laplace operator is applied to enhance the white peaks in the image. The resulting effect can be seen in Figure 2.9 (a): the particle centroids are highlighted. The local brightness maxima of the filtered image indicate the discrete positions of particle centroids. Finally, in order to obtain a subpixel resolution a parabola is fitted to the grey levels along the x and y directions (Capart et al., 2002). The obtained positions of the detected particles are illustrated in Figure 2.9 (b). A detailed explanation and justification of the procedure can be found in Jähne (1995) and Capart et al. (2002). More details regarding image processing techniques can be found in Gonzalez and Woods (1993) and Russ (2011).

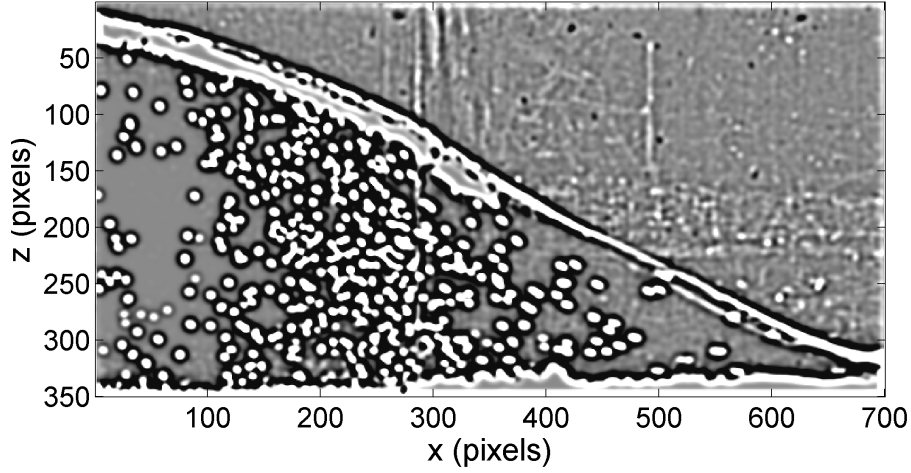


(a) Raw Image for time instant ($T = 0.99$) ($t = 0.18$ s).

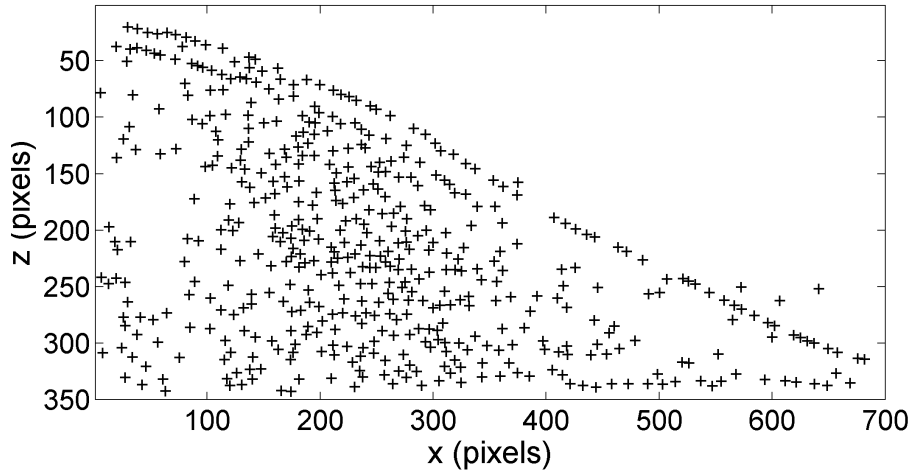


(b) Smoothed image obtained after the application of a low-pass filter to the raw image.

Figure 2.8: The two first steps in the particle detection process: raw image and application of a low pass filter.



(a) Peak enhancement result.



(b) Detected particles coordinates (in pixels).

Figure 2.9: The process of particle detection continues by the application of a Laplace filter in order to enhance the particle's centres, originating an intensity distribution (a). A polynomial fit is then applied to obtained intensity distribution resulting in the determination of the particle's centre coordinates with subpixel resolution.

2.2.1.2 Particle Tracking Schemes - The Voronoï Tracking Algorithm

After the identification of the particles position in the flow images, each individual particle has to be tracked from one image to the other. There are several ways of tracking particles between each frame (e.g. Ohmi and Li (2000)). One common method is the least displacement method (Hassan and Canaan, 1991), here briefly explained.

Let I_1 and I_2 be two consecutive frames of the flow. Let $P_{n,1}$ be the n -th particle in frame I_1 and $P_{m,2}$ the m -th particle in frame I_2 whose position vectors are given respectively by $\mathbf{r}_{n,1}$ and $\mathbf{r}_{m,2}$. Assuming a small time interval between consecutive images it is reasonable to assume that the particles displacement is small. This fact is the basis of the least displacement algorithm, which searches the nearest particle within a given search area, S , of the foreseen particle location as illustrated in Figure 2.10. This method has some disadvantages, such as it is not suitable when dealing with high concentration flows because there will be too many particles inside the search area to make a unique choice. Another limitation arises when dealing with intense flow gradients since these can lead to important particles displacement in a short length scale. To overcome these problems (Capart et al., 2002) proposed a tracking approach based on the Voronoï tessellation.

The Voronoï method developed by (Capart et al., 2002) for dense granular flows is an alternative way of implementing the second (tracking) step. It consists in creating a Voronoï tessellation making from each detected particle centre the respective centre of each Voronoï cell. The Voronoï diagram is a geometrical construction that divides the space in cells V_n surrounding each feature point P_n . These feature points being the positions (x_n, y_n) of the detected particles in the present case. Each Voronoï cell encloses

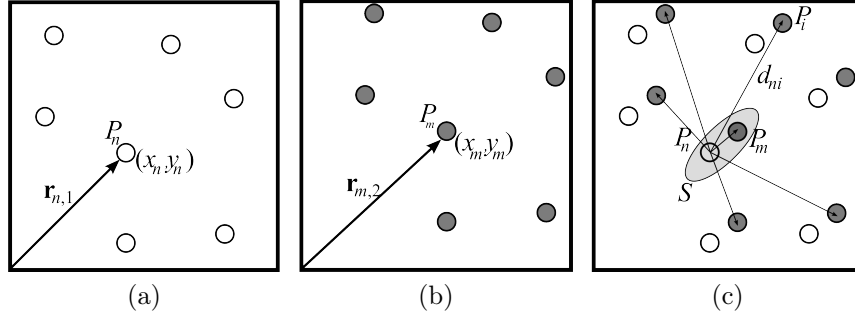
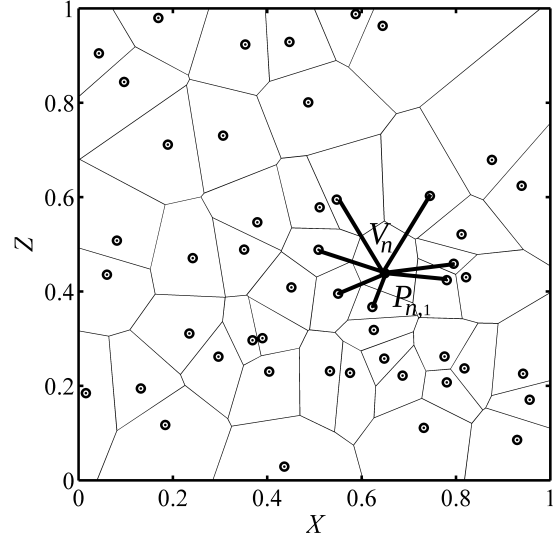


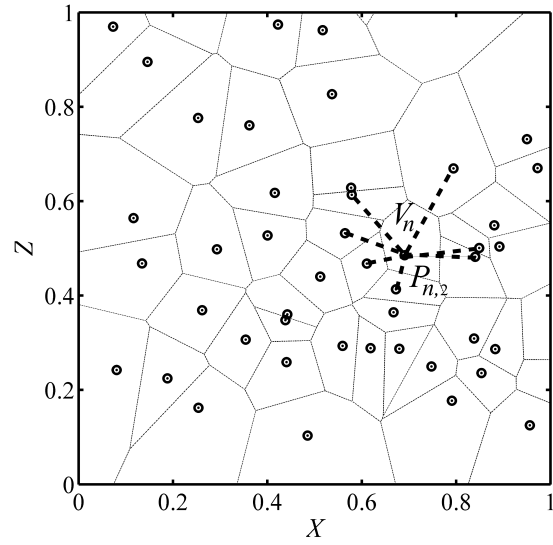
Figure 2.10: The least displacement algorithm is here illustrated. In (a) a flow region is image at time t and a generic particle P_n is identified at the coordinates (x_n, y_n) . (b) depicts the same flow region at $t + dt$ and a generic particle, P_m is also identified at coordinates (x_m, y_m) . (c) is the superposition of both images taken at time t and $t + dt$. A search area S is drawn around particle P_n and the closest particle of (x_n, y_n) found inside S is considered the matched particle.

the region of the plane that is closer to P_n than to any other point in the plane. The cells who share a common edge are referred to as natural neighbours. The Voronoï diagram defines a local pattern of neighbouring particles that can be used as match templates (Ahuja, 1982; Song et al., 1999). For a moving dispersion of particles, these patterns remain stable over a certain time (Okabe et al., 1992). This stability property of Voronoï diagram is used in the present application to detect matching particles from one image to the other.

In each Voronoï diagram, the cell centres are connected to the centre of the neighbour cells, defining a so-called Voronoï star. Such a Voronoï star is shown for particle P_n in Figure 2.11 (a) for instant t and in Figure 2.11 (b) for $t + dt$. Instead of tracking a single particle the goal is now to track Voronoï stars between frames, as the Voronoï star pattern was found still more stable than Voronoï polygons (Capart et al., 2002). In Figure 2.12 (a) two corresponding



(a)



(b)

Figure 2.11: The Voronoi diagram and a star obtained for a particle, P_n distribution at instants (a) t and (b) $t + dt$.

Voronoi stars obtained for t and $t + dt$ (Figure 2.12) are shown.

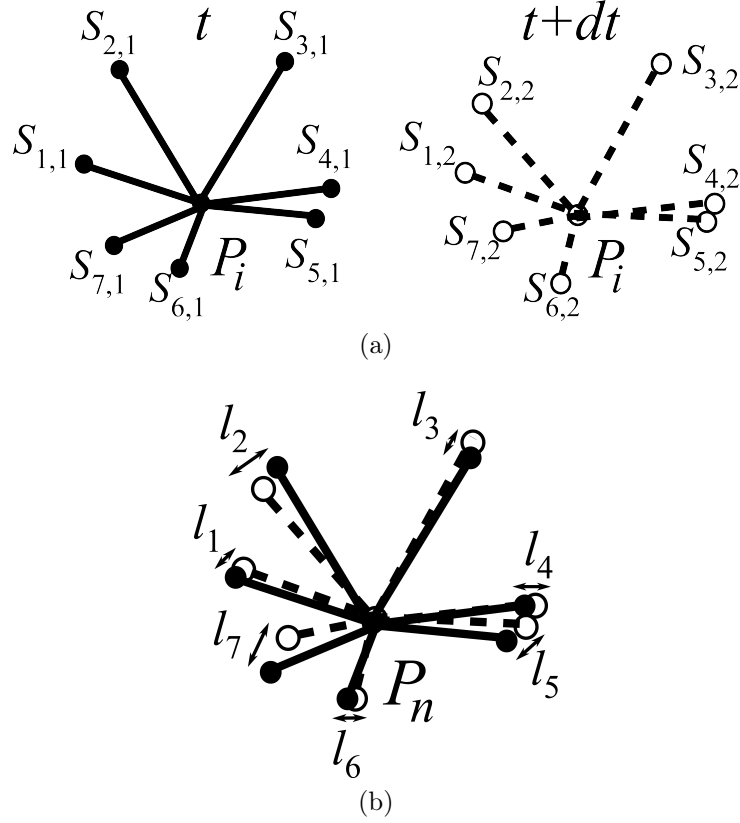


Figure 2.12: (a) Two consecutive Voronoi stars, t and $t + dt$ with centre in P_n (b) The overlapping of the centre particles and determination of the different distances between the stars' extremities, l_i ($i = 1, 2, \dots$) is here represented.

The *star-distance* is a measurement of the degree of discrepancy between the patterns formed by the two stars. As illustrated in Figure 2.12 (b), the star distance between two particles is defined as the median of the distances between the star extremities, $l_1, l_2, \dots$ once these two particles have been made to coincide.

$$\text{match}(P_{n,1}) = \min(\text{dist}_S(S_{n,1}, S_{m,2})) \quad (2.4)$$

where $\text{dist}_S(S_{n,1}, S_{m,2})$ is the *star-distance* that measures the degree of discrepancy between the patterns formed by the two stars. As an illustration, the matching algorithm is applied to the highlighted regions A and B in Figure 2.13, showing a picture of the dam break flow at $T = 0.99$.

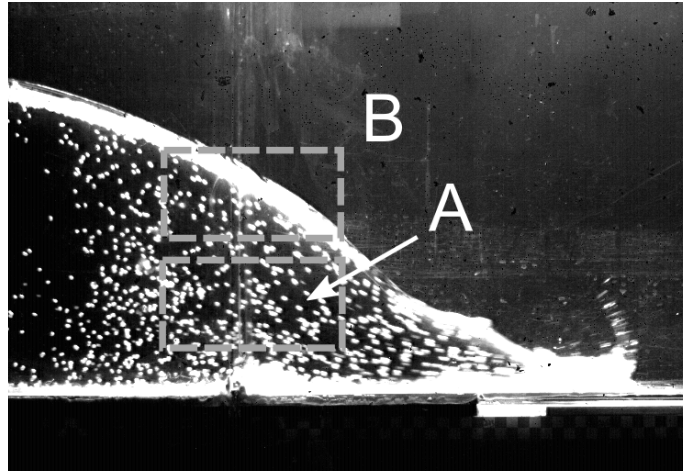


Figure 2.13: Flow image of the dam-break flow taken at $T = 0.99$.

These two regions were chosen to illustrate the ability of the method to handle the presence of flow boundaries as well as wide ranges of particle concentrations. The Voronoï partition around the particles detected at $T = 0.99$ and in the next image at $T = 1$ are shown in Figure 2.14. The stability of the Voronoï patterns can be observed and it is possible to observe the displacement of the Voronoï cells. It is possible to see in Figure 2.14 (b) how some Voronoï polygons are extended to infinity, corresponding in this case to the image edges, due to the presence of the liquid-air interface. The local characteristic of the geometric construction is such that the polygons on the interface will not affect the other polygons.

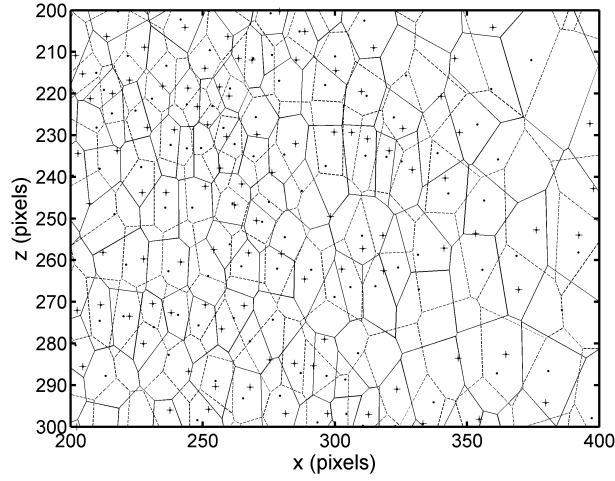
The stability of the Voronoï diagrams assures that, even if one

of the particles leave the focus plane, due, for instance, to an important flow gradient; the algorithm will still work. On the other hand having a small time interval between consecutive images it's important to, in a case of a 3D flow, avoid the loss of too many particles in the focus plane.

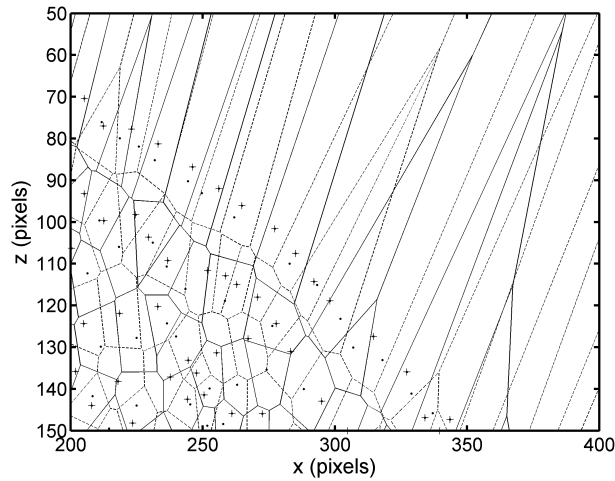
2.2.1.3 Data Structuring of Randomly Dispersed Data

In order to suffice in a quantitative analysis, the measurements performed on sets of randomly distributed particles will have to be organised on a regular grid. Moreover, in order to ensure a sufficient spatial distribution of the measurement points, i.e. the particles in the flow, it is necessary to enhance the data set by repeating each experiment several times. Indeed, it can be observed from the raw images that a uniform distribution of the particles is not always obtained. A necessary condition for the enhancement of datasets by combining several experiments is the repeatability of the experiments. For hydrodynamic dam-break flows, this was already demonstrated in previous studies (Soares-Frazão and Zech, 2008).

In order to further enhance the dataset at a given time, it is also possible to perform a time-averaging of the data over a given time interval Δt . Both types of data sets enhancements, by time averaging and repeatability, are considered here. To derive the vertical velocity profiles from the measurements, the randomly distributed data has to be re-sampled on a structured grid. To pass from a non-structured grid to a structured one several methods exist, (Spedding and Rignot, 1993; Tropea et al., 2007). Usually, the structured grid is obtained by data interpolation. In this work a data binning method is proposed. This method consists in dividing each image in a $m \times n$ grid and averaging the velocity values in each bin (Figure 2.15). Care must however be taken in the choice



(a)



(b)

Figure 2.14: Voronoï diagrams for: (a) region A and (b) region B of Figure 2.13 respectively. Straight lines for $T = 0.99$, dashed lines for $T = 1$. Coordinates are in pixels.

of the bin dimensions. A large bin will filter local features of the flow while a small bin may not contain particles at all.

With the binning method, it is possible to devise a filtering

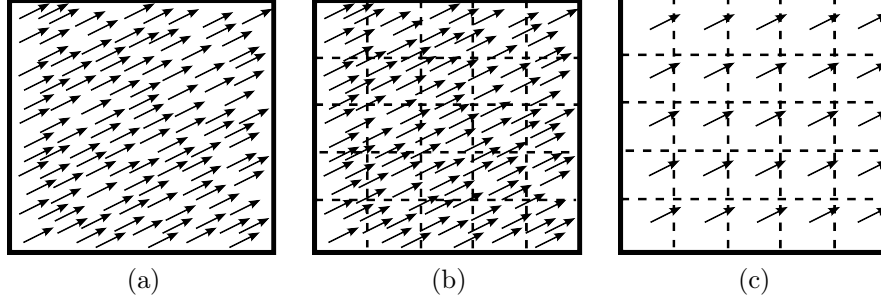


Figure 2.15: The data binning procedure used to structure the PTV velocity data is here illustrated. The binning procedure consists in (a) taking a raw velocity field, where the velocity vectors are randomly dispersed and (b) divide that raw velocity field into a grid of $m \times n$ bins. Finally an average of the velocity vectors inside each bin is computed.

scheme for removing outliers, bin by bin. This scheme is implemented in the following procedure, illustrated in Figure 2.16 for a unitary square bin. Considering a bin i , an initial average velocity, μ_{bu} , μ_{bw} and standard-deviation, σ_{bu} and σ_{bw} are calculated for each velocity component u and w respectively. During the second step all of those vectors included in that bin are compared with the calculated average. A vector is rejected if one of its components falls out of the interval $[\mu_{bj} - k\sigma_{bj}, \mu_{bj} + k\sigma_{bj}]$, with $j = u, w$ and k is a constant, to be defined by the user, as proposed by Chauvenet (1863). The typical value used was $k = 2$, the reason for this relies on the fact that if one assumes a Gaussian distribution of the velocity values, the interval $[\mu_{bj} - 2\sigma_{bj}, \mu_{bj} + 2\sigma_{bj}]$ will include 95.45% of all the values (e.g. Holman (2007)). After this filtering operation a new average and a new standard-deviation are computed. Lastly, after the binning operation, the divergence of the obtained vector field is computed to check again for outliers. The binning method can be seen as a zero-order interpolation scheme. Interpolating the obtained velocity values for a predefined structured grid

using an adequate interpolation scheme (e.g. bilinear, triangular, polynomial, etc. Maron (1973); Watson and Phillip (1984)) is another way of structuring the data. This idea is illustrated in Figure 2.17. A comparison between bilinear interpolation and binning is presented in Annex B.

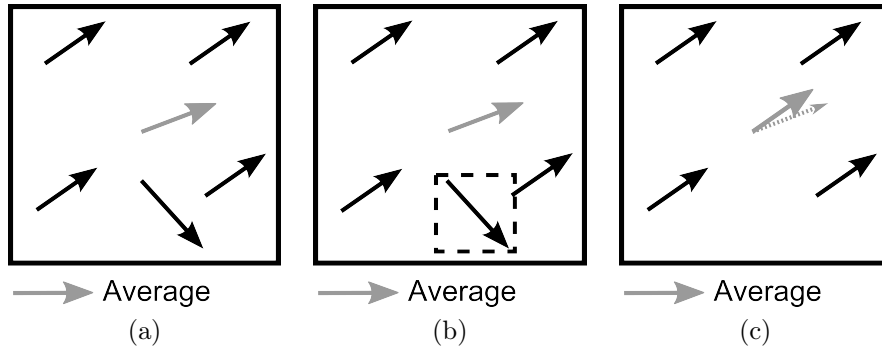


Figure 2.16: The outlier removal procedure is illustrated for a single bin. (a) A bin with some vectors and a calculated average (b) Identification of a spurious vector (inside dashed rectangle) which will be removed. (c) After the removal of the spurious vector a new average is computed. Dashed line vector is the initial average.

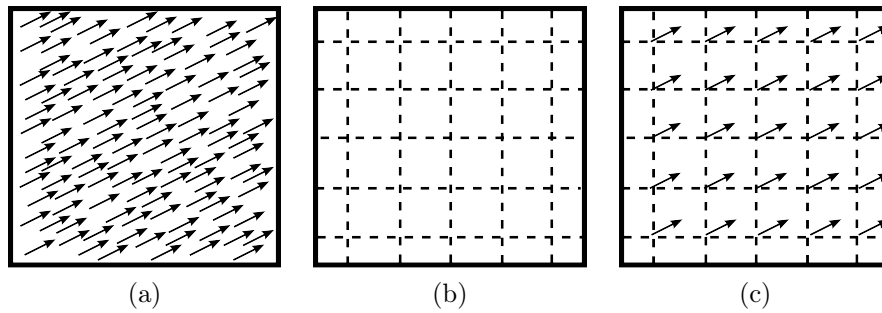


Figure 2.17: The interpolation procedure for data structuring is here presented. (a) a non-structured velocity field is plotted. (b) A structured grid is considered and the velocity is interpolated for the grid points given by the intersections of the grid lines (c) (see Annex B).

2.2.1.4 The Bin Size Effect

An investigation is made to find the influence of the bin size (Δx , Δz) in the velocity profile. As referred in subsection 2.2.1.3, if the bin dimensions are too small there is a risk of not having enough data in each bin and if the bin dimensions are too large it is possible that local features of the flow be filtered out.

An example is here presented: the images chosen for the analysis correspond to the condition defined by $h_0 = 0.325\text{m}$ at $T = 5$, as depicted in Figure 2.18. Time averages were made over 11 images, i.e over a time interval $\Delta t = 0.11\text{s}$. For the same set of images the structured grid is obtained with different bin widths, Δx and heights Δz in the range 5, 10, 20 30 and 40 pixels. Four of the grids are represented in Figure 2.19.

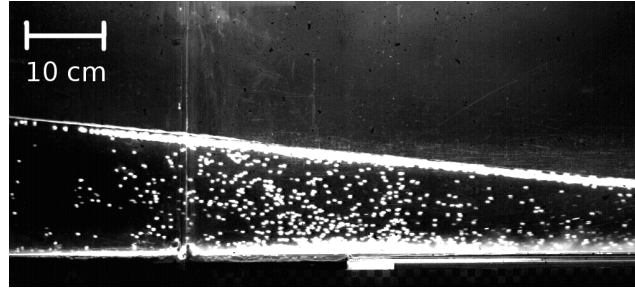


Figure 2.18: Raw flow image for $T = 5$. To notice that at this stage the water surface can be approximated by a straight line.

The first step consisted in investigate the number of non-empty bins, as a function of the bin area width $\times$ height. The void fraction N_b is defined by the ratio between bins without data over the total number of bins:

$$N_b = \frac{B_d}{B_{all}} \quad (2.5)$$

where B_d is the number of bins without data on it, and B_{all} is

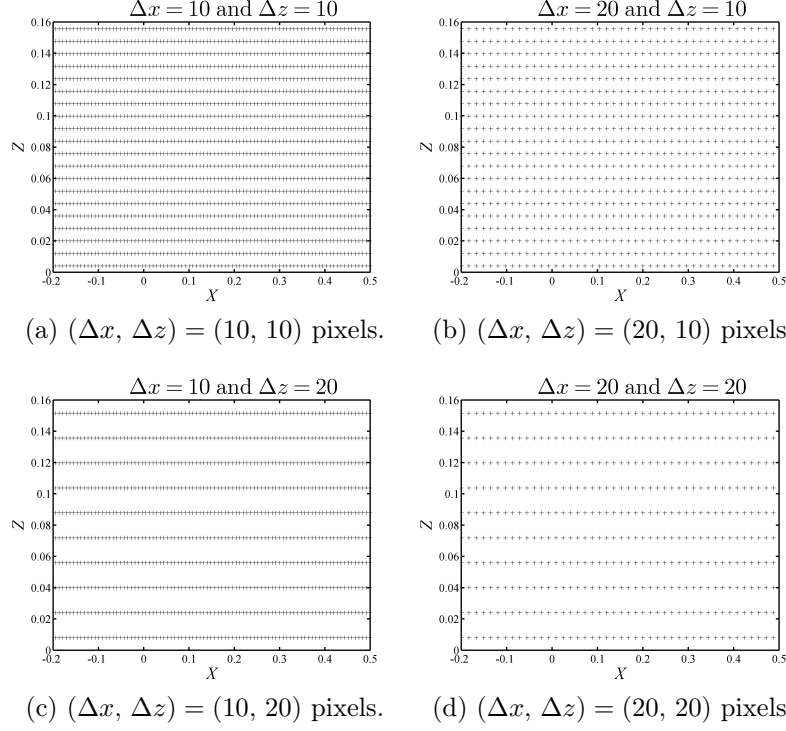


Figure 2.19: Some of the different grids used in the PTV data post-processing and their spacing in pixels, only the bin centres are represented. The values of $(\Delta x, \Delta z)$ refer to each grid spacing.

the total number of bins inside the flow region. The results for the percentage of void bins are expressed in Table 2.5. As it would be expected the void fraction decreases with the increasing bin area, this is also stressed in Figure 2.20.

Defining the bin dimensions in pixels is not necessary a problem. It is more interesting to find a general way to define the bin dimensions needed to obtain a good description of the velocity field. One way to do it is defining the bin dimensions to make sure that between a time step, some of the initial particles still remain inside the bin. This is inspired from the PIV "one-quarter" rule which dictates that the particle displacement should be no greater than

Table 2.5: Void bins fraction N_b for different bin dimensions in pixels.

$\Delta x \backslash \Delta z$	5	10	20	30	40
5	0.675	0.539	0.341	0.231	0.175
10	0.262	0.187	0.117	0.091	0.069
20	0.073	0.059	0.043	0.041	0.032
30	0.029	0.027	0.017	0.021	0.013
40	0.022	0.019	0.014	0.015	0.010

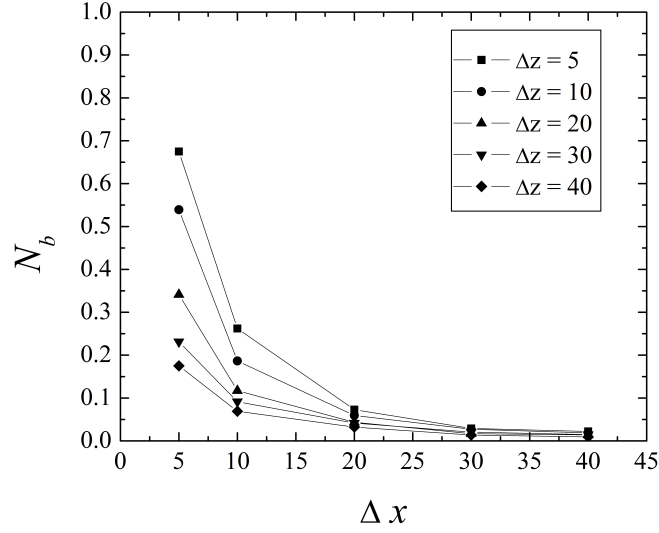


Figure 2.20: Evolution of the void fraction with increasing bin width, Δx and bin height Δz (in pixels).

one quarter of the minimum interrogation window size (Keane and Adrian, 1990).

With this hypothesis the bin size can be adapted as a function of the particles' displacement. This criterion can be restrictive for binning since the goal is not to perform a cross correlation operation. Aleixo et al. (2010) presented a study where the following

expression is used to estimate the minimum bin size Δx , Δz :

$$\Delta x = n \cdot u \cdot dt \quad (2.6)$$

$$\Delta z = n \cdot w \cdot dt \quad (2.7)$$

where u and w are the typical horizontal and vertical velocity scales of the flow (in pixels per time unit as taken from the displacement observed on the flow images), dt is the time interval between consecutive frames and n is an integer. To define n those authors made a comparison between the PIV results with the structured PTV results for different values of $n = 1, 2, 3$ and they have proposed $n = 2$. For the case of $T = 5$, where $u \cdot dt \approx 15$ pixels, then:

$$\Delta x \approx 30 \text{ pixels} \quad (2.8)$$

Whereas for the bin height, the typical vertical displacement was $w \cdot dt \approx 3$ pixels and so:

$$\Delta z \approx 6 \text{ pixels} \quad (2.9)$$

From table 2.5 one can see that for the combination $(\Delta x, \Delta z) = (30, 5)$ one has $N_b = 0.029$, and with $(\Delta x, \Delta z) = (30, 10)$, $N_b = 0.027$. This last pair of values was used for the data processing since it allows for the higher vertical displacements registered in the instants immediately after the dam removal.

For a more efficient bin size definition the expression (2.6) should be used as first estimative and then be complemented with an analysis of the non-void bins fraction (eq. 2.5). Expression (2.6) can be used as a guidance for the measurements parameters, namely to estimate the time interval between images to obtain a determined

bin size in the post-processing process.

In Figures 2.21 to 2.24 the effect of the bin size in the velocity profile for $X = 0$ is presented for the grids depicted in Figure 2.19.

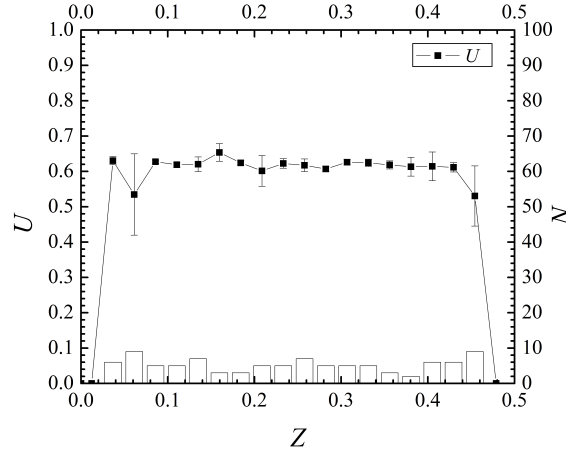


Figure 2.21: Velocity profiles of the horizontal component, U , at the gate section, $X = 0$, in function of the bin size for $\Delta x = 10$ pixels and $\Delta z = 10$ pixels. N denotes the number of samples per bin (right vertical axis) and the error bars are the calculated standard-deviation values.

As expected, by increasing the bin size one loses spatial resolution, while reducing the bin size one increases the spatial resolution but reducing the number of samples per bin. Two important features must be noted. First, since the dam-break flow is, as it was seen, an example of a severe transient flow, the velocity scales will change in time and also the bin dimensions, according to equation (2.6). Secondly, for analysis of the boundary-layer evolution measuring velocity profiles with enough spatial resolution is important and that can be hard to achieve with imaging methods. The limitations to the spatial resolution arise from a) the particle's dimensions, and b) because of the *flocking* of particles as seen near the channel's bed in Figure 2.18. This phenomenon makes individ-

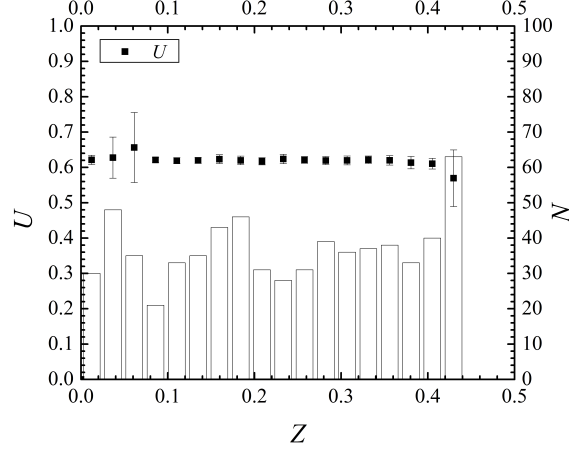


Figure 2.22: Velocity profiles of the horizontal component, U , at the gate section, $X = 0$, in function of the bin size for $\Delta x = 30$ pixels and $\Delta z = 10$ pixels. N denotes the number of samples per bin (right vertical axis). The error bars are the calculated standard-deviation values.

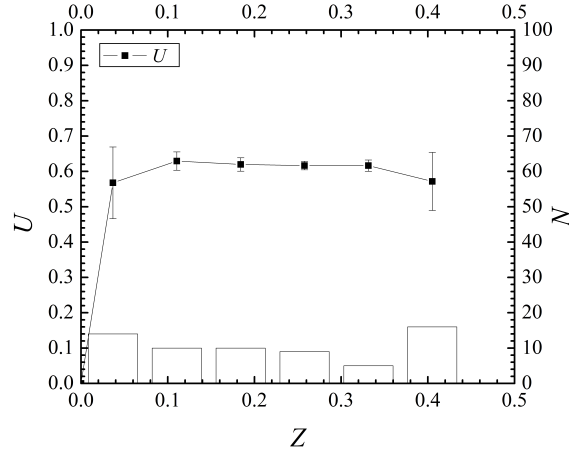


Figure 2.23: Velocity profiles of the horizontal component, U , at the gate section, $X = 0$, in function of the bin size for $\Delta x = 10$ pixels and $\Delta z = 30$ pixels. N denotes the number of samples per bin (right vertical axis). The error bars are the calculated standard-deviation values.

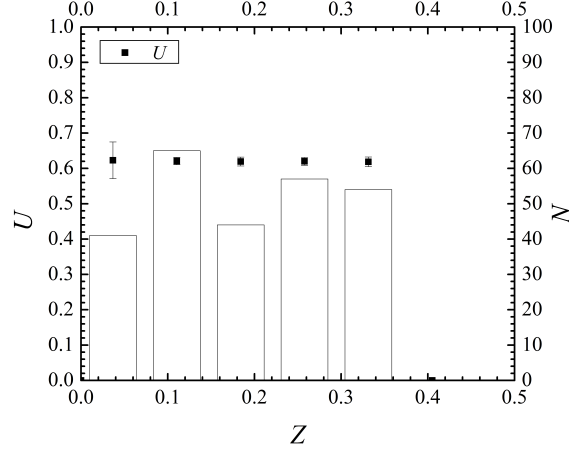


Figure 2.24: Velocity profiles of the horizontal component, U , at the gate section, $X = 0$, in function of the bin size for $\Delta x = 30$ pixels and $\Delta z = 30$ pixels. N denotes the number of samples per bin (right vertical axis). The error bars are the calculated standard-deviation values.

ual particle identification impossible leading to a loss of resolution in the near bed region.

2.2.1.5 An Application Example

A step-by-step analysis is presented here in order to make the previous points clearer. The dam-break flow at $T = 5$ is analysed as example, for which raw images corresponding to the lower limit, the centre and the upper limit of of the time interval $[T - 5\Delta T, T + 5\Delta T] = [4.73, 5.27]$ are depicted in Figure 2.25.

In Figure 2.26 the obtained raw velocity field for the 11 images of the considered time interval: for one run is presented; along with the position of the free surface. For structuring the velocity field the binning method proposed was used. The dimension of the bins was calculated by means of equations (2.6) and (2.7) corresponding to a bin size of $(30 \times 10 \text{ pixels})$ over which the data will be structured.

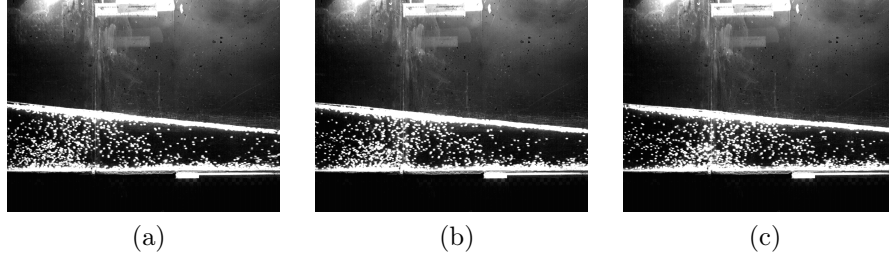


Figure 2.25: Flow images of the dam-break flow taken at (a) $T = 4.73$, (b) $T = 5$ and (c) $T = 5.27$.

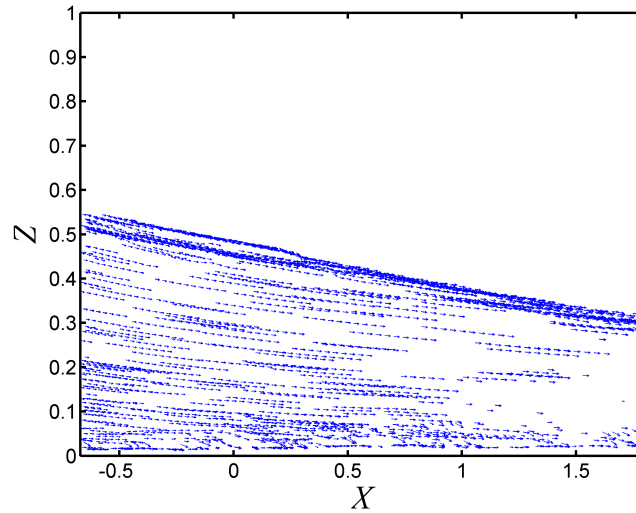


Figure 2.26: The non-structured velocity field obtained with the Voronoï algorithm described above, for $T = 5$. ($t = 0.9$ s).

Figure 2.27 indicates the distribution of correct detected and tracked particles. For this case the maximum was found to be 46 vectors (close to the free surface). In the core region of the flow, i.e., away from the free surface and from the bed, the maximum was found to be 23 detected vectors. In this step a first filter is applied consisting in removing from the data set the spurious par-

ticles detected above the free surface. This spurious particles are often caused by light reflections on the channel window.

The non-dimensional standard deviation was computed in each bin for both horizontal and vertical velocity components, given respectively by:

$$\sigma_U = \frac{1}{c_0} \sqrt{\sum_{i=1}^N \frac{(u_i - \bar{u})^2}{N-1}} \quad (2.10)$$

$$\sigma_W = \frac{1}{c_0} \sqrt{\sum_{i=1}^N \frac{(w_i - \bar{w})^2}{N-1}} \quad (2.11)$$

where $\bar{u}$ and $\bar{w}$ are the mean values of u and w in each bin. The non-dimensional standard-deviation for both components, u and w is illustrated in Figures 2.28 and 2.29 respectively. It is possible to see that the measured value for the standard deviation of the horizontal component is usually less than $0.05c_0$, except for a small region at $X \approx 1.5$ where the lack of particles (see Figure 2.27) does not allow a correct determination of the standard deviation. After the calculation of the standard-deviation the Chauvenet (1863) criterion is applied.

The non-dimensional divergence, $t_0 \nabla \cdot \mathbf{v}$ field is also computed, by means of centred finite difference scheme and presented in Figure 2.30. As it can be seen the measured divergence is close to zero, and the maximum absolute value computed was $|t_0 \nabla \cdot \mathbf{v}| = 4.85 \times 10^{-4}$. Then, in Figure 2.31 the obtained structured velocity flow field is computed.

Finally the same procedure is applied to the three runs. To overlap the results from the three runs the overlap of the images is made by using as reference the gate section. A mean water height is computed and used to filter the spurious particles who were detected above the free surface. The complete raw velocity field is

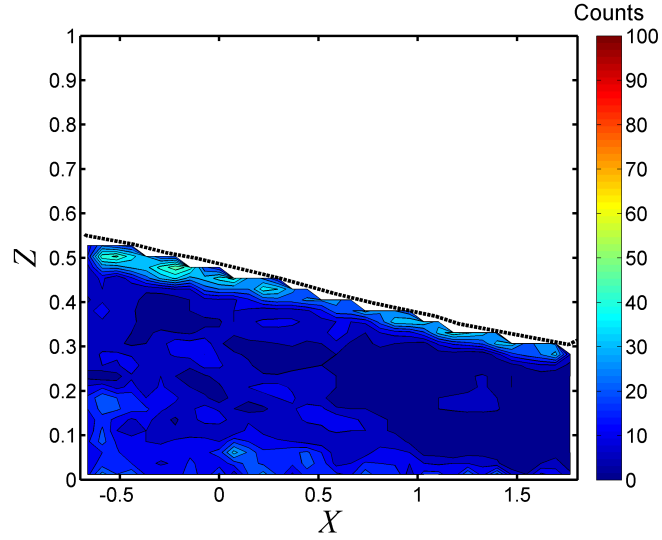


Figure 2.27: Distribution of the correct detected particles in the flow region for one run at $T = 5$.

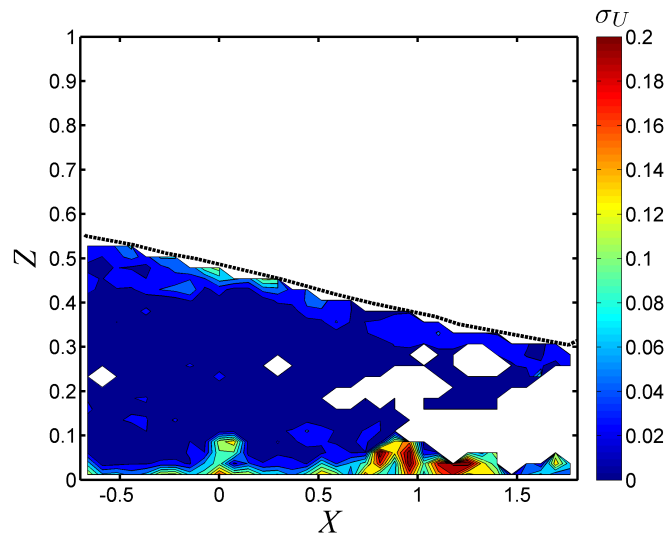


Figure 2.28: Standard deviation distribution of the horizontal component measured for one run at $T = 5$.

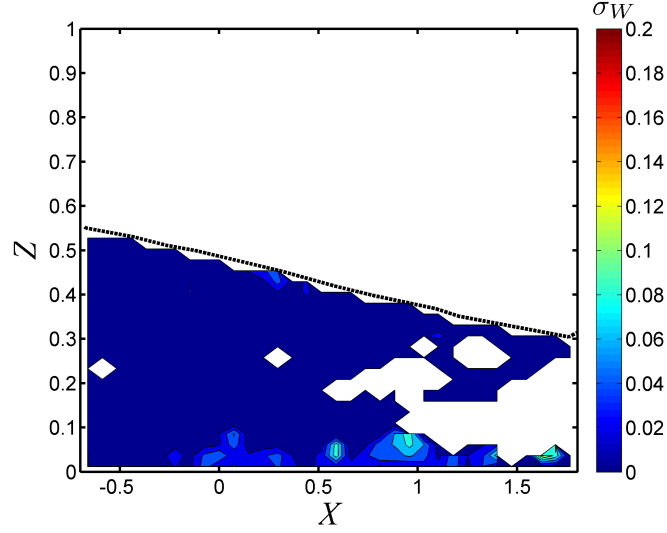


Figure 2.29: Standard deviation distribution of the vertical component measured for one run at $T = 5$.

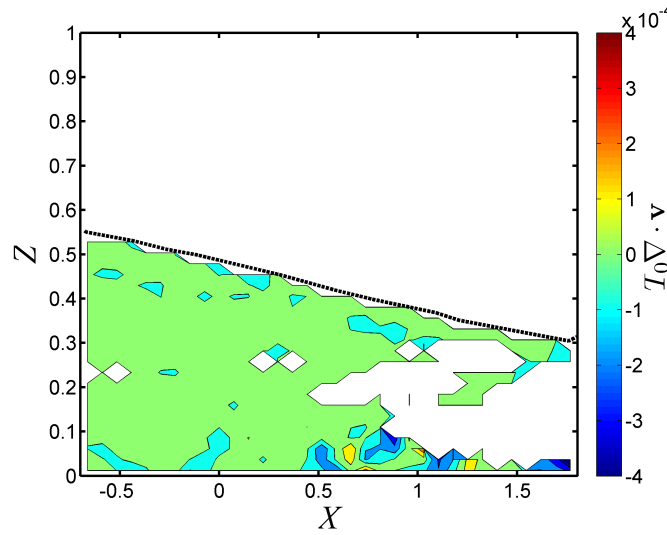


Figure 2.30: Non-dimensional divergence $t_0 \nabla \cdot \mathbf{v}$, calculated for one run at $T = 5$. The lack of particles is the reason for the area without information.

depicted in Figure 2.32, where the vector density is much higher than the one of Figure 2.26. The total number of successfully detected particles is presented in Figure 2.33, and as expected, it is now much higher than for the single run case. It is possible to see from the presented analysis (Figure 2.26 and Figure 2.32) that the proposed method of considering different runs allows to fill in the "empty bins", and to increase the number of vectors per bin.

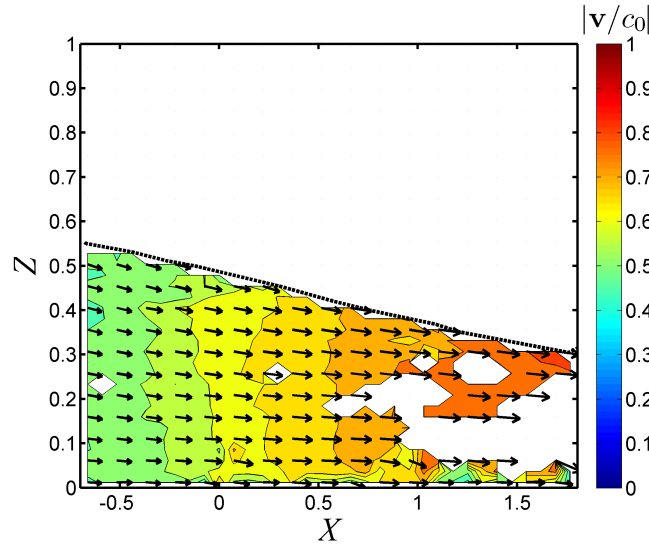


Figure 2.31: Structured measured flow velocity field at $T = 5$ (only 1 out of 2 vectors depicted).

Figures 2.34 and 2.35 depict the calculated standard deviations for both horizontal and vertical components respectively. The respective divergence field is computed and presented in Figure 2.36, again with values close to zero. Finally the computed average is plotted in Figure 2.37. The use of different runs to increase the data set, usually referred as ensemble-average method will be further developed in Chapter 4, where different runs will be used to determine flow statistics. This strategy can be applied to the dam-break flow since, as it was shown by Soares-Frazão and Zech (2008)

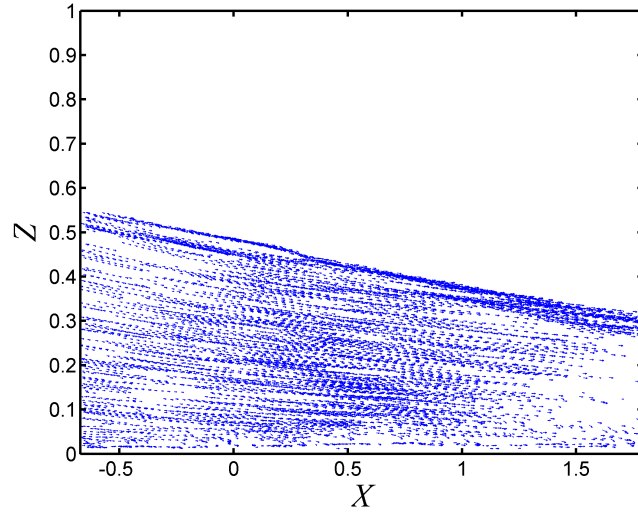


Figure 2.32: Non-structured measured flow field at $T = 5$. The high vector density is obtained by the superposition of the vectors from the three runs.

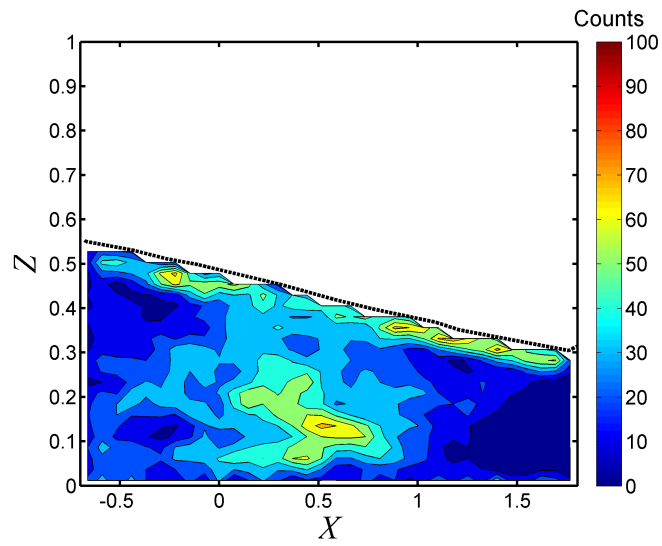


Figure 2.33: Distribution of the total detected particles considering the three runs.

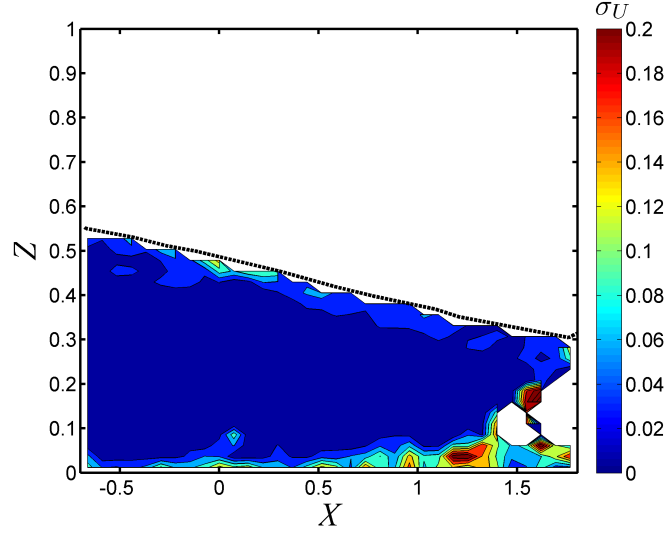


Figure 2.34: The ensemble standard deviation for the horizontal component, σ_U , for $T = 5$.

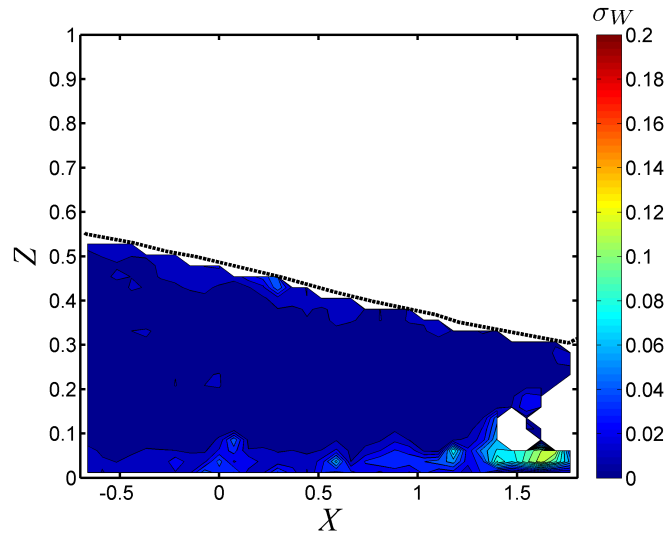


Figure 2.35: The ensemble standard deviation for the vertical component, σ_W , for $T = 5$.

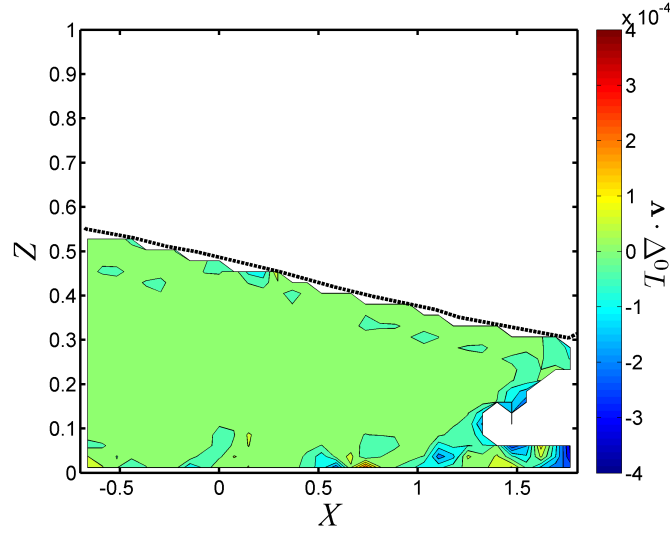


Figure 2.36: The computed non-dimensional divergence, $t_0 \nabla \cdot \mathbf{v}$, for the three runs.

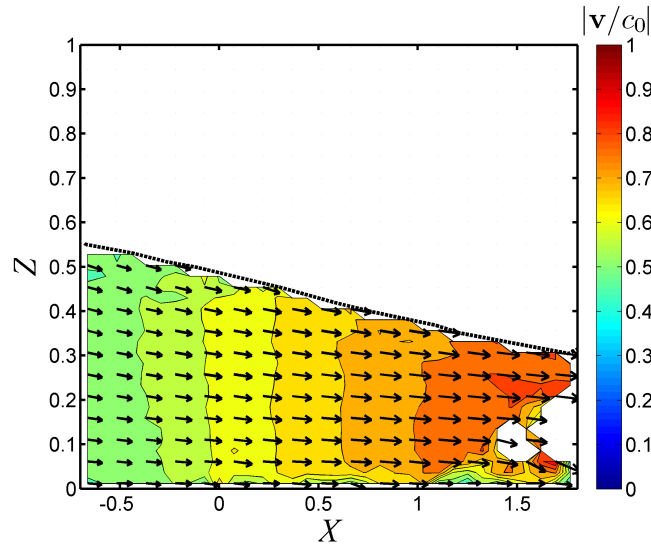


Figure 2.37: The ensemble averaged velocity field for $T = 5$.

the dam-break flow is quite repeatable, and thus it becomes possible to consider ensemble averages.

2.2.2 Particle Image Velocimetry (PIV)

Particle Image Velocimetry (PIV) is another method to extract the velocity field from a sequence of seeding particles' images. Contrary to PTV where individual particles are identified and tracked, PIV uses image cross-correlation to track the displacement of patterns of particles. The main advantage of this method when compared to PTV is a lesser computing time demand when the seeding concentration is high.

Since the works of Meynart (1980, 1982, 1983), one of the pioneers in using flow images to extract the velocity field, to the work of Keane and Adrian (1992), who proposed the cross-correlation of images, the Particle Image Velocimetry has long evolved and it is nowadays an established and mature measurement technique. A historical overview of the developments of the PIV technique is presented in Adrian (2005) and a complete description and revue can be found in Raffel et al. (2007); Tropea et al. (2007); Adrian and Westerweel (2011). This technique allows the measurement of the instantaneous flow velocity field based on the cross-correlation of two flow images obtained over a short time period dt .

A brief introduction to the PIV technique is here presented. For more details and insights the reader is here advised to read the books by Raffel et al. (2007), Tropea et al. (2007) and Adrian and Westerweel (2011) that present a complete and detailed description of Particle Image Velocimetry.

Let one consider a double-pulsed laser which, by means of an appropriated diverging optics, produces a light sheet (e.g. Hecht (2001)) illuminating a section of a flow previously seeded with particles, as depicted in Figure 2.38. Consider also a camera that is

synchronized with the laser, acquiring one image for each of the two laser pulses, I_1 and I_2 . The obtained consecutive images, I_1 and I_2 , are then divided in smaller areas designated as interrogation windows. The interrogation windows of the first image I_1 are correlated with the corresponding interrogation windows of the second image I_2 (Keane and Adrian, 1992) as depicted in Figure 2.43.

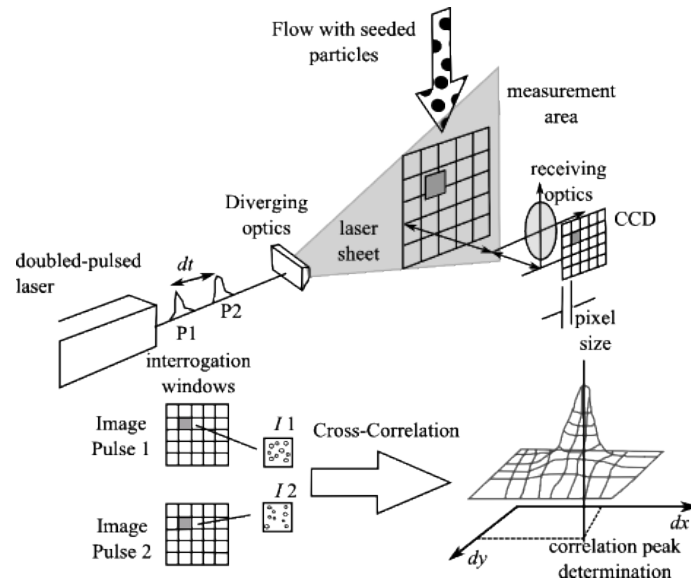


Figure 2.38: Scheme of the PIV technique. A double pulsed laser generates two laser pulses, P_1 and P_2 . The laser pulse passes through a diverging optics system and a laser light sheet is generated. The laser light sheet will light a section of the particle seeded flow. The light reflected by the seeding particles will be collected by a optical lenses and focused in a photo-sensitive element, a CCD captor, for example. Each pulse will generate an image. These images are then divided in interrogation windows and by cross-correlation the correlation peak is determined. Adapted from www.dantecdynamics.com.

The discrete cross-correlation between the interrogation windows, w_1 and w_2 from two consecutive frames, I_1 and I_2 can be defined as (e.g. Adrian and Westerweel (2011); Sveen and Cowen

(2004)):

$$R(sx, sz) = \frac{1}{N^2} \sum_{i=0}^{N-1} \sum_{j=0}^{N-1} I1_{w_1, w_2}(i, j) I2_{w_1, w_2}(i + sx, j + sz) \quad (2.12)$$

where (i, j) is the pixel location within interrogation windows w_1 and w_2 and (sx, sz) denotes the spatial lag at which the cross-correlation is being computed. By using the Discrete Fourier Transform (e.g. Cooley and Tukey (1965)), equation (2.12) can be written as:

$$R(sx, sz) = \mathcal{F}^{-1} \{ \mathcal{F} \{ I1_{w_1, w_2}(i, j) \}^* \mathcal{F} \{ I2_{w_1, w_2}(i + sx, j + sz) \} \} \quad (2.13)$$

where $\mathcal{F}$ denotes the Fourier transform, $\mathcal{F}^{-1}$ the inverse Fourier transform, and the $*$ represents the complex conjugate. The main reason for applying the Fourier transform is related with computing efficiency. It can be proven that it is computationally more economical to compute the correlation by means of equation (2.13) than by direct application of equation (2.12). The function $R(sx, sz)$ will have a maximum when the lag (sx, sz) coincides with the displacement vector $\mathbf{dx} = (dx, dz)$. In Figure 2.39 a simple scheme of the window correlation process is presented and the correlation operation is illustrated. The reader is referred to the Figure legend for more detailed information.

A practical example is presented in Figure 2.40 in which a PIV image is depicted. For the indicated interrogation window, the cross-correlation is calculated and depicted in Figure 2.40.

The position of the correlation peak relative to the correlation plane will be the value of the most probable displacement of the

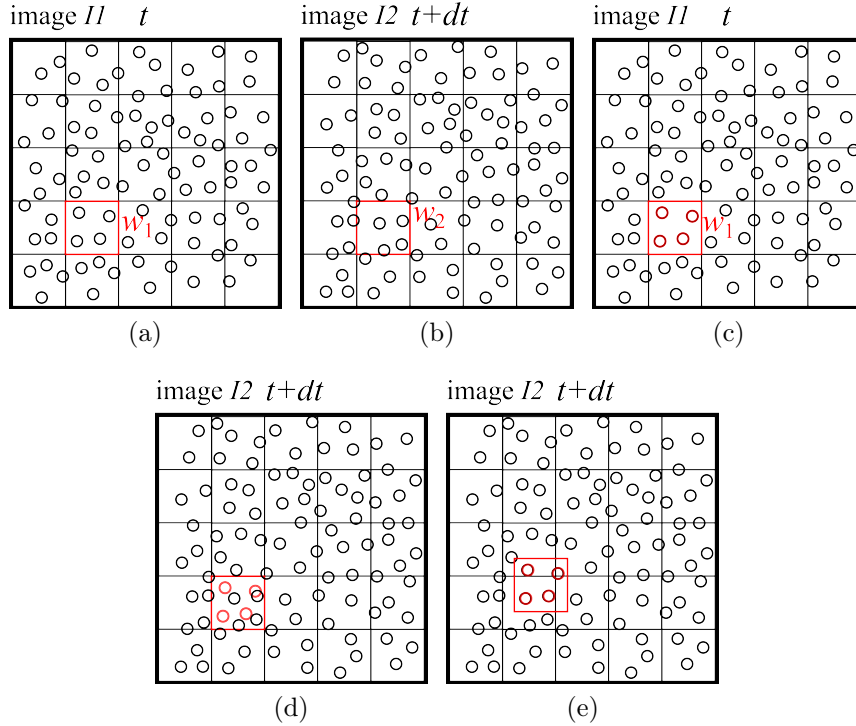


Figure 2.39: A simple scheme to illustrate the principle of cross correlation between interrogation windows of consecutive frames $I1$ and $I2$. (a) An image $I1$ at time t where the interrogation window w_1 is located. (b) Image $I2$ at time $t + dt$ with the corresponding interrogation window w_2 . (c) A template of particles (red) inside interrogation window w_1 is chosen. (d) The chosen pattern is then cross-correlated with the interrogation window w_2 and its neighbourhood in order to find the corresponding pattern in the particles template w_1 with w_2 . (e) Identification of the template in image $I2$ for correlation peak displacement.

particles' ensemble of each interrogation window. Thus, having the displacement vector and knowing the time interval between laser pulses, the velocity vector, $\mathbf{v}_i$, for each interrogation window, i , is obtained as:

$$\mathbf{v}_i = \frac{d\mathbf{x}_i}{dt} \quad (2.14)$$

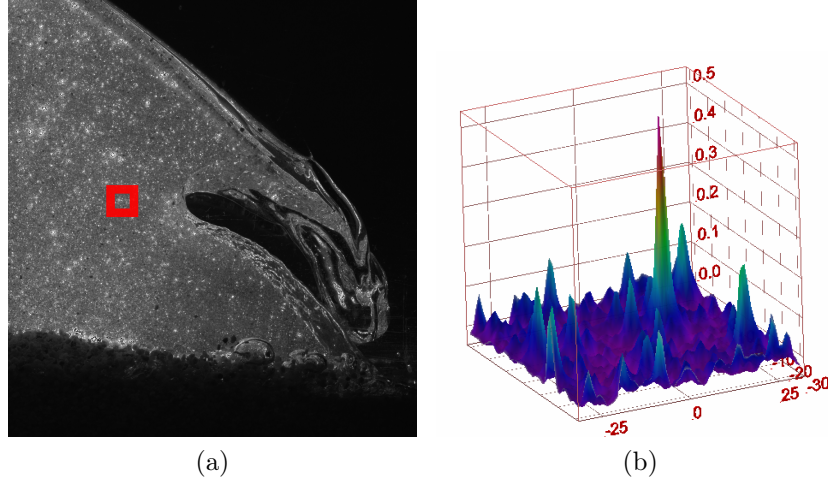


Figure 2.40: A practical example of a correlation peak obtained. (a) raw dam break image at time $t + dt$ and a initial interrogation window. (b) Cross-correlation peak in the red interrogation window.

where $\mathbf{dx}_i$ is the displacement vector of the correlation peak and dt is the time interval between pulses.

Using just one camera for the image acquisition, PIV allows the determination of two of the components of the velocity vector in the light sheet plane. Using two cameras it is possible to extend it to 3 components in a plane by means of a stereoscopic arrangement of the cameras as proposed by Theodor Scheimpflug (Scheimpflug, 1904). Nowadays it is even possible to perform Tomographic-PIV, that is measuring the velocity field in a volume of fluid and with time resolution (e.g. Scarano et al. (2005), Elsinga et al. (2006), Scarano and Poelma (2009), Adrian and Westerweel (2011)). A crucial point in the accuracy of the PIV technique is the determination of the cross correlation peak position. In modern PIV algorithms the detection of the correlation peak position is made with sub-pixel accuracy by fitting a Gaussian curve to the obtained

correlation function peak (Raffel et al., 2007). According to the same authors, this process leads to accuracy of less than 0.01 pixel in the determination of displacement vectors. Modern PIV software uses advanced interrogation techniques like multiple-pass interrogation and iterative window deformation (Scarano and Riethmuller, 2000), in order to adapt the interrogation windows to flow gradients (Raffel et al., 2007). These methods are present in the software (Lavision Davis 7.2) used during the PIV measurements Lavision (2007).

The result of the interrogation windows cross-correlation operation is a function that, in normal conditions, has a well defined peak as depicted in Figure 2.41 (a). However, due to poor seeding, bad lighting conditions, reflections, etc. one may not find a single isolated peak as represented in Figure 2.41 (b), which might lead to ambiguities in the determination of the displacement vector. In PIV it is possible to define a signal-to-noise ratio (SNR), which is defined by the ratio between the volumes of the two consecutive higher peaks:

$$SNR = \frac{V_{1p}}{V_{2p}} \quad (2.15)$$

where V_{1p} is the first (maximum) peak volume and V_{2p} is the volume of the second greatest. According to the Lavision (2007) PIV manual a measurement is good if:

$$SNR \geq 1.3 \quad (2.16)$$

In PIV processing it is usual to allow an overlap between adjacent interrogation areas in order to increase data yield. The reason to do so is the fact that particles near the edges of the interrogation window will contribute less for the correlation result. Allowing window overlap it is also a way of increasing the spatial resolution,

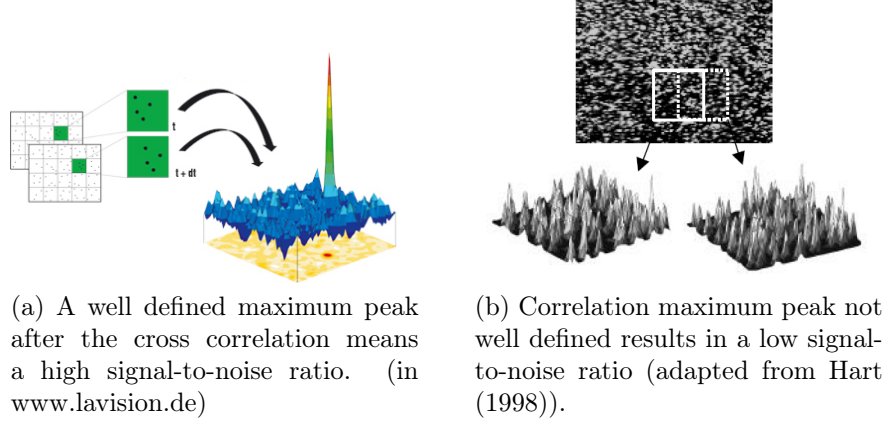


Figure 2.41: Different interrogation windows cross-correlation results. (a) Exemplifying a case of a high signal-to-noise ratio. (b) The case of a low-signal-to-noise ratio.

as illustrated in Figure 2.42. An example is here given, considering an image of size $1024 \text{ pixel} \times 1024 \text{ pixel}$. Considering that this image is divided in interrogation windows of $64 \text{ pixel} \times 64 \text{ pixel}$ a measuring grid of 64×64 points is obtained. However, considering an overlap of 50% between interrogation windows the measurement grid will have 128×128 points. This strategy was also used in the PIV processing applied to the dam-break flow.

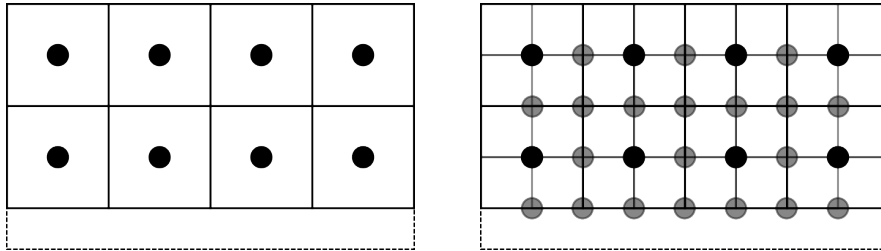


Figure 2.42: Illustration of the interrogation window overlapping. (a) measurement grid without overlapping and (b) measurement grid considering a 50% overlap.

2.2.3 Time Resolved PIV

Typical commercial PIV systems have an acquisition frequency of about 30 Hz, which is often limited by the laser repetition frequency. One way to overcome this is to use a continuous laser and a fast acquisition camera. In this case, the camera shutter plays the role of the laser pulses. The drawback of this approach is that the continuous lasers have usually less power output than the pulsed ones. The fast camera allowing image acquisition in a *continuous way* with a small time interval between them, it is possible to correlate the image at time $t + dt$ with the one at t as depicted in Figure 2.43, thus making possible the determination of the velocity field evolution with time. In this case n acquired images will result in $n - 1$ pairs of images. This approach is often referred as Time Resolved PIV (TR-PIV).

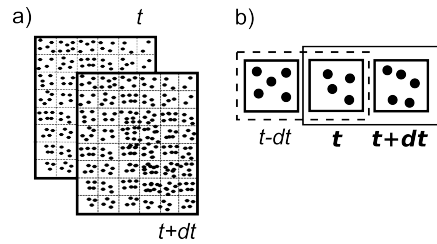


Figure 2.43: (a) Image division in interrogation windows and (b) image sequence used in Time Resolved PIV correlation: image $t+dt$ is correlated with image t .

In Figure 2.44 two consecutive images pairs are shown for the Classic PIV and in Figure 2.45 the two consecutive pairs of a Time Resolved PIV are shown. They are a better illustration of the difference between the two methodologies. It is clearly seen how the acquisition time is important in the case of a transient flow. While the classical PIV (Figure B) does not arrive to capture the dam-break wave collapse, the Time Resolved PIV, due to its time resolution manages to capture it.

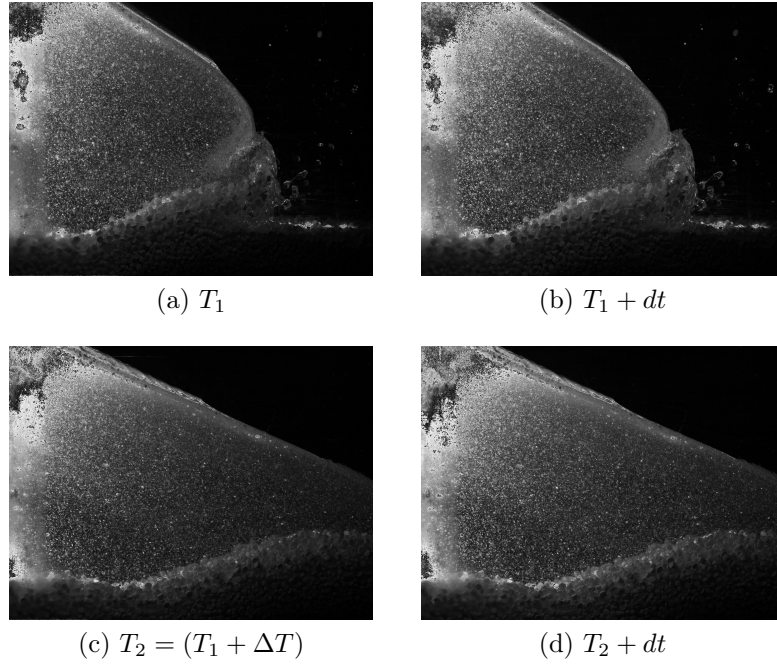


Figure 2.44: Illustrating 2 consecutive pairs of images of the Classical PIV approach. Images of a pair are separated by a time dt whereas two consecutive pairs are separated by a time ΔT .

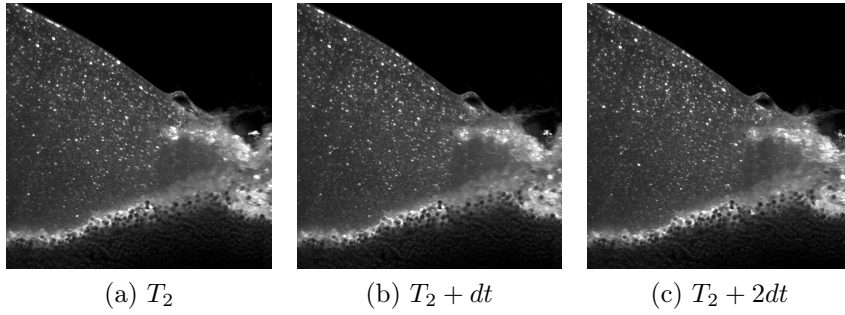


Figure 2.45: Illustrating 2 consecutive pairs of images of the Time Resolved PIV approach. In this case image (c) correlates with image (b) which is correlated with image (a), giving thus origin to two pairs of images.

Due to the transient character of the dam-break flow, the flow domain will change in time due to the changes in water surface and loose sediment bed. This will be visible in the corresponding

flow images as a boundary like the water free surface. To make the processing more efficient, a numerical mask image should be defined. This mask is used to identify the flow region of interest and to avoid useless and time consuming processing, for example above the free surface. The two main advantages in defining the mask are: a) the smaller processing time, since the no flow areas will not be processed; b) the post-processing and results interpretation becomes much easier. An example is given in Figure 2.46 and 2.47 where the velocity fields obtained without and with a mask are shown respectively. As it can be seen from Figure 2.47 the results become more realistic and easier to interpret. For a transient flow like the dam-break flow, the time varying free surface will require that the masks evolve also in time. Since the Lavision Davis 7.2 software was limited in this aspect two strategies were followed: a) for the case of classical PIV a mask was defined for each time instant analysed and b) for the Time Resolved PIV no mask was defined during the PIV processing. However a post process step was added to remove the values outside the flow region. This step was based on the information relative to the free surface taken from the flow images and consisted in imposing $\mathbf{v} = 0$ outside the flow region bounded by the free surface.

2.2.4 Tracer Particles For Imaging Techniques

The techniques described in the previous sections work by detecting and tracking particles (PTV) or by tracking patterns of particles (PIV). Either using PIV or PTV the flow is required to be seeded with particles, that can be detected by the camera. To achieve subpixel resolution the algorithm proposed by Capart and Young (2002) requires that the particle's images be in the range 5-20 pixels, whereas for PIV, Westerweel (2000) reported a 4-pixel lower limit to allow the reconstruction of continuous particles from discrete

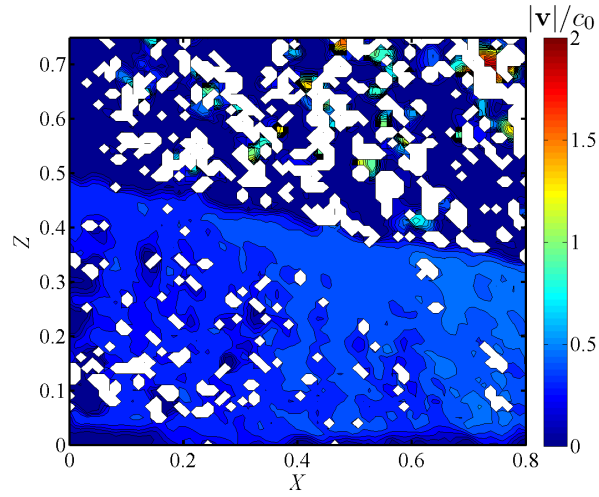


Figure 2.46: Velocity field at $T = 2.5$ of a dam-break flow over a smooth flat bed without using a mask. (Experiment: h32.5-hs0-D0-BP-run06).

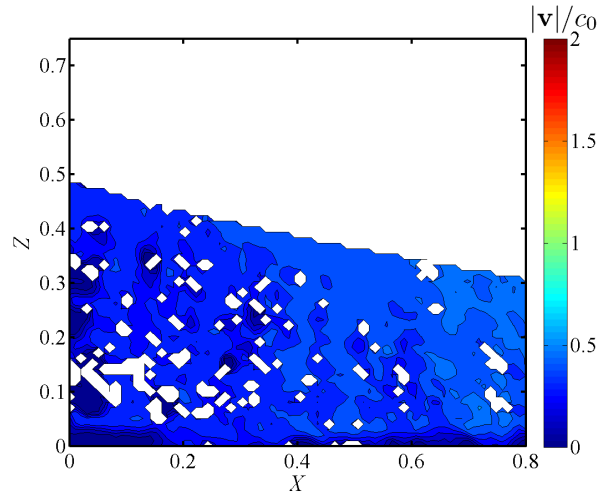


Figure 2.47: Velocity field at $T = 2.5$ of a dam-break flow over a smooth flat bed selecting by means of a mask only the flow region. (Experiment: h32.5-hs0-D0-BP-run06).

samples avoiding the peak lock effect. According to this author the 4-pixel value is a consequence of the Shannon-Whitaker theorem. However, the particles diameter can not be made as large as needed. According to Melling (1997) the seeding particles will act as a low-pass filter, whose cut frequency, f_c , is given by:

$$f_c \propto \frac{\mu}{\rho_p d_p^2} \quad (2.17)$$

where ρ_p and d_p are the particle's density and diameter respectively, and μ is the dynamic viscosity of the fluid. This equation can be used to determine the particles' diameter as a function of the flow frequencies one needs to resolve.

Particles made of high-density polyethylene ($\rho_p/\rho_w = 0.94$) with a mean diameter of 2 mm were used for the PTV measurements. To improve light reflection these particles were covered with a white coating. Combination of the 0.94 particle density and the denser white coating made the resulting particles roughly neutrally buoyant. The fall velocity of these particles was measured by means of the PTV algorithm, using for the effect the images of the particles' in the reservoir long before the gate removal. Figure 2.48 depicts the channel region where the fall velocity was measured and in Figure 2.49 the non-dimensional fall velocity $(U_f, W_f) = (u_f/c_0, w_f/c_0)$ is depicted. It is possible to conclude that the fall velocity is negligible regarding the natural dam-break velocity scale, c_0 .

For the PIV technique the particles used were made of Eliokem's pliolite[®], ($\rho_p/\rho_w = 1.03$) with a mean diameter of 0.5 mm. Although its density is close to water's density, to achieve a better dispersion in the water, the pliolite particles were first immersed during two hours in soapy water. This allowed to minimize the surface tension effects, allowing a better dispersion of the seeding in the flow.

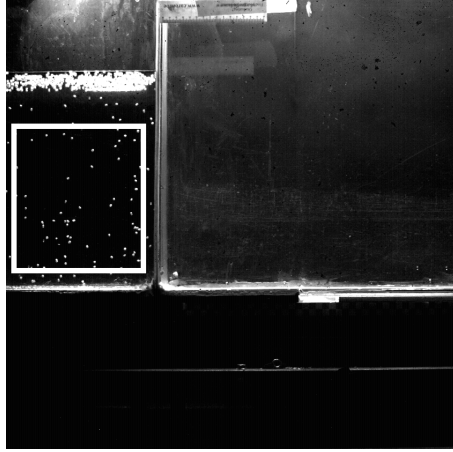


Figure 2.48: Imaged area upstream of the gate where the fall velocity of the seeded particles, U_f and W_f , is to be determined. The water is at rest.

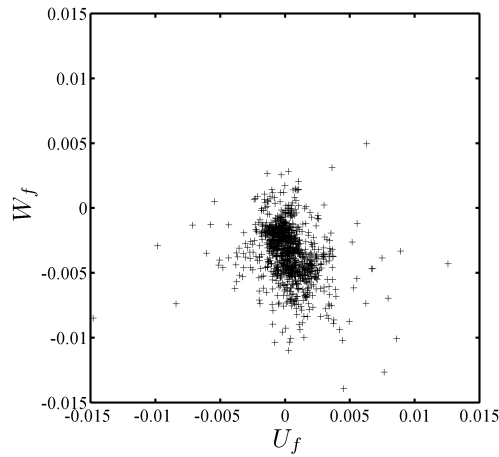


Figure 2.49: Non-dimensional fall velocity components, $U_f = u_f/c_0$ and $W_f = w_f/c_0$ of the particles imaged in the test section of Figure 2.48 with an initial water height of $h_0 = 0.325$ m.

2.2.5 Velocity Profiles Measurements by Doppler Effect - The UVP

The imaging methods are intriguing since they offer a way of describing the complete velocity field. In order to properly use the imaging techniques certain requirements must be met. For example: the calibration, optical access to the flow, stiff support for camera and lenses, etc. Another limitation of imaging techniques concerns the low spatial resolution near the wall, which is not enough if one is interested in determining accurately, for example, the shear stress, for which velocity measurements near the wall are required (Durst et al., 1996). The Ultrasonic Velocity Profile meter is a measurement technique that allows to overcome part of the referred limitations.

An Ultrasonic Velocity Profile meter (UVP) is an acoustic based technique that measures a velocity profile at a certain location. This technique is based on the acoustic Doppler Effect (Doppler, 1843), discovered by Christian Doppler. It consists in the change in frequency of a wave (for the present case, a sound wave is considered) for an observer moving relative to its source. This frequency shift is called the Doppler shift frequency, hereafter referred as Doppler frequency (e.g Halliday et al. (2001)).

The UVP was first developed by Takeda (1987, 1995) and its working principle is illustrated in Figure 2.50.

The UVP transducer has two working modes emission and reception. On the emission cycle the UVP works like a loudspeaker emitting an acoustic wave pulse that propagates through the flow. This acoustic wave burst is defined by the number of cycles n and its length is a function of the sound wavelength as depicted in Figure 2.51. The number of cycles n will control the spatial resolution and it is imposed by the user.

In the reception mode the UVP works like a microphone and

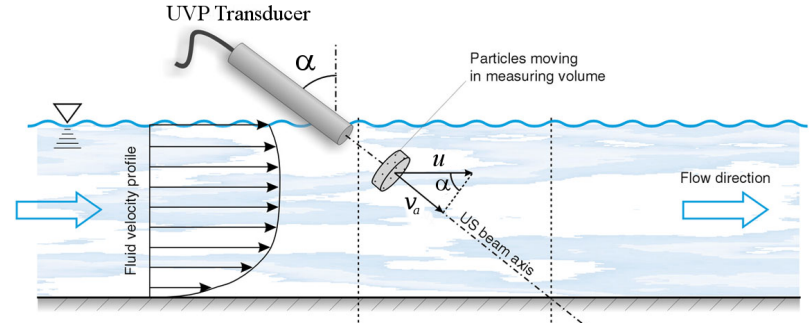


Figure 2.50: Illustration of the UVP working principle (in www.met-flow.com). A UVP transducer is immersed in a channel where a flow is established, with an angle α regarding the vertical. By emitting sound bursts with certain frequency and by receiving the echoes of the flow seeding particles, the UVP is able to determine the velocity of the flow by means of the Doppler Effect. v_a is the velocity measured in the axis of the transducer and u is the longitudinal component.

listens to the echoes reflected by the seeding particles of the flow. These echoes signals contain the information about the velocity of the seeding particles, since the echoes' frequency is now different from the emitting signal frequency, due to the Doppler Effect.

For the configuration of the UVP the expression that relates the velocity with the measured Doppler frequency is given by e.g (Halliday et al., 2001; Met-Flow, 2002)

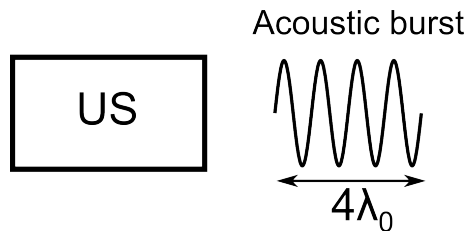


Figure 2.51: Example of an acoustic burst emitted by the ultrasonic transducer, US, with $n = 4$ as number of cycles. The burst length is in this case, $4\lambda_0$, where λ_0 is the sound wavelength.

$$v_a = f_D \frac{\lambda_0}{2} \quad (2.18)$$

where v_a is the measured velocity along the UVP axis, f_D is the Doppler frequency, and $\lambda_0 = c_s/f_0$, is the wavelength of the emitted sound. c_s is the speed of sound in water $c_s = 1480$ m/s for $T = 20^\circ$ C, and f_0 is the transducer emitting frequency, also referred as transducer natural frequency whose typical values are: 1 MHz, 2 MHz 4 MHz. By knowing the angle α it is thus possible to calculate the projection, $u = v_a/\sin \alpha$, along the horizontal direction.

Lemmin and Rolland (1997) presented a comparison between velocity profiles measured with the UVP, Hot-Film and Pitot-tube Anemometry. The authors' results show that UVP results are comparable with the other techniques, with maximum differences of about 10% for the Hot-Film Anemometry. The same authors also presented comparisons between the results of UVP and the different analytical laws of the wall. The time resolution of the UVP (less than 10 ms) allowed the same authors to compare the results for turbulent variables (e.g. $\overline{u'^2}$) measured with UVP with the ones obtained by means of the before mentioned techniques.

2.2.6 The Working Characteristics of the UVP

The present subsection deals with the intrinsic parameters of the UVP that have to be taken into account when setting up the experiments. Properties such as spatial resolution, time resolution and maximum velocity and depths ranges are here introduced and defined.

The acoustic beam can be seen as a set of control volumes, or measurement channels (hereafter referred as channels), that propagate along a certain direction as depicted in Figure 2.50.

Each control volume (channel) can be seen as a cylinder with a

width given by the transducer diameter, d and a height, h_u , that can be calculated by:

$$h_u = c_s \frac{n}{2f_0} = n \frac{\lambda_0}{2} \quad (2.19)$$

where n is the number of cycles of the burst, and λ_0 is the emitted sound wavelength, $\lambda_0 = c_s/f_0$. The particles inside each control volume are assumed to be uniformly dispersed, and the measured velocity, at the centre of the control volume, is taken as the average of those particles' velocities. The distance between the centre of two consecutive control volumes is called the channel distance, and it is a measure of spatial resolution. For a transducer with $f_0 = 4$ MHz it was possible to obtain spatial resolutions up to 0.37 mm. For practical reasons, however, the minimum spatial resolution used was 0.74 mm.

The time resolution is a key parameter in the transient flows analysis. N_r measurements are needed in order to obtain a velocity profile, where N_r stands for the repetition number; $N_r = 8$ means that eight measurements are needed to obtain one profile measurement. The values of N_r are defined by the user as $N_r = 2^n$ where n is an integer such as $n < 8$. Another parameter that will affect the time resolution is the pulse repetition frequency, F_{pr} , that is the frequency at which the instrument can generate a beam of acoustic bursts. F_{pr} is mainly a function of the flow depth that the UVP has to scan and of the sound velocity in the medium. The time resolution is given by:

$$\Delta t = \frac{N_r}{F_{pr}} \quad (2.20)$$

The minimum time resolution achieved with the used system was 4 ms, for a number of repetition, $N_r = 8$. Therefore each measured profile was a mean of 8 profiles measured in 4 ms. Of

course the more points one considers in a profile to measure, the slower will be the measurement.

A limitation of the UVP arises from the maximum pulse repetition frequency, F_{pr} . This is the maximum frequency available to sample the flow. By the Nyquist theorem (e.g Glisson (1985)), the maximum frequency of the phenomenon that one can resolve is:

$$f_{max} = \frac{F_{pr}}{2} \quad (2.21)$$

And the maximum Doppler frequency that one can measure is therefore:

$$f_{Dmax} = \frac{F_{pr}}{2} \quad (2.22)$$

where f_{Dmax} is the maximum Doppler frequency shift measured. By equation (2.18) the maximum measurable velocity is then (Met-Flow, 2002)

$$V_{max} = F_{pr} \frac{c_s}{4f_0} \quad (2.23)$$

The range of velocity, V_r is, according to the manufacturer, given by

$$V_r = 2V_{max} = F_{pr} \frac{c_s}{2f_0} \quad (2.24)$$

Where the factor of two comes from the possibility of measuring also negative velocities. The flow velocities to be measured v_f have to be inside the velocity range interval, i.e., $v_f \in [-V_{max}, V_{max}]$. On the other hand, the maximum depth that the UVP is capable to scan is given by:

$$h_{\max} = \frac{c_s}{2F_{pr}} \quad (2.25)$$

This equation states that before a new pulse is emitted, it is necessary that the echo of the previous pulse had the time to arrive to the transducer. From equations (2.25) and (2.24) one has:

$$h_{\max} V_r = \frac{c_s^2}{4f_0} = \Theta_{uvp} \quad (2.26)$$

where Θ_{uvp} is a constant. This is one of the fundamental equation of the UVP technique. This states that the velocity range times the maximum depth is constant, therefore maximum depth and velocity range have opposite behaviours. If one gives priority to increase the velocity range, the depth range has to be reduced, and vice-versa. As an example, let the sound of speed in water be $c_s = 1480 \text{ m/s}$; consider also a transducer with $f_0 = 4 \text{ MHz}$. Under these conditions one has $\Theta_{uvp} = 0.1369 \text{ m}^2/\text{s}$. Considering as an example a flow depth of 0.5 m , for which the complete profile is to be measured with the UVP, so that $h_{\max} = 0.5 \text{ m}$. This means that the maximum velocity the UVP can measure is

$$V_r = 0.273 \text{ m/s} \quad (2.27)$$

Since V_r is the range, that is the measurable velocity interval, one of the following three cases is possible with the UVP equipment used during the experiments:

1. $V_r = [0, 0.273]$, or
2. $V_r = [-0.273, 0]$, or
3. $V_r = [-0.136, 0.137]$.

The first and second cases can be used when it is known in advance that there will be no negative or positive velocities respectively; the third case, when there are both positive and negative velocities.

2.2.7 Advantages and Disadvantages of the UVP

An advantage of the UVP is its spatial and time resolution. Velocity profile measurements were carried out with spatial resolution of 0.74 mm and time resolution up to 4 ms, allowing the measurement of turbulent fluctuations. The processing is straightforward and obtains measurements in real time. Its simplicity to use when compared to the image processing methods described before is another of its advantages. Another advantage is the fact that it allows a larger duration of acquisition, and it is not limited by the internal RAM memory, like the case of the Photron camera. The fact that it is based on the Doppler effect allows for two important features:

1. it requires no calibration, either than the knowledge of the sound speed in the medium.
2. It is able to measure both positive and negative velocities.

The above are some of the reasons that propelled the application of the UVP to the dam-break flow. It is known that the sound speed depends on different factors namely the density of the fluid and its temperature. Figure 2.52 shows how the sound speed in water changes with temperature. In the experiments to be presented in Chapter 3 the water temperature measured was 15.6° C. The corresponding sound velocity was determined by a polynomial fit to the data presented in Figure 2.52.

One major disadvantage is that it only measures the velocity in the axis of the transducer. The UVP system used from MET-FLOW, allowed to acquire up to 21 transducers. At a first sight it

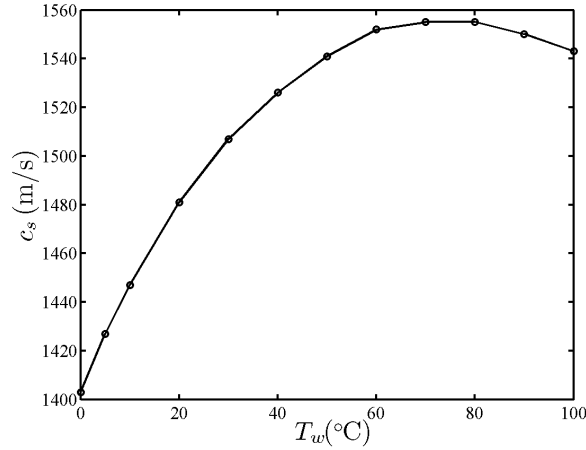


Figure 2.52: Variation of the speed of sound in water in function of water's temperature (Lide, 1992).

would seem possible to associate different transducers to measure more components as proposed by (Taishi et al., 2002). However the MET-FLOW system is a multiplexed one, that is the 21 different signals coming from the same number of transducers are not acquired simultaneously. In a multiplexed system, the transducers' signal is read one by one; thus in a given sequence and increasing the acquisition time, becomes too slow to use in the case of the dam-break flow. For the used system, it was observed that by adding another transducer the acquisition sampling time increased by a factor of 10.

Being an acoustic device, the transducer active tip must be in contact with the fluid either directly as depicted in Figure 2.50, or by means of a coupling medium (as curiosity the same principle applies to the echo based imaging used in pregnancy diagnosis, where an acoustic coupling medium is also used between the device and the woman's belly). Typical coupling materials are the propylene glycol, glycerine, water, vegetable and mineral oils (Stocksley, 2001). In the present case a water based gel pro-

vided by Dahlhausen[®] was used. The sound speed in the gel is $1480 \text{ m/s} < c_{gel} < 1560 \text{ m/s}$ which is quite close to the velocity in water as shown in Figure 2.52.

2.2.8 Placement of the UVP Transducers in the Dam-Break Channel

As it is known and as it was presented in Chapter 1, the water level in a dam-break flow changes with time; therefore, trying to place the transducers as shown in Figure 2.50 poses a serious practical problem. In Figure 2.53 two possible situations are represented. In the case depicted in Figure 2.53 the transducer A is completely submerged. In this case the measurement technique is no longer non-intrusive, and furthermore after a certain time the transducer will no longer be acoustically coupled with the water. On the other hand, the case depicted in Figure 2.53 shows that only after the transducer B is submerged, i.e., after a certain time interval, will it be able to measure the velocity, and therefore the information regarding the time instants before submergence will not be acquired.

To overcome the problems with the placement of the transducers a different strategy was devised. It was decided to place the UVP transducers at the channel's bottom. In Figure 2.54 a stretch of the bottom is depicted. Three sequential holes were made in the bottom as depicted in Figure 2.54 allowing the combination of up to three transducers to be applied in case of slow flows the strategy proposed by Nowak (2002) to measure two velocity components of the flow. As depicted in Figure 2.54 the drills did not cross all the perspex' thickness to ensure the waterproofing of the channel's bottom. The acoustic coupling between the transducers and the perspex was achieved with a water based acoustic gel and the sound propagation through the remaining perspex thickness was verified by preliminary tests. One of the key factors controlling sound prop-

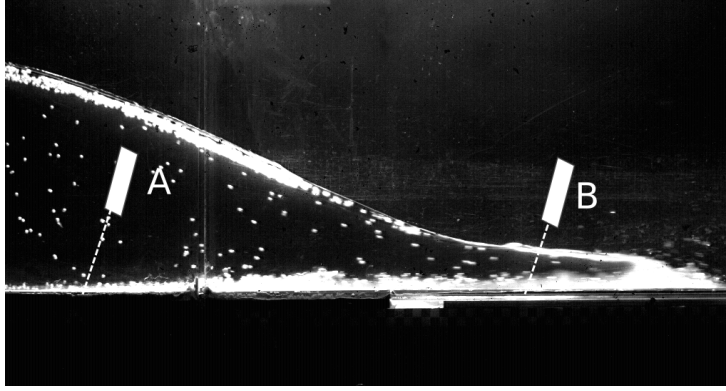


Figure 2.53: An example of two possible configurations for UVP transducer placement in the dam-break flow channel. The transducer A is completely immersed and it will be able to measure until the instant it is in air. The transducer B is not immersed and it will start measuring when immersed. In this configuration, both transducers intrude significantly the flow.

agation between different media is the acoustic impedance coupling. The acoustic impedance of a certain medium M is given by (e.g. Kinsler et al. (2000)):

$$Z_M = \rho_M c_{sM} \quad (2.28)$$

where Z_M is the acoustic impedance of the medium M , ρ_M its density and c_{sM} is the speed of sound in the referred medium M . To ensure an optimal sound transmission the manufacturer advises to have at the interface solid-liquid a ratio of impedances such as:

$$2 \leq \frac{Z_2}{Z_1} \leq 3 \quad (2.29)$$

For water $Z_w = 1.5 \times 10^6 \text{ Ray} = Z_1$ and for the Plexiglas $Z_p = 3.3 \times 10^6 \text{ Ray} = Z_2$, one has $Z_2/Z_1 \approx 2.2$.

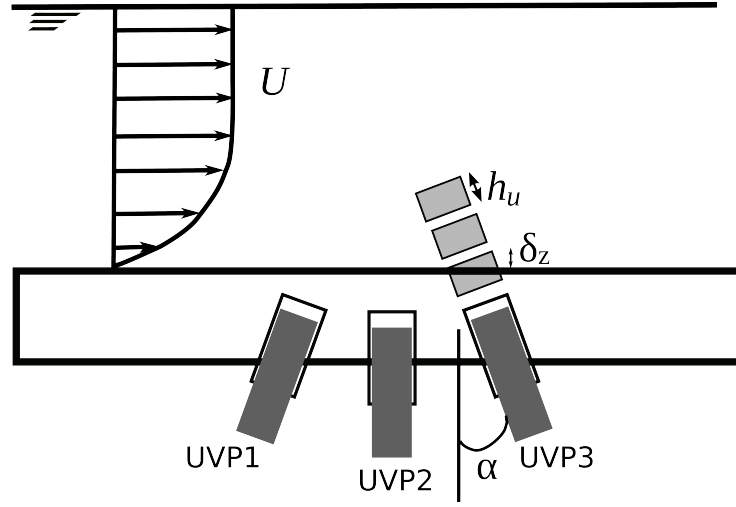


Figure 2.54: Stretch of the channel's bottom illustrating the UVP transducers placement adopted for the experiments. From the three positions only one was used during these study, and it is represented by the emitting transducer. The emitted burst, with height h_u will interact with the channel's bottom and there will be a location uncertainty associated with, δ_z , as discussed in section 2.2.9.

2.2.9 Wall Detection and Correction

A major problem associated with the use of the velocity profile for shear stress determination is the accurate determination of the first measurement point above the wall. This subsection presents a method to correct the wall effect on the UVP control volume.

In the case of the UVP, as it was referred in subsection 2.2.6, the measurement point is taken as the centre of the control volume. However this is the case when the control volume is completely immersed in the flow. Near the wall it might occur that a part of the control volume is immersed in the flow and another part inside the wall, as depicted in Figure 2.54. This problem was addressed by Nowak (2002) and Taishi et al. (2002). The fact that a) it is not possible to control the thickness of the acoustic coupling gel placed and b) the fact that different positions along the channel were used

were the reasons to use a simpler criterion. It is here assumed that the uncertainty associated with the location of the control volume is half of its height (Durst et al., 1988). This is a criterion often used in the Laser Doppler Anemometry. To help clarify this statement, Figure 2.54, illustrates the control volume. For the present case, the uncertainty associated to the wall location, δz , can be estimated as, (considering $\alpha = 20^\circ$):

$$\delta_z = \frac{h_u}{2} \cos \alpha \approx 0.35 \text{ mm} \quad (2.30)$$

and δ_z is the vertical coordinate of the control volume as depicted in Figure 2.54.

Since it is not possible to know in advance where the first control volume will be because of the variable coupling gel thickness, the $z = 0$ coordinate in the profile has to be determined by analysing the measured profiles. This can be done easily by means of the UVP software. An example is illustrated in Figure 2.55 where a typical velocity profile and echo Doppler signal are plotted. The echo signal has a maximum in the interface UVP transducer-gel. Velocity is measured zero up to the first measuring point and this information is used to identify the channel's bottom location, as indicated in the figure by the dotted line.

2.3 Shear Stress Measurement Methods

By definition the shear stress at the wall is given by:

$$\tau_w = \mu \left. \frac{\partial u}{\partial z} \right|_{z=0} \quad (2.31)$$

where μ is the dynamic viscosity and $\partial u / \partial z|_{z=0}$ is the velocity gradient at the wall. As shear stress is a function of the velocity

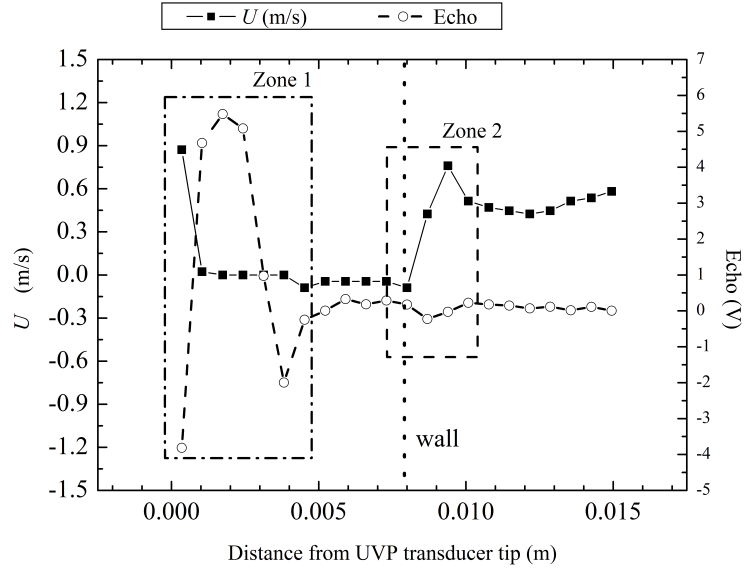


Figure 2.55: Typical velocity and echo profile obtained from a UVP transducer. Dashed-point box Zone 1 indicates the first points measured. The echo signal is used for interface identification. Velocity is zero up to the dotted line placed at the channel's bottom location. Dashed box Zone 2 indicates the first measurements in the flow region. Profile measured in $X = 2.46$ and $T = 9.67$.

gradient, it is a very difficult variable to measure experimentally since it imposes to determine the velocity profile very accurately near the bed.

In order to measure the shear stress two classes of methods exist: direct methods and indirect methods. Direct methods, as their name indicates, are methods that measure directly the shear-stress. Examples of such methods are the Oil Film Interferometry where Fizeau interferometry can be used to measure the thinning rate of an oil film deposited on a surface and subjected to a bounding flow (Tropea et al., 2007). This technique was first proposed by Tanner and Blows (1976). Usually these interferometry based techniques work better for air flow than liquid flows.

Another method to obtain directly the shear stress is by means

of floating element force balances e.g. Dhawan (1952), Acharya et al. (1985). As pointed out by Winter (1977), floating element devices although simple in theory, have some challenges associated with like the effect of gaps around the element and the effect due to the misalignment of the element, for example. The development of Micro Electro Mechanical Systems (MEMS) allowed to conceive smaller floating elements and to avoid numerous problems associated with the ancient floating element technique (Lofdahl and el Hal, 1999; Tropea et al., 2007). An optoelectronic based shear stress transducer was proposed by Measurement Science Enterprise. This transducer is based on the Doppler effect to determine the shear stress (Modaress and Wilson, 2006). An application in the fluvial hydraulics domain was presented by Carvalho et al. (2010).

The other type of method, the indirect methods relies on the measurement of a certain variable that can be easily related with the shear stress. One example is the shear stress determination by heat transfer measurements (e.g. Sanborn (1972); Tavoularis (2009)). Zidouh et al. (2009) proposed an electrochemical transducer based on the studies of Hanratty and Campbell (1983) for the measurements of the shear stress in unsteady flows. Another indirect method consists in doing spatial resolved velocity measurements in order to determine the flow velocity profile. It is known that for boundary layer flows, the velocity profile obeys certain laws according to the region of the flow, as proposed by Prandtl and von Kármán in the 1930s (e.g. White (1991)). These laws are usually function of the shear stress of the wall.

A scheme of the different regions of the flow is depicted in Figure 2.56, for fixed smooth beds, hypothesis that will be kept throughout this section. The region closest to the bed (here also referred as wall) is usually denominated sub-viscous layer (e.g. (Kundu et al., 2011)), and is the region where the viscosity effects are predominant. The law of the wall for the sub-viscous layer is given by (e.g

Schlichting (1966)):

$$\frac{u}{u_*} = \frac{u_* z}{\nu} \quad (2.32)$$

where $u_* = \sqrt{\tau_w/\rho_w}$ is the shear velocity, ρ_w is the density of the fluid, z is the distance from the wall and ν is the kinematic viscosity. Another way of writing equation (2.32) is using wall coordinates and therefore:

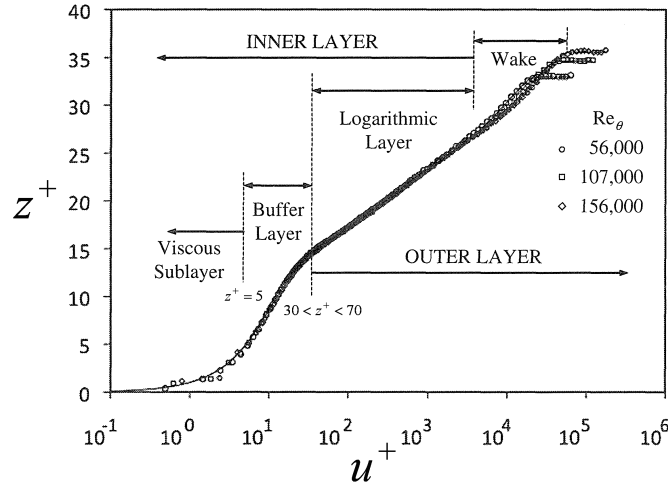


Figure 2.56: Velocity profile over a smooth flat surface and representation of the different regions of the flow. Adapted from Oweis et al. (2010) and Kundu et al. (2011).

$$u^+ = z^+ \quad (2.33)$$

where $u^+ = u/u_*$ and $z^+ = u_* z/\nu$. This shows that in the near-wall region the velocity profile is linear. However, this region is confined to a very narrow region of the flow, usually $0 < z^+ \leq 5$ in wall coordinates, or within a range of a few micrometers (Carvalho et al., 2010), which makes it difficult to access that region.

Nevertheless, Durst et al. (1996) using Laser Doppler Anemometry proposed a method to obtain shear stress measurements from the velocity profile measured in the sub-viscous region.

Between the sub-viscous layer and the logarithmic layer a buffer layer exists (e.g. Kundu et al. (2011)). The logarithmic region, located above the buffer layer (see Figure 2.56) can be described by the following law of the wall (Prandtl, 1935; Millikan, 1938; Landau and Lifshitz, 1987):

$$\frac{u}{u_*} = \frac{1}{\kappa} \ln \left(\frac{u_* z}{\nu} \right) + B \quad u_* = \sqrt{\frac{\tau_w}{\rho_w}} \quad (2.34)$$

where for a smooth bed, $B = 5$ and $\kappa \approx 0.41$ is the von Kármán constant. These are the commonly accepted values; however, in the literature it is possible to find that $0.37 < \kappa < 0.41$ and $3.5 < B < 6.2$. As curiosity, Zanoun et al. (2003) defines κ and B as functions of e (Euler number) as: $\kappa \approx 1/e$ and $B \approx 10/e$. From the works of Tritton (1988), Nezu and Nakagawa (1993), Pope (2000), Oweis et al. (2010), Kahler et al. (2012) the log region starts at about $z^+ \approx 30$. Zagarola and Smits (1998) proposed $z^+ > 50$ and Winckelmans and Bricteux (2008) using DNS suggested that the log region starts at $z^+ = 70$. The upper bound limit of the logarithmic layer is at $z/\delta \approx 0.15 - 0.2$, where δ is the boundary layer height, that is, the height for which the velocity is 99% of the undisturbed potential velocity, U_o , $u = 0.99U_o$.

The log method and its derivatives like the Clauser plot method (Clauser, 1956) have been widely used in many applications to obtain measurements of shear stress, nevertheless some authors (e.g. Barenblatt (1993); Barenblatt and Chorin (1996)) claim that the conditions for the existence of a logarithmic region are not always met. Above the logarithmic region there is the wake region whose law of the wall can be written as a velocity defect law (Coles, 1956):

$$\frac{U_o - u}{u_*} = F\left(\frac{z}{\delta}\right) \quad (2.35)$$

where F is a function that depends on the ratio between the distance to the wall, and δ the height of the boundary layer.

2.4 Instrumentation Utilised

To measure the different types of dam-break flows different instruments were used in the experimental campaign. This section details the different equipments, focusing on their features and technical details.

2.4.1 Acoustic Water Level Probes

Acoustic water level probes work on the principle of sound emission and reflection (echo). By measuring the time between sound emission and reception, and knowing the speed of sound in air it is possible to determine the distance between the acoustic level probe and the obstacle, in this case, the water surface (Figure 2.57). Four Ultrasonic Baumer transducer (hereafter referred as Baumer) were used to obtain the point-wise evolution of the water surface at different locations. A National Instruments acquisition card converts the analogue output signal of the acoustic level probes into a digital signal which is processed and stored by National Instruments software Labview.

To work properly the Baumer has to be placed perpendicular to the surface. For the case of the dam-break flow, the water surface presents an important slope in the first time instants. In this case the echo does not travel back towards the transducer as depicted in Figure 2.57 (b), resulting in the lack of measurements.

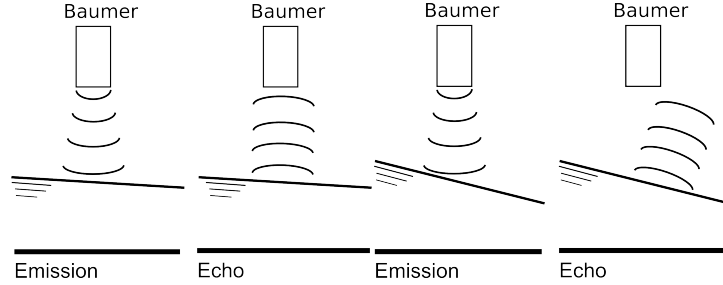


Figure 2.57: Example of the working principle of the Baumer ultrasonic transducer. (a) Case of a mild slope water surface where the echo is reflected back to the transducer. (b) Case of an important water surface slope; in this case the echo is not reflected back to the transducer.

The four Baumer acoustic level probes were placed above the channel as illustrated in Figure 2.58. Their position is indicated in Table 2.6.

Table 2.6: Location of the Baumer transducers.

Transducer #	US1	US2	US3	US4
Position x (m)	-0.05	0.3	0.6	1.2
Position X	-0.15	0.92	1.84	3.69

2.4.2 Image Acquisition Systems: Cameras and Lenses

Different types of cameras and lenses were used to capture the dam-break flow in order to extract from them quantitative information about the velocity field. They were associated with different velocity measurement techniques as indicated below.

The camera Dalsa 1M150 was used for the PTV measurements. It has a maximum resolution of 1 Mpixel and adjustable acquisition frequency up to a maximum of 150 frames per second (fps) at full resolution. Its memory buffer was of 1.5Gb. To obtain a total

acquisition time of 15 s, the acquisition frequency was set at 100 fps. With this camera a lens of 8 mm was used. Dalsa's cameras were operated using Dalsa's acquisition software.

With the PIV system a PCO camera with a maximum acquisition frequency of 30 Hz was used. This camera was equipped with a Nikon lens of 50 mm f:1.7. The acquisition time of this camera was associated to the one of the PIV and it was of 20 s. The images were acquired by means of the Lavision Davis 7.2 software. For each PIV run an acquisition of 320 images at 14.773 Hz was made.

Finally a Photron SA300 Fast camera was used to apply the Time Resolved PIV technique. A Carl Zeiss lens of 50 mm f:1.4 equipped this camera. At its full resolution of 1 Mpixel it allows to acquire up to 2000 fps. However its memory buffer was limited to 1.361 Gb, meaning an acquisition time of 1.361 s at 1000 frames per second. The images were acquired and stored by means of the Photron software PFView 7.1,

2.4.3 The PIV System

The PIV system used is a system provided by LaVision (Lavision, 2007). It consisted in a two-head Litron pulsed laser at a nominal frequency of 15 Hz (in reality: 14.773 Hz). Each laser head had a maximum energy per pulse of 30 mJ. The laser power, and time interval between consecutive pulses was controlled by Lavision Davis 7.2 software. To light the desired flow section, a movable optical arm was used to guide the laser light from the generator to the channel. At the laser arm's exit a diverging optical system converts the laser beam into a laser light sheet. The thickness of the laser light sheet was 1.5 mm. In Figure 2.58 the guiding arm, A, leads the light from the laser generator to a optical diverging system, B, on the channel top. A camera, E, placed perpendicular to the chan-

nel is used to capture the seeding particles images. The calibration target, D, is used to convert pixels in physical units, e.g. in meters.

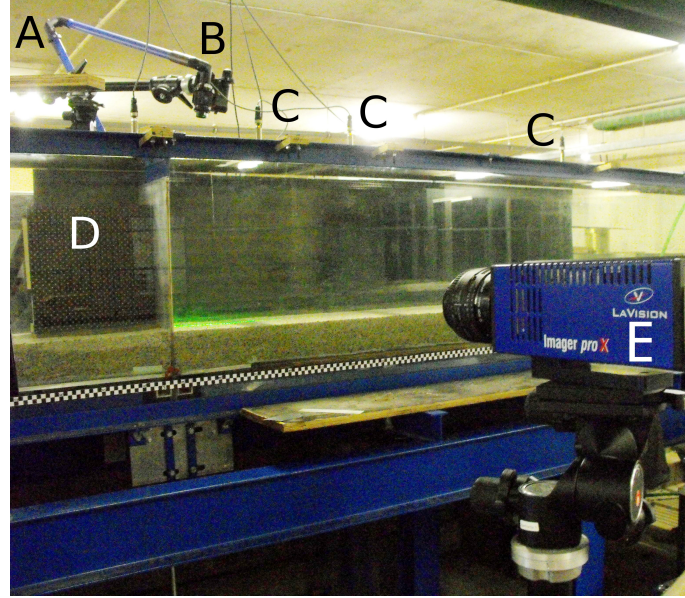


Figure 2.58: Overview of the PIV system used in the sedimentologic dam-break experiments used. A: is the guiding arm. B: is the diverging optics to convert the incoming laser into a laser light sheet. C: indicates the acoustic level transducers. D is the calibration target and E is the camera with the receiving optics installed. The green line over the sediment bed is a flash of laser captured in the photography. For details concerning the elements location see Table 2.6, Table 2.10, Figure 2.59 and text.

As referred in subsection 2.4.2 the images for the PIV were obtained by means of a PCO camera. The synchronization between laser and camera is made by means of the Lavision Programmable Time Unit 9 (PTU9). The obtained images were processed with Davis 7.2 software in order to determine the velocity field. Figure 2.58 illustrates a part of the PIV system and its placement in relation to the channel.

2.4.4 The UVP System

The UVP system used was made by MET-FLOW and it can be divided in three parts:

1. the acoustic transducer working at a certain nominal frequency, f_0 .
2. the acquisition and conditioning unit responsible for the signal generation, acquisition and filtering.
3. the processing unit consisting in a computer with the UVP control software UVP Monitor by MET-FLOW.

Three sets of transducers were available defined by their nominal frequency f_0 . In Table 2.7 the characteristics of each transducer set are presented.

Table 2.7: Physical characteristics of the different UVP transducers: Natural frequency, f_0 , wave length, λ_0 , velocity range, V_r , minimum control volume dimension, h_u

Set #	f_0 (MHz)	λ_0 (mm)	V_r (m/s)	h_u (mm)
1	0.5	2.96	3.6507	2.96
2	1	1.48	1.825	1.48
3	4	0.37	0.4563	0.37

Although the velocity range of sets #1 and #2 seem according to the expected maximum velocity for the dam-break flow performed at the laboratory (from equation (1.16) $2c_0 \approx 3.57$ m/s), their spatial resolution is not enough to perform near wall measurements, for which resolutions less than 1 mm are often needed (e.g. Durst et al. (1996)), therefore the third set was used.

2.5 List of Experiments

In this section the list of the performed and analysed experiments is presented along with the respective parameters.

Different experiments were made during this study considering the dam-break flow processed over a fixed bed (hydrodynamic dam-break flow) and over a loose sediment bed, the sedimentologic dam-break.

For the hydrodynamic dam-break flow the parameters changed were the initial water height and the channel fixed bed type: wood, glass and perspex. The physical properties of the fixed beds used are described in Table 2.8 and Table 2.9 presents the different configurations used with each initial water level.

Table 2.8: Properties of the different types of bed tested and the respective initial water heights. (k_s denotes the rugosity values taken from Lencastre (1996))

Material	Code	k_s (mm)
Glass	G	0.0036
Plywood	W	0.07
Perspex	P	0.0036

Table 2.9: Channel bed types for the Hydrodynamic cases tested.

h_0 (m)	Upstream Reach	Downstream reach
0.325	Wood	Glass
0.325	Perspex	Perspex
0.40	Wood	Glass

For the sedimentologic case only one initial water height was used: $h_0 = 0.325$ m. This height is measures above the sediment bed height, h_s . Two different types of sediments were used: sand and PVC pellets, whose properties are described in Table 2.10. When using sediments the fixed bed over which they were placed was always made of plywood.

Table 2.10: Sediments used in the sedimentologic dam-break flow.

Material	Code	Diameter (d_{50}) (mm)	Density (kgm^{-3})	Bed height h_s (m)
Sand	S	1.72	2.63×10^3	0.10
PVC Pellets	P	3.3	1.1×10^3	0.10

From Table 2.10 it is possible to see that the PVC pellets are lighter than the sand, and so, they will be easily eroded and transported by the dam-break wave. On the other hand they are easier to set up since no bed compacting operation is required.

To better identify the different measurements, a code is assigned to each experiment that resumes the most important variables involved. This will allow a better association between processed data and experimental conditions. This code is in the form:

code: hAAA-hsTAAA-DAA-BXY-SA-RunAAA

where AAA is a number. The following convention is adopted:

hAAA: refers to the initial water height in centimetres. So h40 means that the initial water height is 40 cm.

hsTAAA: refers to the sediment bed used. T is used to indicate the type of sediment used. T=S the sediment used is sand. T=P the sediments used were the PVC pellets. AAA refers to the height of the sediment bed in centimetres. As an example hsS10 means a sediment bed of 10 cm made of sand. If there is no sediment bed, that hsTAAA=hs0.

DAA: refers to the diameter of the used sediments in millimetres. D0 refers to a bed without sediments.

BXY: refers to the type of fixed bed used, useful for the hydrodynamic dam-break case. X and Y refers to the upstream and

downstream fixed bed material, respectively. The different types of bed used were plywood (code: W), glass (code: G) and Perspex (code: P). So BWG means a dam-break experiment over a channel bed that upstream of the gate was made of wood, and made of glass downstream of the gate location. If the material is the same upstream and downstream, then only one letter is used.

SA: this only applies for the cases concerning UVP measurements and refers to the sections where the UVP was placed.

RunAAA: refers to a specific run. This allows identifies those cases where, for the same conditions, different runs were made. Example Run03 is the third run in the conditions expressed by the precedent references.

Two practical examples are here given:

h40-hs0-D0-BWG-Run004: is the 4th run of an experiment made without sediments and with an initial water level of 40 cm. The upstream bed was made of plywood and the downstream bed made of glass.

h32.5-hsS10-D1.73-BW-Run002: is the 2nd run of an experiment made with a bed sediments made of sand whose $d_{50} = 1.73$ mm, with an initial sediment level of 10 cm and initial water level of 32.5 cm. The upstream and downstream channel bed beneath the sand bed was made of plywood.

In Figure 2.59 the measurement sections of each set of experiments are depicted. This information is complemented with Table 2.12. Note that in experiments h32.5-hs0-D0-BWG and h40-hs0-D0-BWG all the width of the channel was used (no laser light sheet used) and the results are referred to the channel's centreline. The

distance camera to channel was a function of the chosen lenses and section to be analysed.

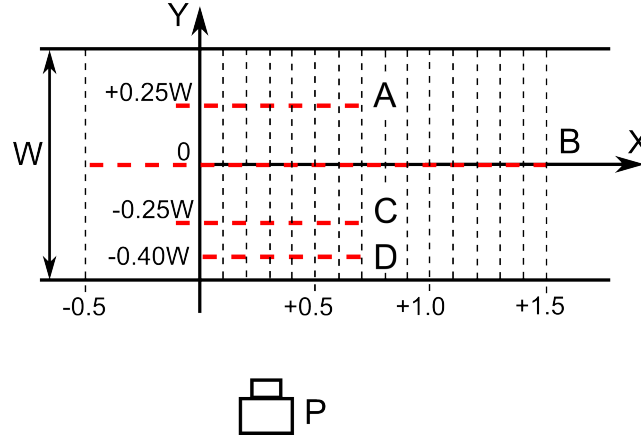


Figure 2.59: The location of the different measurement sections regarding the gate ($X = 0$) and channel's centreline ($Y = 0$). W denotes the channel width (0.25 m) and P indicates the photo camera location.

Table 2.11 indicates which technique was mainly used with each group of measurements, and in Table 2.13 a list of all experiments and respective parameters are listed. In that same table n refers to the total number of experiments and p to the number of successful ones.

Table 2.11: List of the measurement techniques used with each experiment.

Experiment	Measurement Technique
h32.5-hs0-D0-BWG	PTV
h32.5-hsS010.D1.73-BW	Classical PIV
h32.5-hsP010.D3.3-BW(A)-(C)	Classical PIV
h32.5-hsP010.D3.3-BW	Time-resolved PIV
h32.5-hs0-D0-BP	Time-Resolved PIV
h32.5-hs0-D0-BP-S1	UVP
h32.5-hs0-D0-BP-S2	UVP

2.5. List of Experiments

Table 2.12: Measurement sections used with imaging techniques

Experiment	Section	Y	$X_{\min}$	$X_{\max}$
h32.5-hs0-D0-BWG	B	0	-0.5	1.5
h40-hs0-D0-BWG	B	0	-0.5	1.5
h32.5-hsP010.D3.3-BW(A)	A	0.25W	-0.1	0.65
h32.5-hsP010.D3.3-BW(B)	B	0	-0.09	0.65
h32.5-hsP010.D3.3-BW(C)	A	0.25W	-0.09	0.65
h32.5-hsP010.D3.3-BW	D	-0.4W	0	0.7
h32.5-hs0-D0-BP	D	-0.4W	0	0.7

Table 2.13: List of all experiments made using the notation presented before. (A)-(C) experiments made with classical PIV; (*) and (**) experiments made with the UVP.

Experiment	Case	h_0 (m)	h_s (m)	d_{50} (mm)	Material upstream	Material downstream	runs (p/n)
h32.5-hs0-D0-BWG	Hydro.	0.325	0	0	Plywood	Glass	4/6
h40-hs0-D0-BWG	Hydro.	0.40	0	0	Plywood	Glass	4/6
h32.5-hsS010.D1.73-BW	Sand	0.325	0.10	1.72	Plywood	Plywood	3/4
h32.5-hsP010.D3.3-BW	PVC pellets	0.325	0.10	3.3	Plywood	Plywood	3/4
h32.5-hsP010.D3.3-BW(A)-(C)	PVC pellets	0.325	0.10	3.3	Plywood	Plywood	3/3
h32.5-hs0-D0-BP	Hydro.	0.325	0	0	Perspex	Perspex	7/21
h32.5-hs0-D0-BP-S1 (*)	Hydro.	0.325	0	0	Perspex	Perspex	15/15
h32.5-hs0-D0-BP-S2 (**)	Hydro.	0.325	0	0	Perspex	Perspex	10/21

Chapter 3

Dam-break Experiments

*And if the dam breaks open many years too soon
And if there is no room upon the hill*
Waters (1973)

This chapter deals with the experiments made on the dam-break flow. Two types of dam-break flows were considered: hydrodynamic, i.e. without sediments, and sedimentologic with sediments. For the hydrodynamic case, two initial water heights were considered and for the sedimentologic case, the initial water height was kept constant, but two types of sediment beds, one made of sand and the other made with PVC pellets, were used, as described in section 2.5. The experiments analysed in this chapter will be referenced by the respective code name as defined in section 2.5.

The goals of this chapter are the following:

1. To analyse quantitatively the early stages of the dam-break flow.
2. To determine the velocity field of both the water layer and sediment layer for the different cases of dam-break flows.
3. To compare the obtained results with the Ritter (1892) and Pohle (1952) models.

Parts of this chapter were published in Aleixo et al. (2009), Aleixo et al. (2010), Aleixo et al. (2010), Aleixo et al. (2011) and Swartenbroekx et al. (2011).

3.1 Study of the Hydrodynamic Case

This section is dedicated to the hydrodynamic dam-break flow, i.e., when no sediments are present. The hydrodynamic case here presented was analysed with the Voronoï-PTV algorithm proposed by Capart et al. (2002) and described in Chapter 2. The experiments analysed in this section are the experiments referred by h32.5-hs0-D0-BWG-run## and h40-hs0-D0-BWG-run## (where # denotes the experiment number).

For the hydrodynamic case two conditions defined by two different initial reservoir water heights were studied: $h_0 = 0.325$ m and $h_0 = 0.40$ m. The natural time and velocity scales associated with each one of these cases are given by equation (1.26) and (1.14) and are systematized in Table 3.1.

Table 3.1: Time and velocity times scales of the two hydrodynamic dam-break flow cases.

Case	(h_0) (m)	t_0 (s)	c_0 (m/s)
h32.5-hs0-D0-BWG-run##	0.325	0.182	1.785
h40-hs0-D0-BWG-run##	0.40	0.202	1.980

Froude similitude is used to compare the two different conditions. Although the two cases have different initial heights, comparison will be made using the same non-dimensional times, $T = t/t_0$, using the values presented in Table 3.1 for scaling purposes.

Considering the same non-dimensional time instants, Figure 3.1 and Figure 3.2 depicts some images of the initial instants, ($-0.385 < T < 1.09$), of dam-break flow for two of the performed experiments:

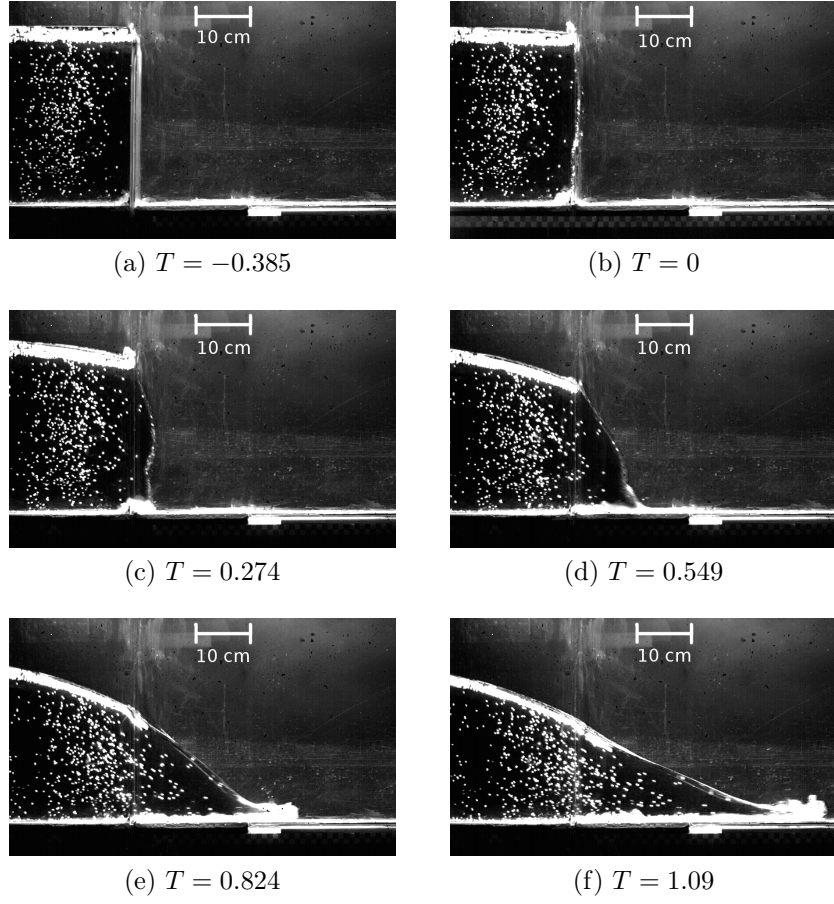


Figure 3.1: Initial instants of the dam-break flow for experiment h32.5-hs0-D0-BWG-run4 ($h_0 = 0.325$ m).

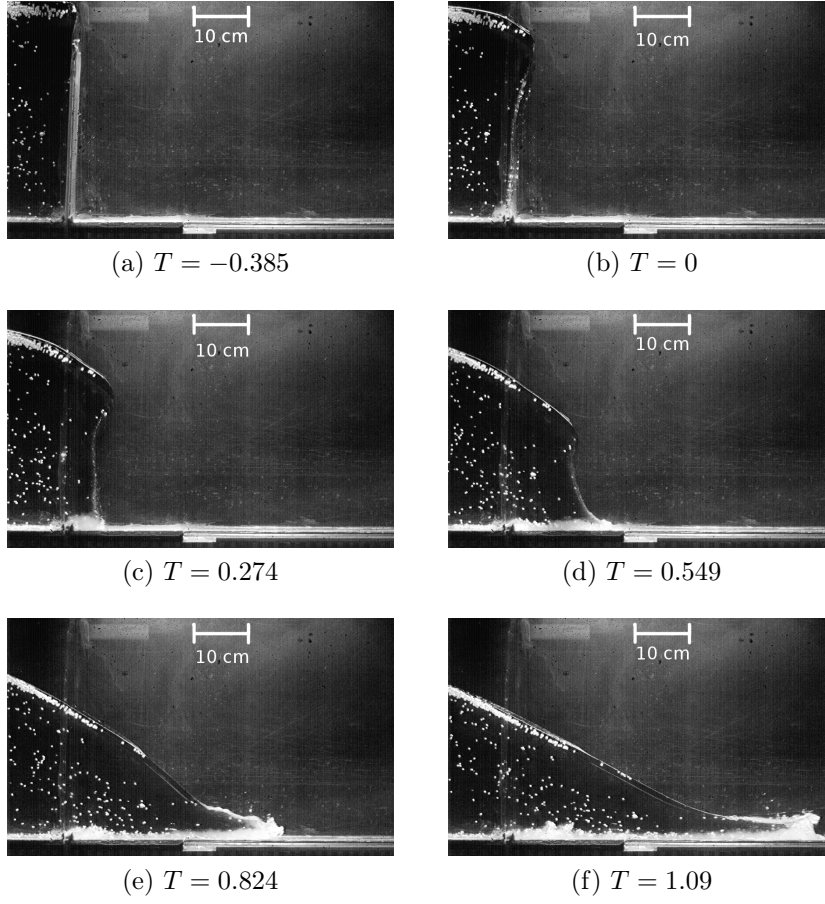


Figure 3.2: Initial instants of the dam-break flow for experiment h40-hs0-D0-BWG-run2 ($h_0 = 0.40$ m).

h32.5-hs0-D0-BWG-run4 and h40-hs0-D0-BWG-run2 respectively. It is clear when comparing Figures 3.1 (b) and 3.2 (b) the beginning of weir-like flow in this last case, a feature that is still visible in Figure 3.2 (c). Figures 3.1 (c) and (d) show that in this case the front wave is a convex shape, where in Figures 3.2 (b) and (c), due to the initial weir-flow characteristic, the shape of the wave front is concave.

3.1.1 The Very Near-Field Velocity Measurements

The very near-field of a dam-break flow refers to the immediate vicinity of the gate and to the very first time instants after the gate opening. The time $T = 0$ was considered as the moment when the gate is completely removed and the corresponding raw images are depicted in Figure 3.3.

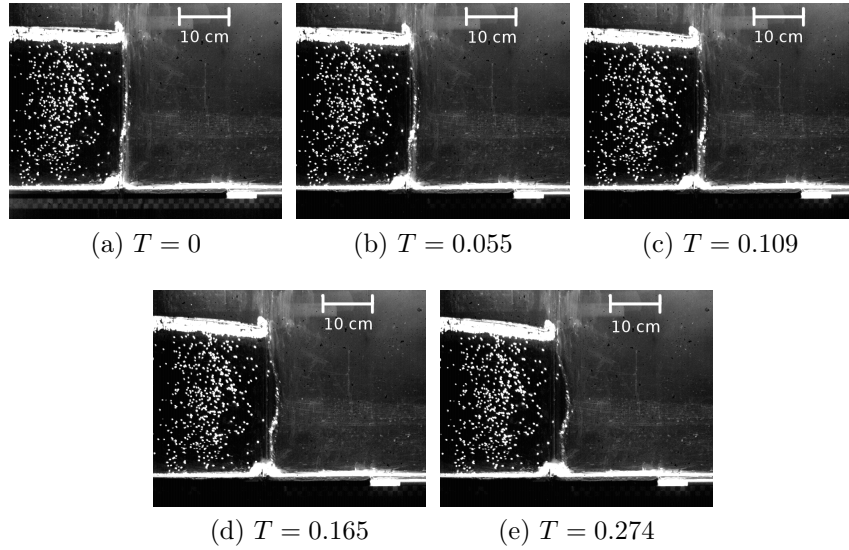


Figure 3.3: The very first instants of the dam-break flow for experiment h32.5-hs0-D0-BWG-run4 that are used to determine the velocity field by means of the Voronoï PTV algorithm.

For this study, a time-averaging of the results over $\Delta t = 0.05$ s is performed, i.e. over five images of the flow. This time was cho-

sen for two reasons: a) to minimize the time average effect on the results, and b) to ensure that between the first image and the last image the free surface has not changed significantly. Moreover, by using five images instead of only two it is possible to increase the reliability of the tracking since the same particle was successfully detected and tracked between successive frames. For example, existing PTV methods use three (Papantoniou and Dracos, 1989), four (Hassan and Canaan, 1991) and five images (Wernet and Pline, 1993) for a correct tracking.

For $h_0 = 0.325$ m the velocity field at $T = 0.11$ (centre of the considered time interval) is shown in Figure 3.4.

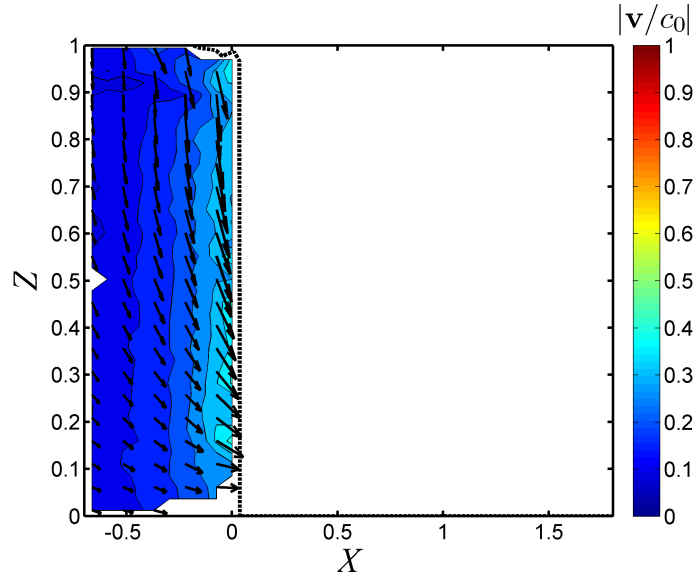


Figure 3.4: Velocity vectors distribution for $T = 0.11$ for an initial water height of $h_0 = 0.325$ m. For clarity only 1 out of 2 vectors are depicted.

The polar plot of Figure 3.5 represents the modulus (concentric radius) and angle of the velocity vectors, 0° is horizontal and 270° is the downward vertical direction. From this figures it is possible to see that for $Z > 0.75$ the flow has a strong vertical component,

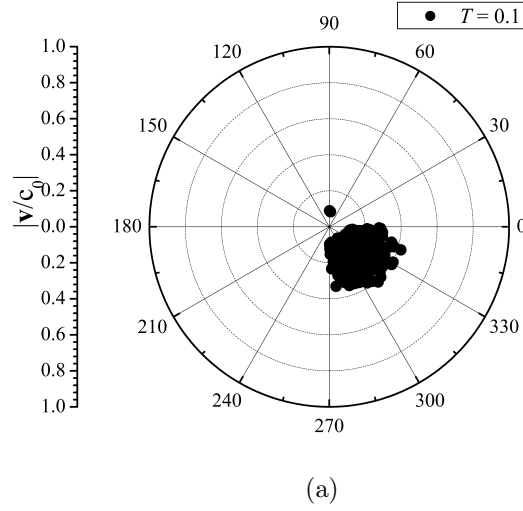


Figure 3.5: Flow field and velocity-vectors distribution for $T = 0.1$ for an initial water height of $h_0 = 0.325$ m.

as it would be expected, whereas, for $Z < 0.2$ the flow tends to be horizontal. This means that the vertical component is important, and therefore the shallow-water hypotheses are not yet valid. This is also seen in Figure 3.6, where the profiles of U and W at $T = 0.1$ are depicted. It is possible to see that $|U/W| \approx 1$ and therefore the vertical component is not negligible.

As it can be seen from Figures 3.4 near the free surface the flow is practically vertical as suggested by Pohle (1952). Figure 3.7 depicts different profiles of $-w/(gt_m)$, where t_m is the mean time used for the velocity determination (0.03 s in this case) and $-gt_m$ is the velocity of a free fall body. It is possible to observe that near the free surface the measured velocity is in the order of magnitude of $-gt_m$. At the gate section however, $-w/(gt_m) \approx 2$, probably due to the gate movement.

In section 1.5.2 the analysis of the initial instants developed by Pohle (1952) was presented. The PTV algorithm allows to describe the flow in a Lagrangian way, that is, by individualizing and tracking particles. In this way the comparison between

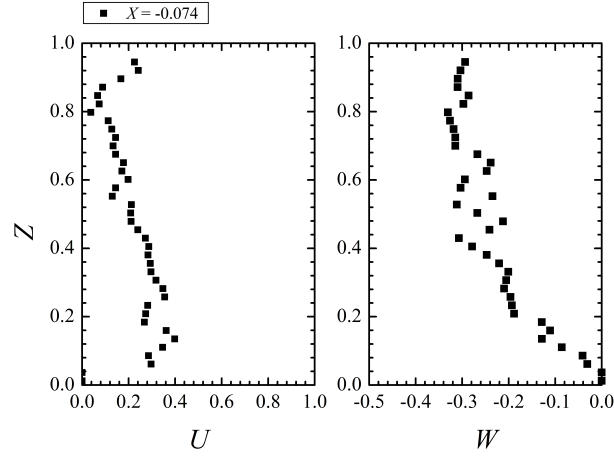


Figure 3.6: Velocity profiles, U and W , for $T = 0.1$ and initial water height of $h_0 = 0.325$ m. (For this time only it was considered the nearest section to the gate's available.)

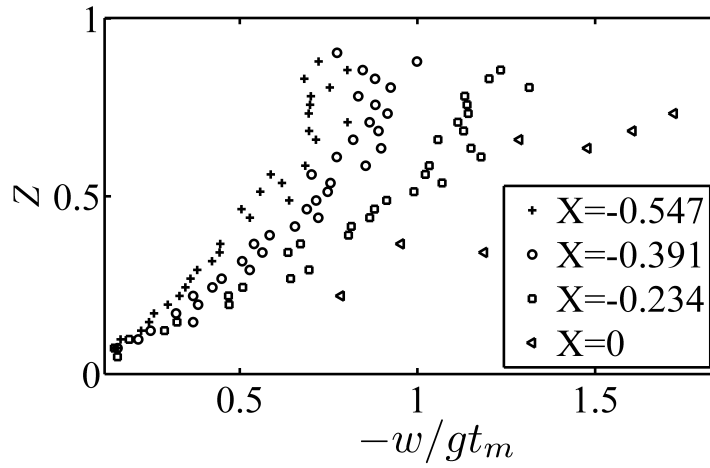


Figure 3.7: Non-dimensional fall velocity, $-w/gt_m$, profiles measured for $T = 0.1$ at different sections upstream the gate section. Initial water height of $h_0 = 0.325$ m.

experimental results and Pohle (1952) model is straightforward.

For the analysis of the flow the interval $-0.77 \leq T < 1$ was chosen. In Figure 3.8 the raw images corresponding to $T = -0.77$ and $T = 1$ are depicted.

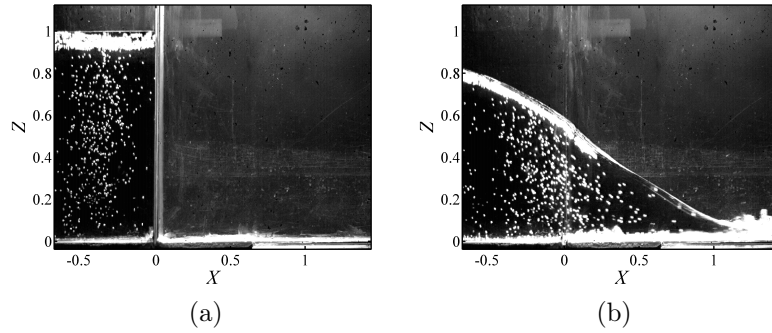


Figure 3.8: Flow raw images for (a) $T = -0.77$ and (b) $T = 1$. (Experiment: h32.5-hs0-d0-BWG-run4.)

Figure 3.9 (a) depicts the tracked trajectories by means of the PTV algorithm presented in subsection 2.2.1 and in Figure 3.9 (b) only the trajectories of particles identified and tracked in all the time instants are depicted, i.e, from $T = -0.77$ to $T = 1$, their density shows how the Voronoï-PTV algorithm is capable of tracking the different scales of velocity. Four particles were chosen to be compared with Pohle's (1952), and the results comparison is shown in Figure 3.10. As referred in subsection 1.5.2, according to Pohle (1952), immediately after the dam-break, the particles in the free surface would move like a free falling body. This is shown to happen in Figure 3.10 for particles's trajectories A, B, C and D. Pohle (1952) model (equation (1.33) predicts pretty well the spatial evolution of the water layer top particles as it can be seen in Figure 3.11.

An investigation was also conducted to assess the time interval for which Pohle's equation (eq. 1.34) is valid. Let β_P defined as:

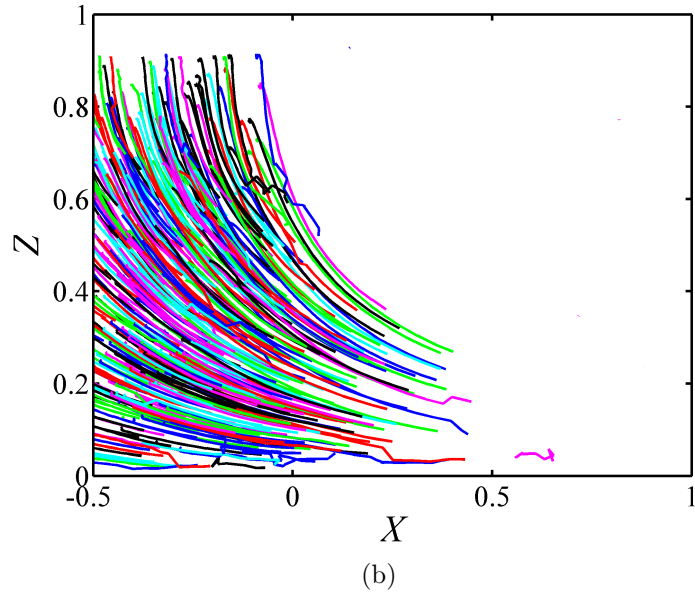
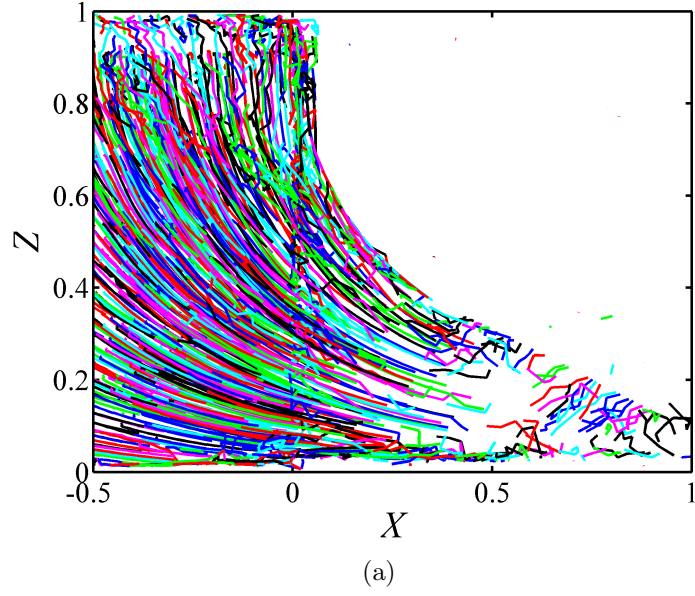


Figure 3.9: PTV computed trajectories between $T = -0.77$ and $T = 1$, (a) All trajectories from detected and tracked particles from Figure 3.8(a); (b) Only the trajectories of the particles tracked during the all duration $-0.77 < T < 1$.

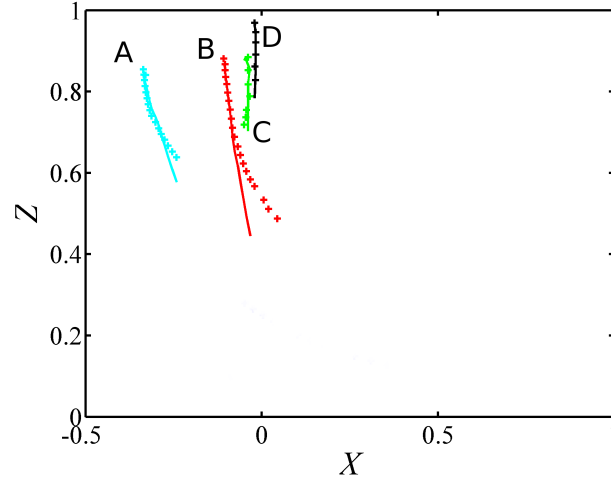


Figure 3.10: Different particles, A, B, C, and D, trajectories determined by the Voronoï-PTV algorithm (dots) and the respective ones obtained (lines) by means of Pohle (1952) model.

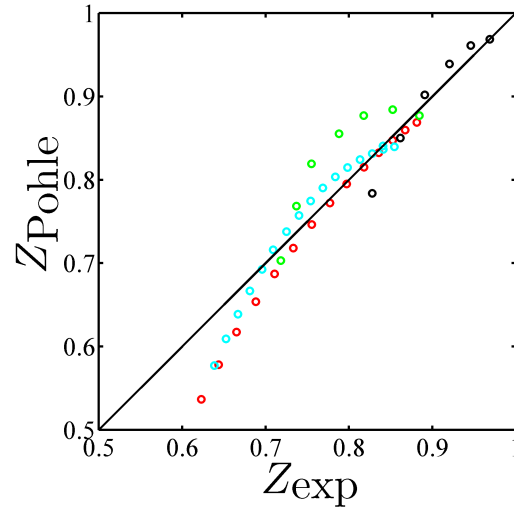


Figure 3.11: Comparison between different trajectories (particles A, B, C and D, as indicated in Figure 3.9 determined by the Voronoï-PTV algorithm and the ones obtained by means of Pohle (1952) model.

$$\beta_P = \frac{|Z_{exp} - Z_{Pohle}|}{Z_{exp}} \quad (3.1)$$

be the difference between experimental and theoretical values. β_P is a function of time, and it is expected that β_P will increase with time, as it can be observed in Figure 3.11. The time evolution of equation (3.1) was computed for a set of 15 particles and the results are shown in Figure 3.12. For a limit of 5% difference the average time validity of Pohle's model is found $T = 0.71$, which is in close agreement with the value $T_{PE} = 0.7$ obtained by Estrade (1967).

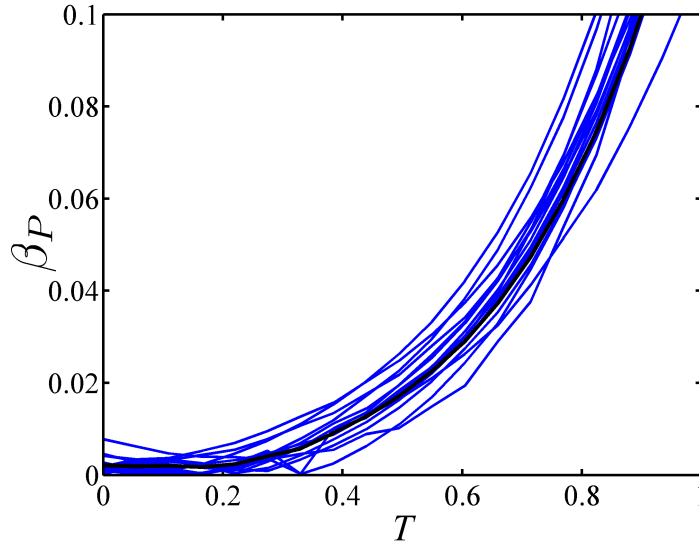


Figure 3.12: Time evolution of equation (3.1). Black line: average value.

For $T = 1$ the velocity field and the polar distribution of the velocity field are represented in Figure 3.13 and 3.14. The time evolution of the flow is significant. The dam-break flow is propagating downstream and the water surface is continuously evolving. The velocity increases with X , as depicted in Figure 3.13 and the Voronoï PTV algorithm shows that it can track different scales of

velocity. This illustrates how the Voronoï algorithm is capable of tracking different velocity scales in the flow field images.

As it can be seen in Figure 3.14 the angle of the velocity vector is now about 345° . From the profiles at $X = 0$ in Figure 3.15, one sees that U has increased, when compared with the profile measured in Figure 3.6.

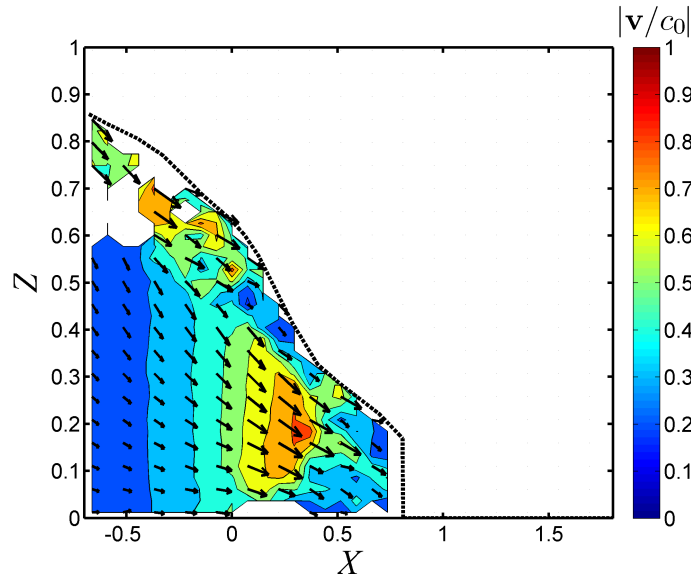


Figure 3.13: Flow field obtained by means of PTV Voronoi tracking algorithm for $T = 1$ for and initial water height of $h_0 = 0.325$ m.

3.1.2 Evolution Towards Shallow-Water Flow

Results for $T = 2.5$, $T = 5$ and $T = 10$ are shown in Figures 3.16-3.23. The continuous evolution of the flow can be clearly identified: (i) the water depth decreases to reach the theoretical constant value of $H = 4/9$ at the gate section ($X = 0$) and (ii) the vertical component of the velocity becomes negligible, conforming to the shallow-water assumption (Stoker, 1957). Starting from the results obtained for $T = 2.5$ one can first see that the water height is reduced (Figure 3.16), and secondly that at the gate region it tends

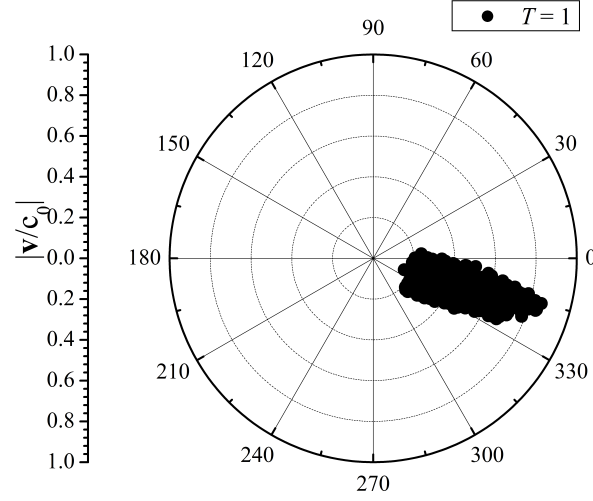


Figure 3.14: Velocity-vectors distribution (modulus and angle) for $T = 1$ for an initial water height of $h_0 = 0.325$ m.

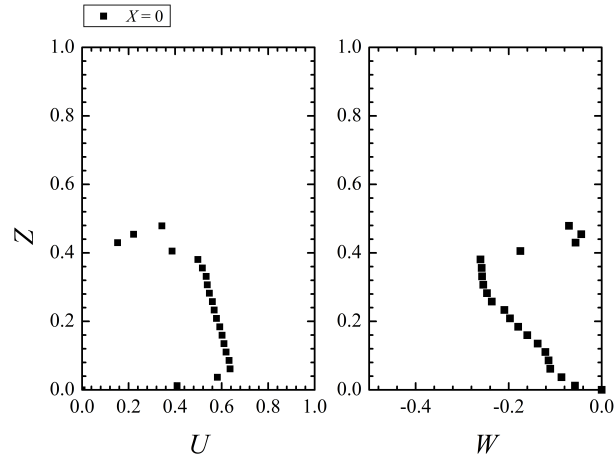


Figure 3.15: Velocity components, U and W , for $T = 1$ and initial water height of $h_0 = 0.325$ m.

to the $Z = 4/9$. Also from Figure 3.16 one sees that the velocity is not uniform throughout the domain. For $X = 0$ one sees that U continues to increase while W decreases. The angle is now smaller and tends to zero.

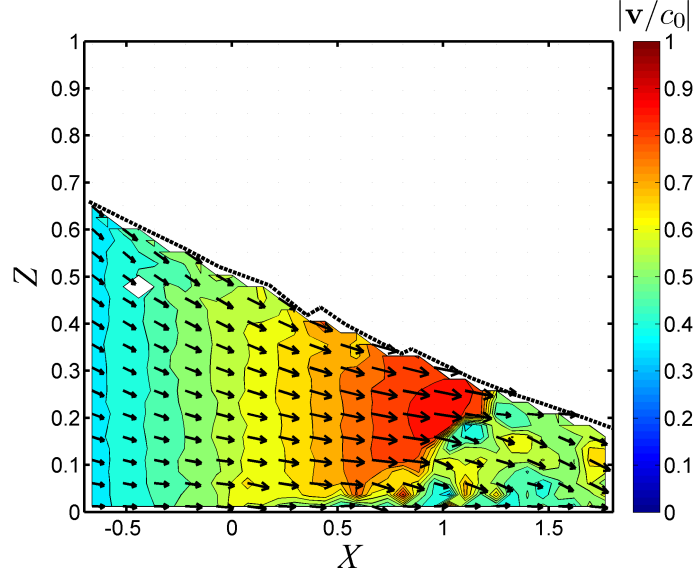


Figure 3.16: Flow field and velocity-vectors distribution for $T = 2.5$ for an initial water height of $h_0 = 0.325$ m.

For $T = 5$ (Figures 3.19-3.21) and $T = 10$ (Figures 3.22-3.24) one has the natural evolution of the flow. Velocity tends to be more uniform and its angle is now about 0° , meaning that the flow is parallel to the bottom. Comparing both profiles at $X = 0$ in Figure 3.21 and 3.24, one sees little difference between the two, meaning that in the imaged section the flow is established and the St.-Venant are valid.

It is now possible to define a limit for which the St.-Venant equations are valid. It is clear from Figures 3.20 and 3.23 that the flow is parallel to the channel's bed, and the velocity profile is practically constant, furthermore $U \gg W$. For this non-dimensional times the St.-Venant hypotheses are clearly valid. For $T = 2.5$

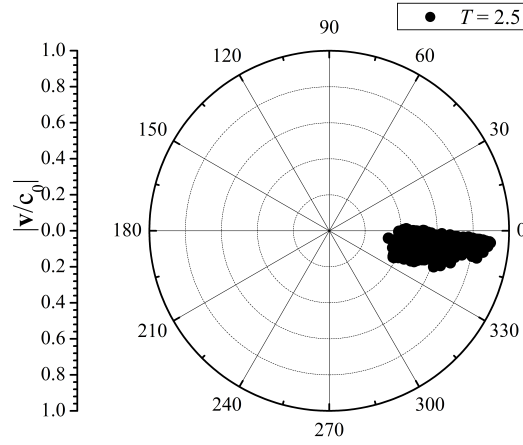


Figure 3.17: Velocity-vectors distribution for $T = 2.5$ for an initial water height of $h_0 = 0.325$ m.

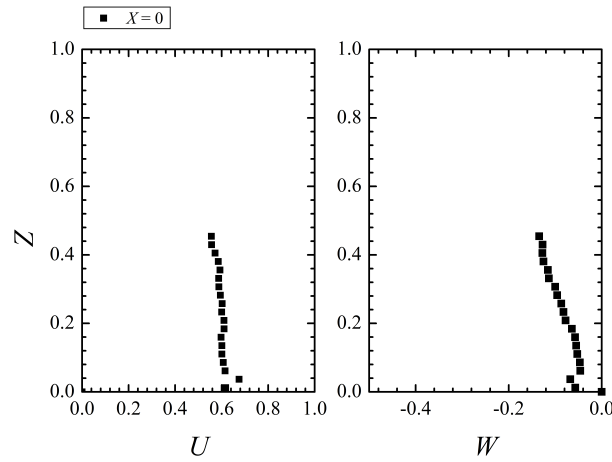


Figure 3.18: Velocity components, U and W , for $T = 2.5$ and initial water height of $h_0 = 0.325$ m.

one has a situation in accordance with the St.-Venant hypotheses. Although the velocity vectors angle is not zero, it will be demonstrated in 3.1.3 that the flow obeys the St.-Venant conditions. The limit for application of the St.-Venant equations is thus placed here at $T = 2.5$. As it was mentioned in section 1.4, Bukreev and Gusev (2005) found $T = 2.4$ to obtain the $Z = 4/9 \approx 0.44$ at the gate

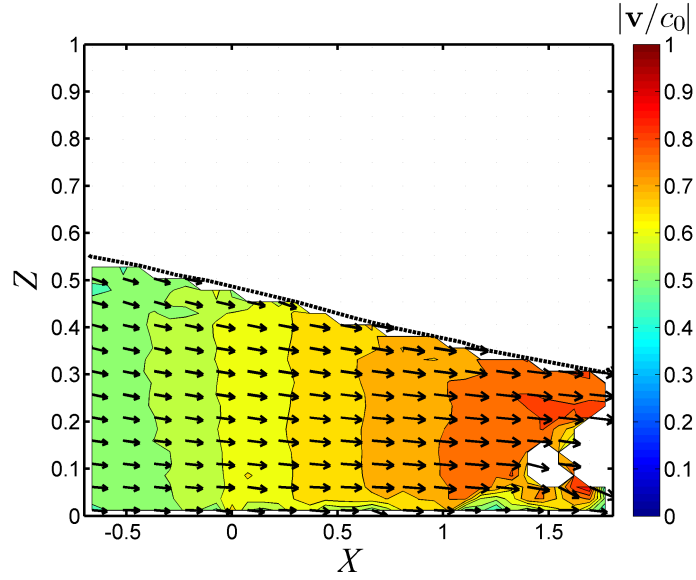


Figure 3.19: Flow field for $T = 5$ for an initial water height of $h_0 = 0.325$ m.

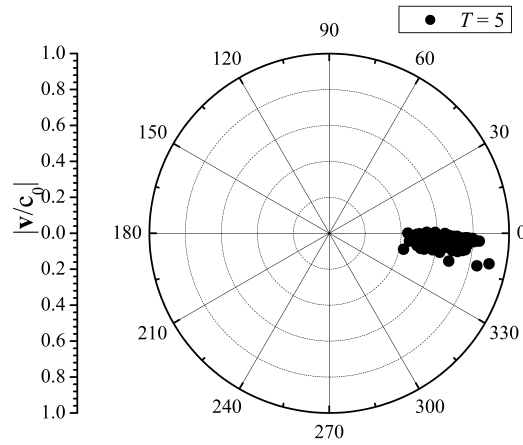


Figure 3.20: Velocity-vectors distribution for $T = 5$ for an initial water height of $h_0 = 0.325$ m.

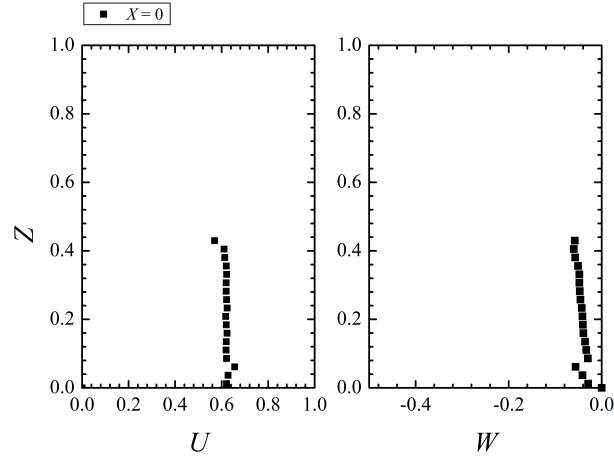


Figure 3.21: Velocity components, U and W , for $T = 5$ and initial water height of $h_0 = 0.325$ m.

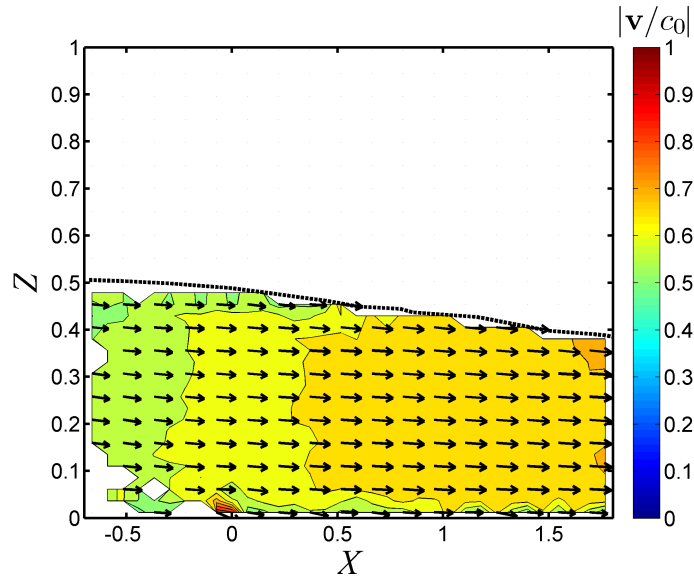


Figure 3.22: Flow field for $T = 10$ for an initial water height of $h_0 = 0.325$ m.

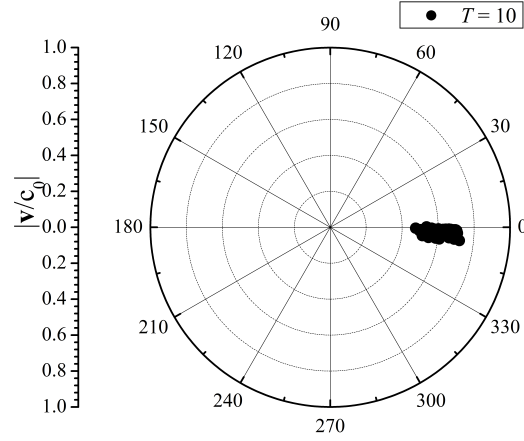


Figure 3.23: Velocity-vectors distribution for $T = 10$ for an initial water height of $h_0 = 0.325$ m.

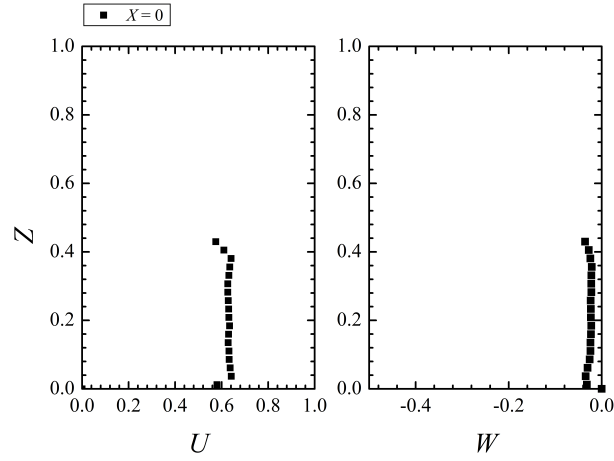


Figure 3.24: Velocity components, U and W for $T = 10$ and initial water height of $h_0 = 0.325$ m.

section, however in the present case it was obtained $Z = 0.52$ for $T = 2.5$.

3.1.3 Comparison with Ritter (1892) Theory

An analytical solution for the dam-break flow in an infinite smooth channel with a fixed bed was proposed for an ideal fluid by Ritter in 1892 and presented in Chapter 1. This solution is derived from the shallow-water equations in a depth-averaged framework, i.e. assuming that the vertical component of the velocity is negligible and thus that the pressure distribution is hydrostatic. In Chapter 1 the expressions for the evolution of the water depth Z , equation (1.20) and the depth-averaged velocity U , equation (1.21) were presented. These equations indicate a self-similar solution, i.e. for given X and T , Z and U will take the same value regardless of the initial condition h_0 in the reservoir.

Figures 3.25 and 3.26 depict the water level evolution with time in different locations measured with a ultrasonic Baumer transducer (see section 2.4.1). Measurement refer to experiments h32.5-hs0-D0-BWG and h32.5-hs0-D0-BWG and are the average for the four runs. Except for $X = -0.125$ in the vicinity of $T = 0$, it can be seen that the results for the water level agree pretty well with Ritter equation (1.24) for $T < 10$. This indicates also that Froude similitude is valid in that domain. The discrepancies observed for $X = -0.125$ in the initial stages are certainly related with the gate removal effect, that, as it was seen in section 2.1.1 can not be considered instantaneous. For large values of T ($T > 10$) the experimental values diverge from the Ritter (1892) solution. This is due to the finite size of the reservoir. Figures 3.25 and 3.26 can be used to determine the time the reservoir can be considered to be in the Ritter (1892) conditions for both initial conditions.

For the velocity this can also be checked for the presented exper-

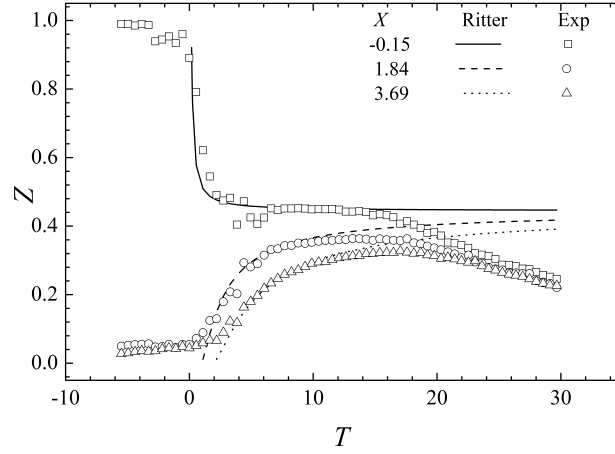


Figure 3.25: Comparison between the average values of the water level evolution in three different points and the Ritter (1892) solution. (Initial water level: $h_0 = 0.325$ m, experiments: h32.5-hs0-D0-BWG).

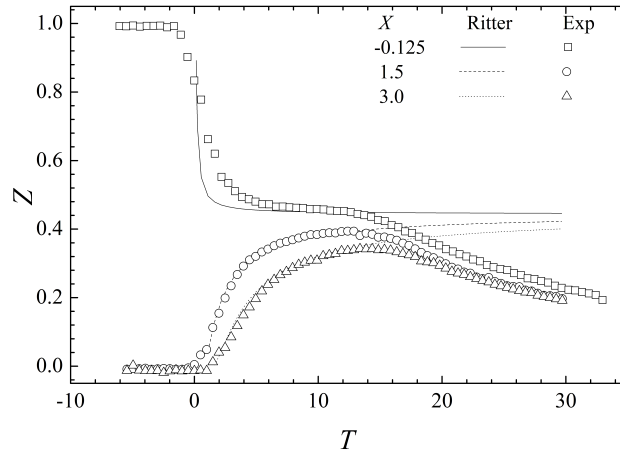


Figure 3.26: Comparison between the average values of the water level evolution in three different points and the Ritter (1892) solution. (Initial water level: $h_0 = 0.40$ m, experiments: h40-hs0-D0-BWG).

imental series at times when the flow corresponds to the shallow-water equations, i.e. when the vertical component of the velocity has become negligible. This is clearly the case for $T = 5$ and $T = 10$ (Figure 3.19 and 3.22). In Figures 3.27 to 3.30 a comparison of the theoretical depth-averaged velocity and of the measured velocity profiles $U(Z)$ is shown for $X = -0.23$, $X = 0$, $X = 0.38$, $X = 1.41$, and for $T = 5$ and $T = 10$.

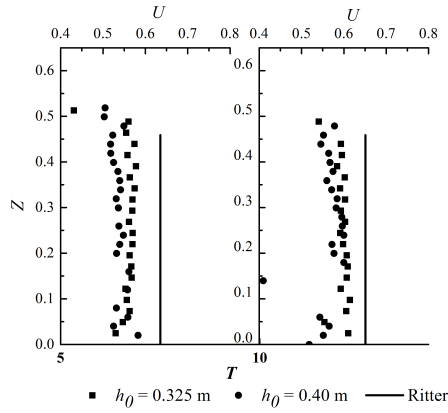


Figure 3.27: Comparison between theoretical mean velocity obtained from Ritter (1892) model (eq. 1.23) and measured velocity profiles for $T = 5$ and $T = 10$ at $X = -0.23$.

The experimental results presented in Figures 3.27 and 3.29 show that the velocity profiles measured for two different initial conditions are practically coincident and so the Froude similitude can also be extended to the velocity profiles.

The differences between the theoretical values and experimental results can be explained by the limitations of Ritter's (1892) model, since it takes into account neither the vertical component of velocity nor the friction with the channel's bed (see Figure 3.30). But as it can be seen from Figures 3.19 and 3.22, for the chosen times one has $U \gg W$ and thus the main reason for the differences can be accounted on the friction effect on the flow.

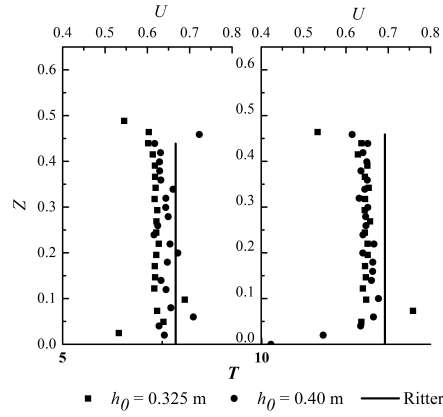


Figure 3.28: Comparison between theoretical mean velocity obtained from Ritter (1892) model (eq. 1.23) and measured velocity profiles for $T = 5$ and $T = 10$ at $X = 0$.

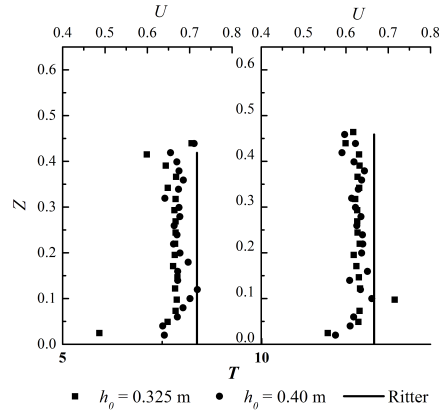


Figure 3.29: Comparison between the theoretical mean velocity obtained from Ritter (1892) model (eq. 1.23) and measured velocity profiles for $T = 5$ and $T = 10$ at $X = 0.38$.

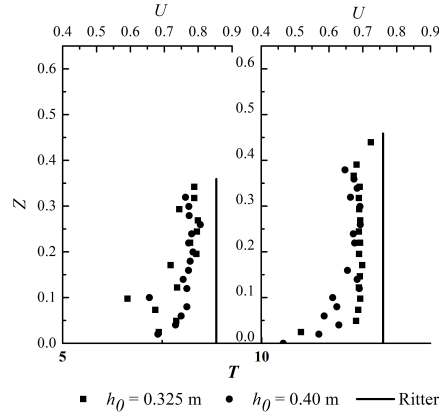


Figure 3.30: Comparison between the theoretical mean velocity obtained from Ritter (1892) model (eq. 1.23) and measured velocity profiles for $T = 5$ and $T = 10$ at $X = 1.41$.

3.2 Study of the Sedimentologic Case

The presence of loose sediments in the channel bed will influence the propagation of the dam-break wave. Momentum and mass transfers between the water column and the bed have now to be taken into account.

The sediment bed will also contribute to a more complex experimental set-up. The bed preparation is therefore a key issue as discussed in chapter 2. The measurement techniques to be used have to be able to discriminate both phases of the flow: solid and liquid, and methods to identify the moving sediment bed have to be put forward.

3.2.1 The Sand Bed Case

In this section the results obtained for the sedimentologic case, i.e., the dam-break flow over a bed of sediments, are presented. This section uses the experiments made with an initial water level, considered above the sediment level to be $h_0 = 0.325$ m, with a sediment height of 0.10 m, characterised by $d_{50} = 1.73$ mm (sand).

Using the convention adopted in section 2.5 they are here referred as h32.5-hsS010.D1.73-BW. Three successful runs made in the same conditions were obtained. The measurements of this section were made with classical PIV with an acquisition frequency of 14.773 Hz. At this stage there was no triggering that allowed for synchronizing the gate with the PIV acquisition, and so the possibility of ensemble averaging was left for later. Nevertheless each run allowed for capturing the dam-break flow at different time steps. In Figures 3.31 to 3.33 PIV images acquired during the initial instants, for $T = 0.44$, $T = 0.67$ and $T = 0.77$ are shown.

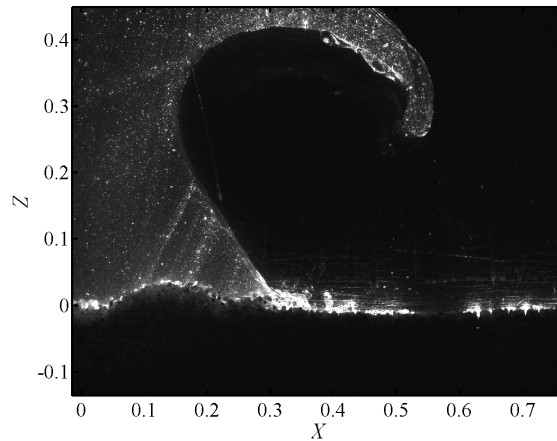


Figure 3.31: Dam-break flow raw image at $T = 0.44$. (Experiment: h32.5-hsS10-D1.73-BW-Run03).

Since the dam-break flow has proven to be quite repeatable (Soares-Frazão and Zech, 2008), this procedure allowed for obtaining a better temporal resolution than the one provided by the utilised PIV system ($\Delta T = 0.37$ which corresponds to the nominal 15 Hz acquisition rate of the classical PIV).

Having no trigger associated with the gate raises the issue of determination of the instant $t = 0$, considered here to be the instant where the gate is completely removed. This was made by launching

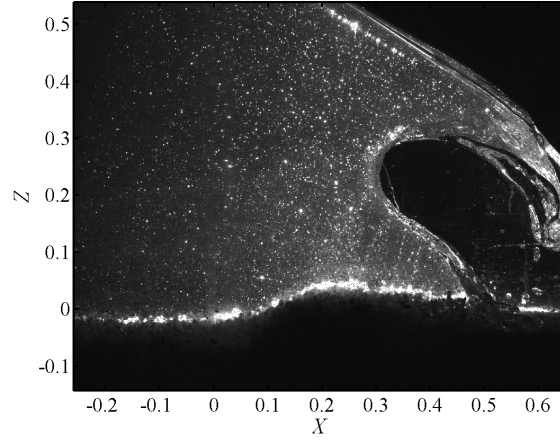


Figure 3.32: Dam-break flow raw image at $T = 0.67$. (Experiment: h32.5-hsS10-D1.73-BW-Run01).

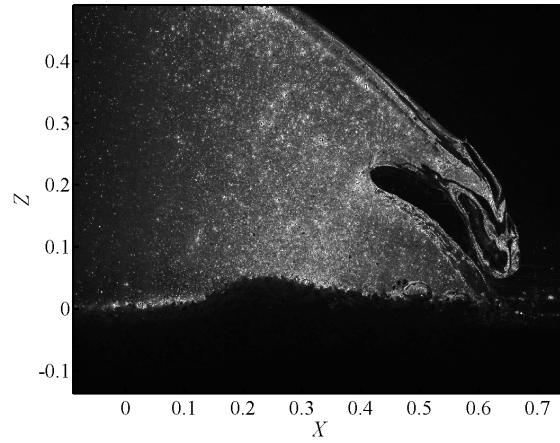


Figure 3.33: Dam-break flow raw image at $T = 0.77$. (Experiment: h32.5-hsS10-D1.73-BW-Run02).

the PIV acquisition before removing the gate, allowing the image of the gate to be captured by the images. From the position of the gate in the images it was possible to determine the instant $t = 0$ by a previously made calibration using a DALSA fast camera (100 fps). Although the three runs had different fields of view, the gate position, that was always visible in the images, allowed the comparison of the spatial distribution of velocity between runs.

Besides the notorious drastic changes in the water surface as seen in Figures 3.31 to 3.33 it is possible to see how the sediment bed is also illuminated by the laser light sheet. This fact inspired the use of the PIV algorithm to obtain the velocity of the sediment bed as well.

3.2.1.1 The Near Field: Water Layer

For the near-field study, six instants were considered, $T = 0.44$, $T = 0.67$, $T = 0.77$, $T = 0.81$, $T = 1.04$ and $T = 1.14$. These instants were obtained from three different runs and since the dam-break flow shows a good repeatability (Soares-Frazão and Zech, 2008) they are put here in sequence in Figures 3.34 to 3.39 to give a perspective and description of the the near field flow evolution.

Figure 3.31 shows the PIV image at $T = 0.44$ while Figure 3.34 illustrates the measured velocities at the same instant ($T = 0.44$). It is possible to see on the same figure the time instants just before the breaking wave. As the gate is pulled down to simulate the dam break, the falling down movement of the top of the water column might be faster than the horizontal water motion. This will mostly depend on the initial water level, the friction between sediment layer and water layer, and the water shear-stress over the sediments. The bed sediments in motion are also captured by the camera, and this will be used later to determine the sediment layer velocity. The velocity vector fields for the remaining instants are

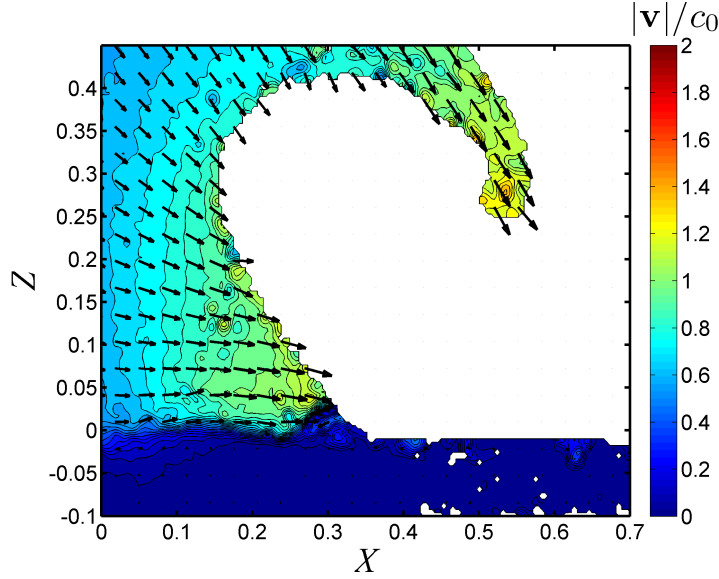


Figure 3.34: Dam-break flow velocity field at $T = 0.44$. For clarity only 1 out of 8 vectors is represented.

depicted in Figures 3.35 to 3.39 respectively. Considering the water layer, it is clear from the colour maps of Figure 3.36 that there is a spatial acceleration of the flow towards the wave front. The water velocity at the front comes close to $|\mathbf{v}|/c_0 \approx 2$ for $X = 0.5$ and $T = 0.77$. From the flow results depicted in Figures 3.34 to 3.39 it is also possible to divide the flow in three regions. The first one defined for $Z \leq 0$ corresponds roughly to the bed evolution and sediment motion. The second one, defined between $0 < Z \leq 0.15$ the water flow seems to be practically horizontal. And finally a third one defined for $Z > 0.15$, where a vertical velocity component is clearly visible in the above mentioned figures.

The velocity vectors angular dispersion is depicted in the polar plot of Figure 3.40, obtained for $T = 0.44$.

One of the characteristics associated with the near-field regime of the dam-break flow is the fact that the velocity field has a strong vertical component. This can be seen by drawing in polar coordi-

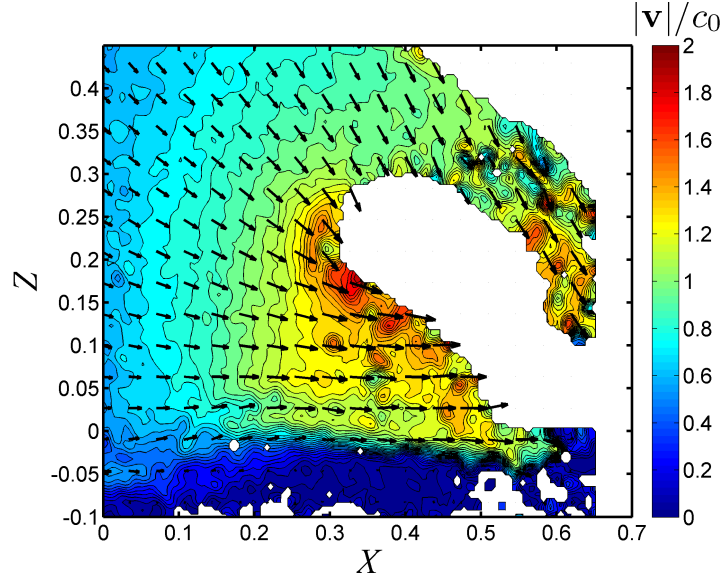


Figure 3.35: Dam-break flow velocity field at $T = 0.67$. For clarity only 1 out of 8 vectors is represented.

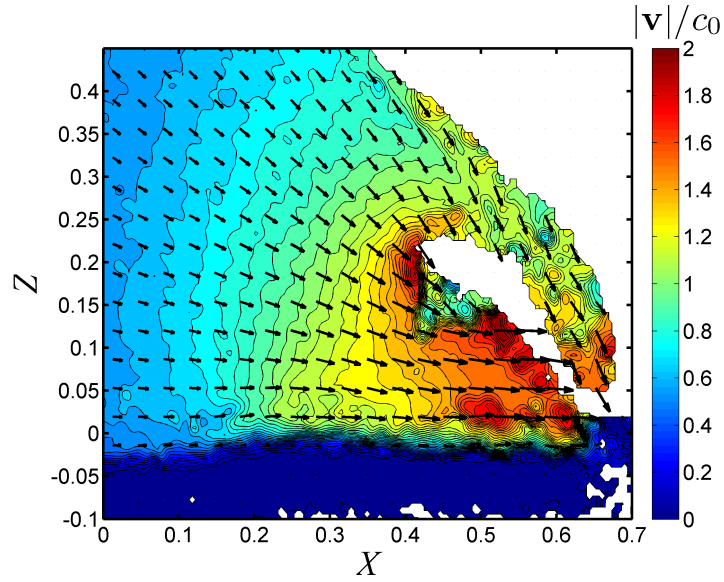


Figure 3.36: Dam-break flow velocity field at $T = 0.77$. For clarity only 1 out of 8 vectors is represented.

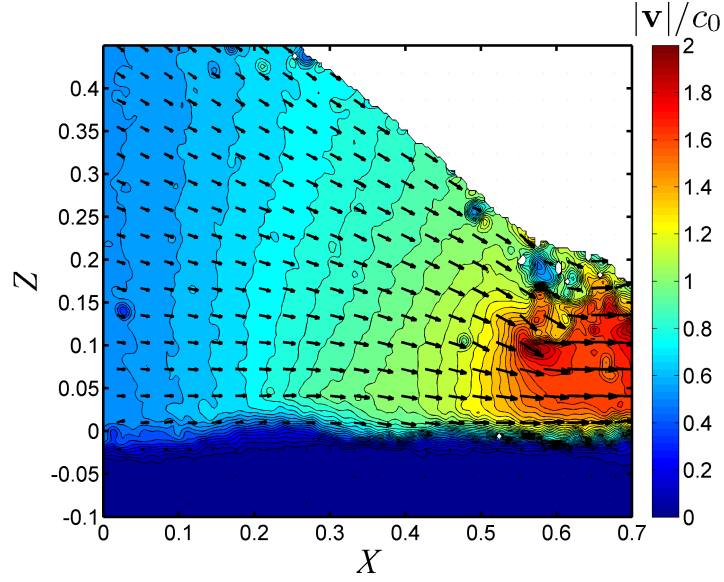


Figure 3.37: Dam-break flow velocity field at $T = 0.81$. For clarity only 1 out of 8 vectors is represented.

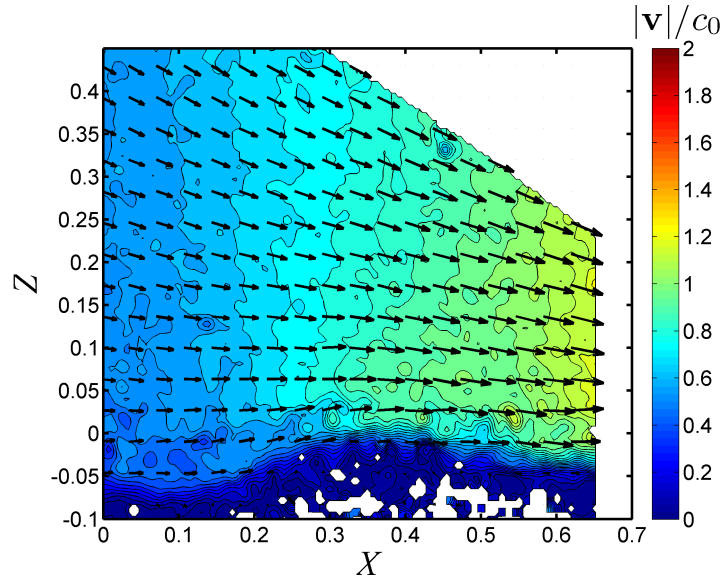


Figure 3.38: Dam-break flow velocity field at $T = 1.04$. For clarity only 1 out of 8 vectors is represented.

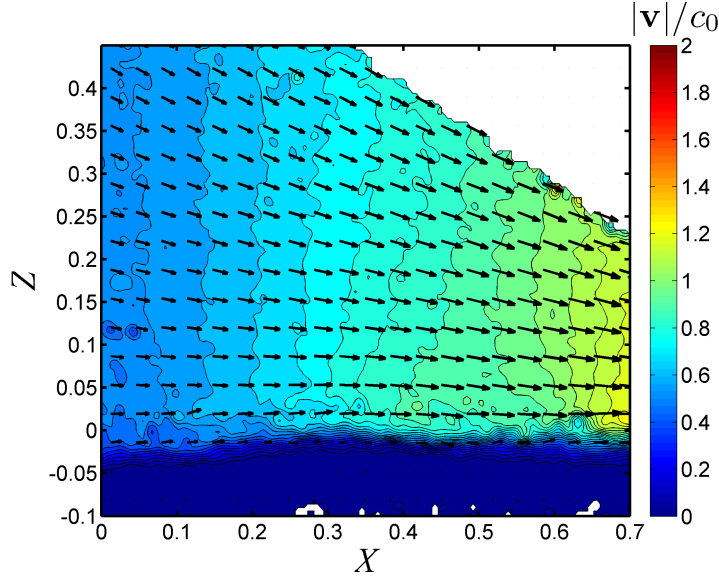


Figure 3.39: Dam-break flow velocity field at $T = 1.14$. For clarity only 1 out of 8 vectors is represented.

nates the velocity field modulus and angle as shown in Figure 3.40, for $T = 0.44$. As it can be seen the majority of the points lie within the 4th quadrant. The points in the 1st quadrant are due to the upward sediment velocity as can be seen Figure 3.31. The sediment velocity field has an important vertical upward component due to the erosion created by the dam-break wave, and to the water velocity field near the bed who to go over the initially produced bed forms. For the considered times $T = 0.44, 0.67$ and 0.77 the velocity profiles for the horizontal and vertical components at the gate section, $X = 0$, are depicted in Figures 3.41 and 3.42 respectively. It is possible to observe that for $T = 0.44$ to $T = 0.67$ the horizontal component does not change significantly, but between $T = 0.67$ and $T = 0.77$ the horizontal component tends to decelerate. At $X = 0$, the flow deceleration is observed in the interval $0.1 < Z < 0.3$.

After the collapse of the wave, the flow tends to evolve towards a semi-permanent flow until the moment when the upstream reservoir

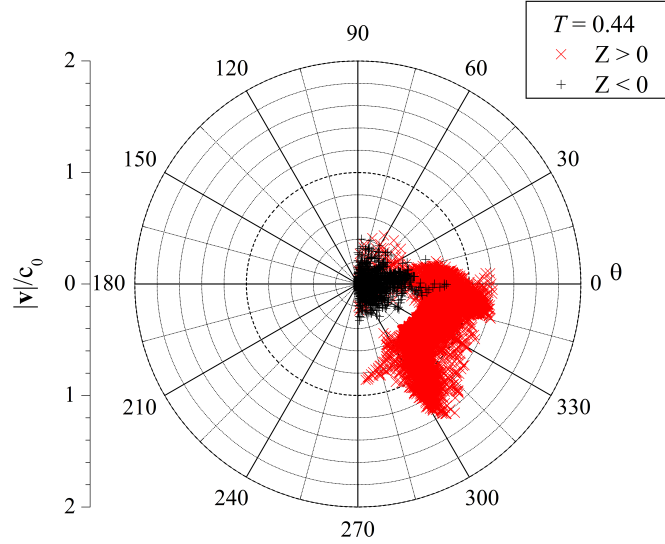


Figure 3.40: Angle and modulus of the velocity vector $\mathbf{v}/c_0$ for $T = 0.44$. Concentric circles represent a value of constant velocity modulus, whose scale is represented by the vertical line adjacent to the plot. Only 1 out of 8 vectors is depicted.

can no longer be considered as infinite. It is then possible to observe that, in the imaged region, the water surface does not change as drastically as it did in the first instants. In Figures 3.37 to 3.39 the velocity fields at $T = 0.81$, $T = 1.04$ and $T = 1.14$ are depicted. It is seen from those images the flow acceleration with increasing time. Near the bed region it is also possible to identify some velocity vectors with an upward velocity component, suggesting that the flow adapts to the bed geometry.

In Figures 3.41 and 3.42 it is possible to observe how the velocity profiles for both horizontal, U , and vertical W components evolve in the first time instants at the gate section. It is also possible to see a spatial acceleration of the flow put into evidence in Figure 3.43, where for $T = 1.04$ different velocity profiles are plotted. Although

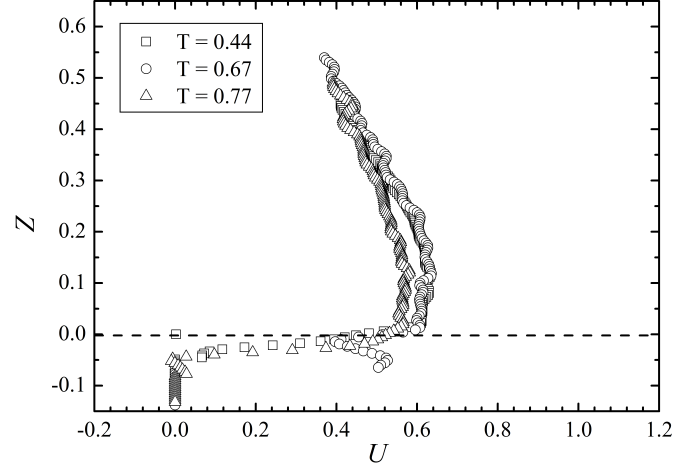


Figure 3.41: Velocity profiles for the horizontal component U at the gate section, $X = 0$ for $T = 0.44$, 0.67 and 0.77 . The dashed line indicates the initial bed level.

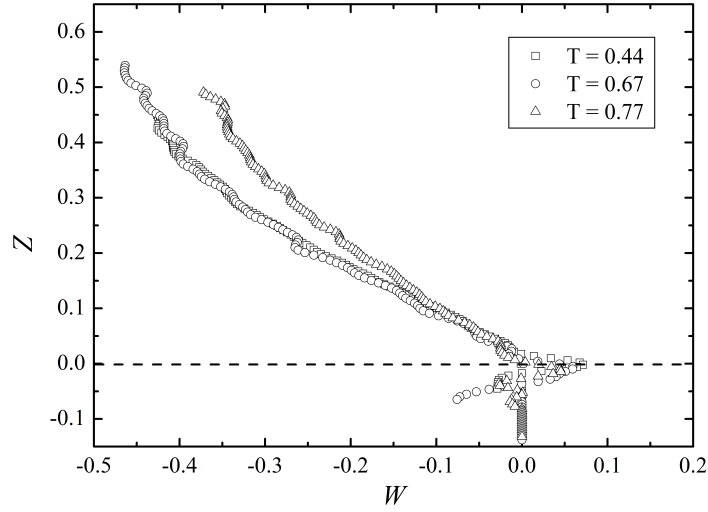
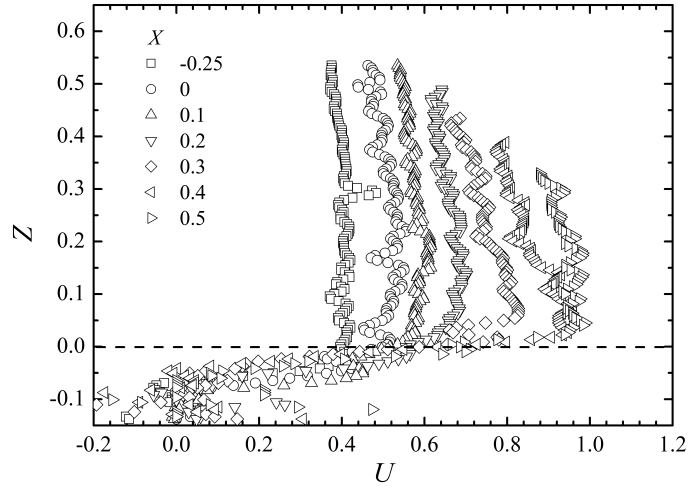
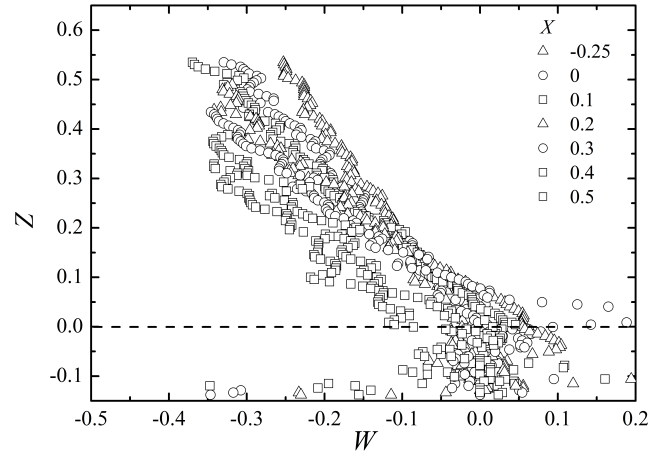


Figure 3.42: Velocity profiles, vertical component W , at the gate section, $X = 0$ for $T = 0.44$, 0.67 and 0.77 . The dashed line indicates the initial bed level.



(a)



(b)

Figure 3.43: Velocity profiles for $T = 1.04$ for different values of X (a) U . (b) W .

the vertical component, W , has more or less the same behaviour, the horizontal component, U has a strong acceleration ranging from $U \approx 0.4$ at $X = -0.25$ to $U \approx 1$ at $X = 0.5$. The distribution of velocity vector directions is depicted in Figure 3.44. Although there is still a vertical component of the velocity field, it is possible to see when comparing with Figure 3.40 that the angle is smaller, indicating that the flow tends to be horizontal in the long term. It is also clear from Figure 3.43 that the horizontal component of the velocity profile, $U = U(z)$, in the water layer is almost constant for each considered section, although different from section to section, as it will be seen later the profiles will tend to a constant value.

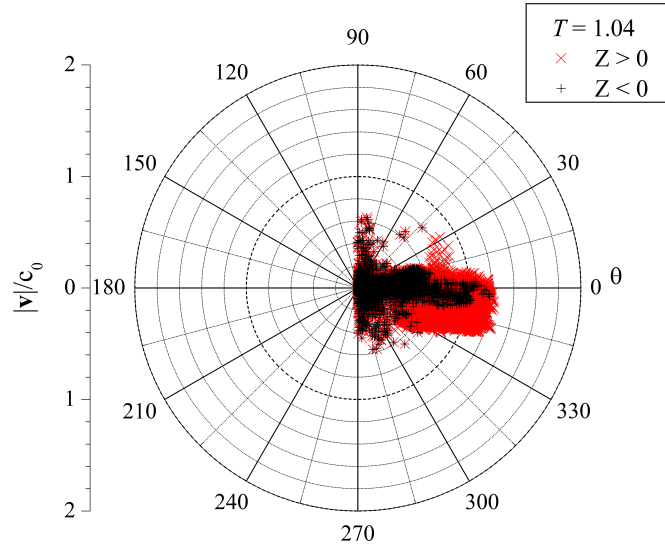


Figure 3.44: Angle and modulus of the velocity vector $|\mathbf{v}|/c_0$ for $T = 1.04$. Concentric circles represent a value of constant velocity modulus, whose scale is represented by the vertical line adjacent to the plot.

3.2.1.2 Evolution Towards a Shallow-Water Flow: Water Layer

The dam-break flow is continuously evolving in time, and the flow characteristics change accordingly. For the Hydrodynamic case the time limit for which the flow starts behaving like a shallow water flow was set at $T \approx 2.5$. Analysing the respective time instant for the sedimentologic case the velocity profiles one has in Figure 3.45 the velocity field at $T = 2.52$, and in Figure 3.46 the velocity vectors angle distribution. It is possible to see that now the flow is practically horizontal. From the obtained velocity profiles (Figures 3.47 and 3.48) it is also possible to conclude that the horizontal gradient of the velocity $\partial U / \partial X$ is now much smaller when compared with the situation depicted in Figure 3.43 corresponding to $T = 1.04$. It is also possible to notice that the vertical component is now smaller and that $U \gg W$.

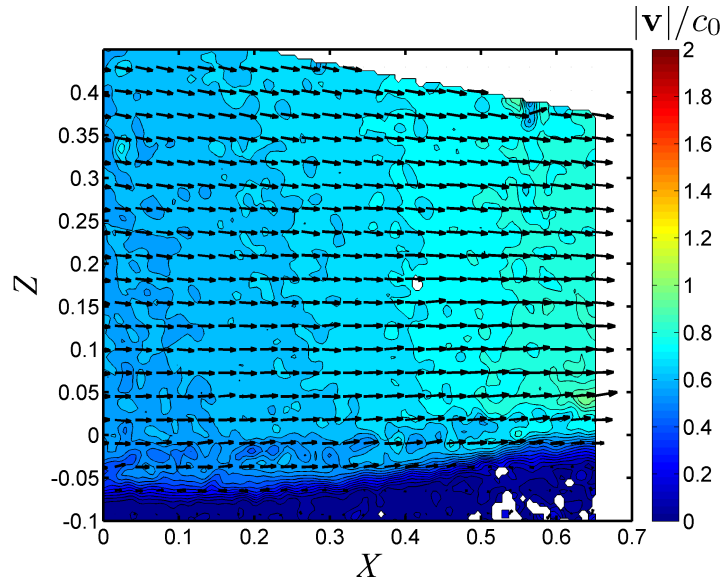


Figure 3.45: Measured dam-break flow velocity field at $T = 2.52$.

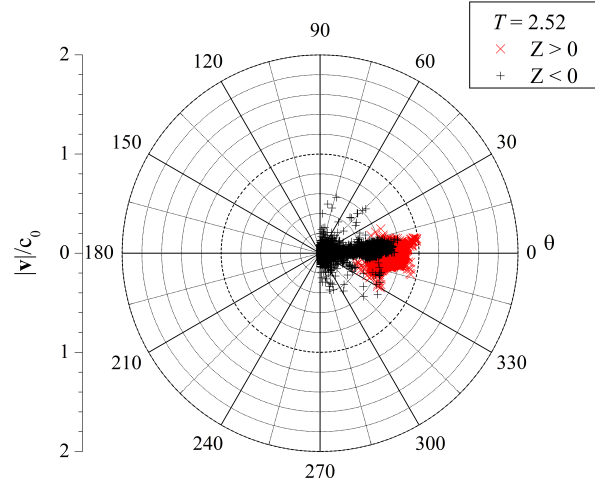


Figure 3.46: Obtained angle and velocity modulus distribution for $T = 2.52$.

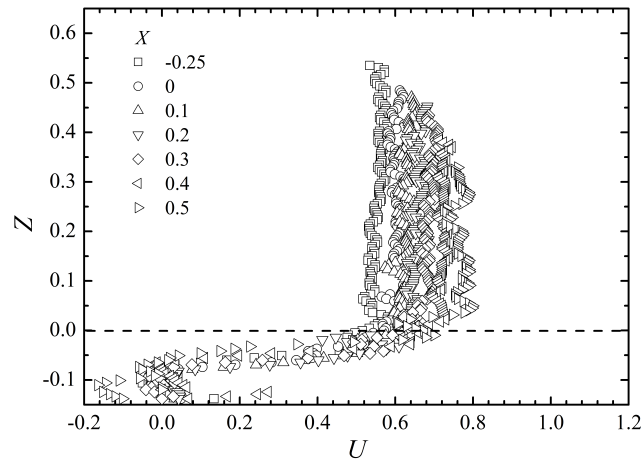


Figure 3.47: Horizontal velocity component U profiles for $T = 2.52$ for different values of X .

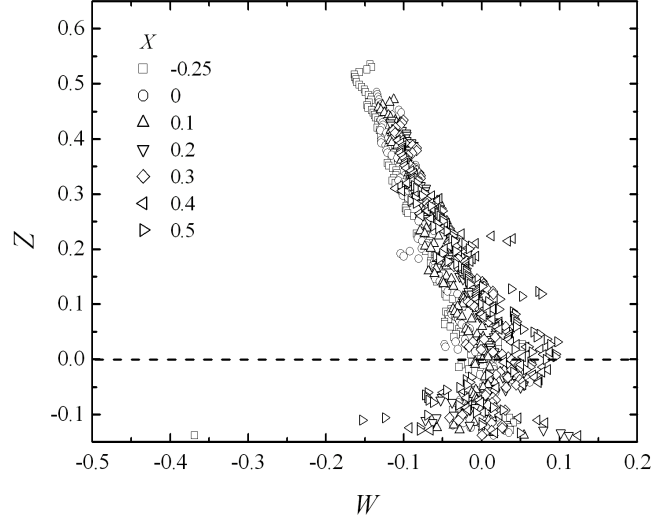


Figure 3.48: Vertical velocity component W profiles for $T = 2.52$ for different values of X .

3.2.1.3 The Shallow Water Regime: Water Layer

For $T \approx 5$ the flow tends to be established in the imaged region as depicted in Figures 3.49 and 3.50, where the presented color maps are practically uniform. It is quite interesting to see the flow evolution from the first instants where there is a variation in time and space to the later instants where the flow acquires some properties of established open-channel flows.

The velocity profiles depicted in Figure 3.51 show that all of them are practically coincident and that, near the bed, the flow velocity profile is quite linear.

On the other hand, the vertical component is now practically zero over the entire depth (Figure 3.52). This is also seen in Figure 3.53, where the velocity vector angle distribution is a cloud around 0° .

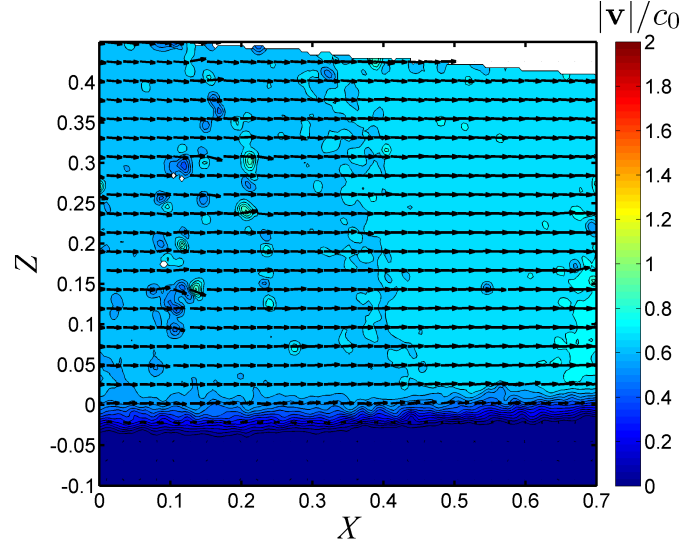


Figure 3.49: Sedimentologic dam-break flow at $T = 4.90$. For clarity only 1 out of 8 vectors is plotted.

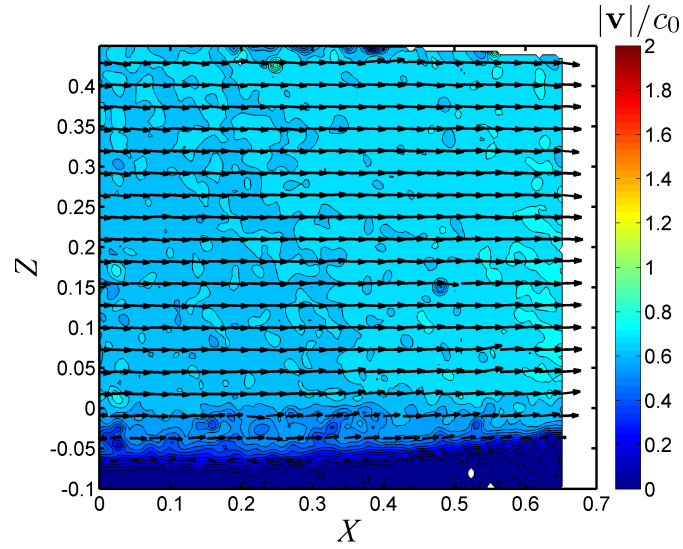


Figure 3.50: Sedimentologic dam-break flow at $T = 5.13$. For clarity only 1 out of 8 vectors is plotted.

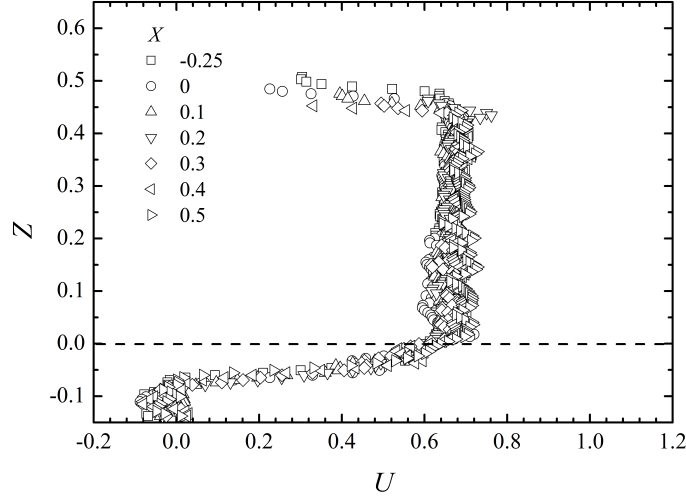


Figure 3.51: Velocity profiles of the horizontal component U , for $T = 5.13$ at different values of X .

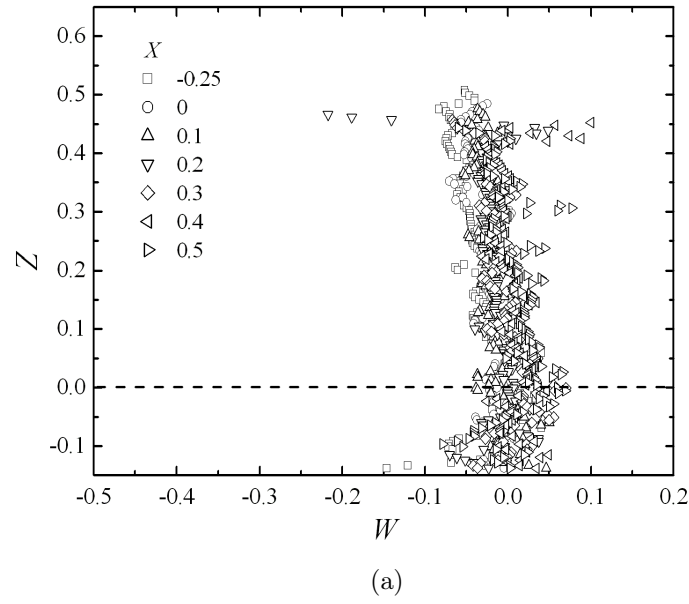
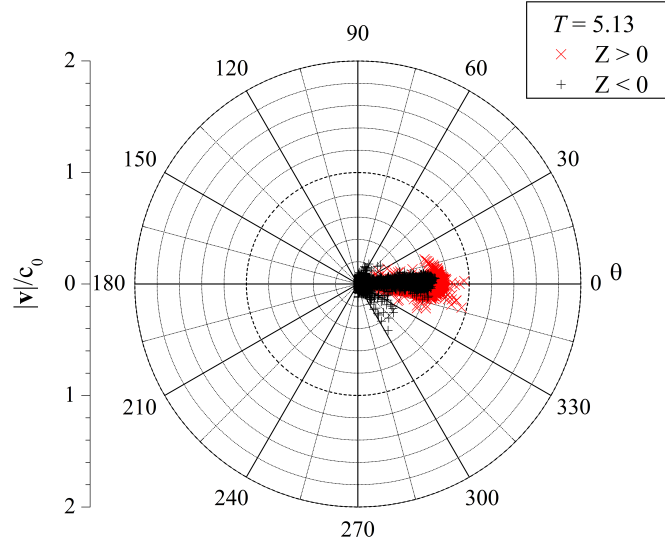


Figure 3.52: Velocity profiles of the vertical component W for $T = 5.13$ for different values of X .

Figure 3.53: Angle distribution for $T = 5.13$.

3.2.2 Analysis of the Sediment Layer

3.2.2.1 Interface Identification

One of the major problems when dealing with sediment flow is to determine the location of the sediment transport layer. Besides the information concerning the velocity field it is also important to know the dimensions of each layer to allow, for instance, a proper determination of the solid discharge. This information was attempted to be obtained from the PIV images and this attempt method is put forward here.

To determine the presence of moving elements between two consecutive frames, one of the easiest methods is image subtraction, i.e., two consecutive images are subtracted in order to identify which regions have moved. However, due to the high concentration of seeding for the PIV measurement associated with the fact that in

classical PIV the first image (Figure 3.54) is always darker than the second (3.55), the results were not as expected (Figure 3.56), where the information about the interfaces is not quite visible.

The fact that it is not easy to identify the sediment layer by means of a simple image subtraction led to the research of another process that could be used to determine the sediment layer location.

As seen in subsection 1.5.3 the two-layer model was proposed as an intuitive way to address the fact that the water and sediments could be interpreted as two different fluids moving with different velocities. Here, this fact is explored with the goal of finding a method to identify each flow layer. Two fluids moving with respect to each other with different velocities, will originate a shear layer in their interface (e.g Townsend (1976)). The fact that one can see the sedimentologic dam-break flow as two moving fluids (water and sediment) that is, as a shear layer flow (Capart and Fracarollo, 2011) suggests the use of the vorticity as a way to identify the shear layer. Let,

$$\boldsymbol{\omega} = \nabla \times \mathbf{v} \quad (3.2)$$

be the curl of the velocity field, and Ω the non-dimensional curl of the velocity defined as

$$\Omega = t_0 \nabla \times \mathbf{v} \quad (3.3)$$

where $t_0 = \sqrt{h_0/g}$ as defined in subsection 1.5.1.

Calculating the non-dimensional curl of the velocity field for the dam-break flow at $T = 5.13$ one has the results presented in Figure 3.57. As presented, these results may seem difficult to interpret. By taking the absolute value of equation (3.3) and computing it for the dam-break flow at $T = 5.13$ one has the results depicted in Figure

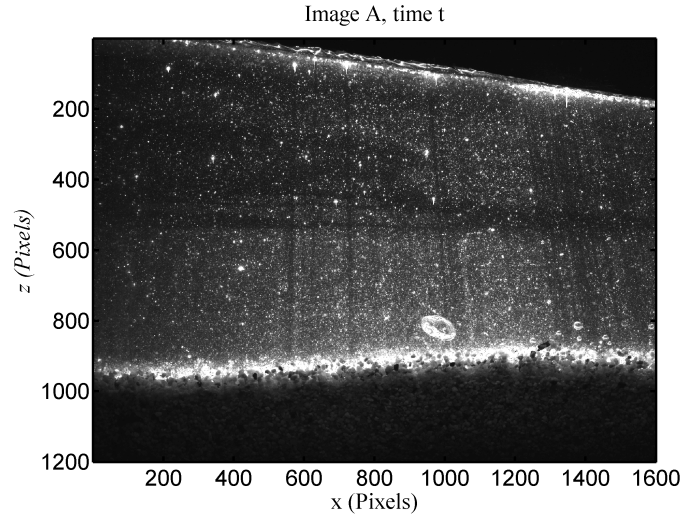


Figure 3.54: First image (Image A) from the PIV pair of images taken at $T = 5.13$.

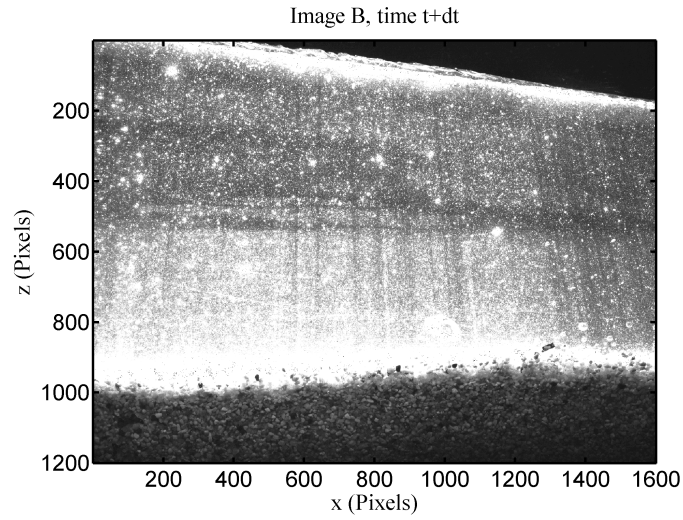


Figure 3.55: Second image (Image B) from the PIV pair of images taken at $T = 5.13$.

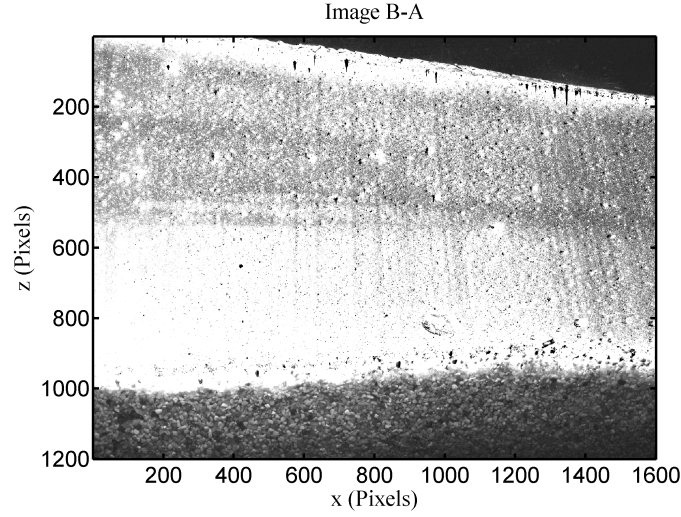


Figure 3.56: Subtraction of the two images: Image B (Figure 3.54) - Image A (Figure 3.55).

3.58. As it can be seen, the water layer is essentially irrotational, i.e., $|\Omega| = 0$ and the flow can be assumed as potential flow. However near the bed one has $|\Omega| \neq 0$.

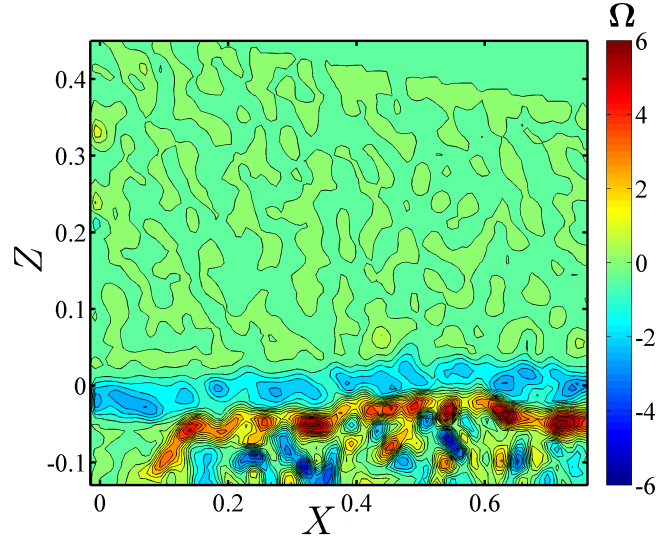


Figure 3.57: Contour plot of Ω obtained for $T = 5.13$.

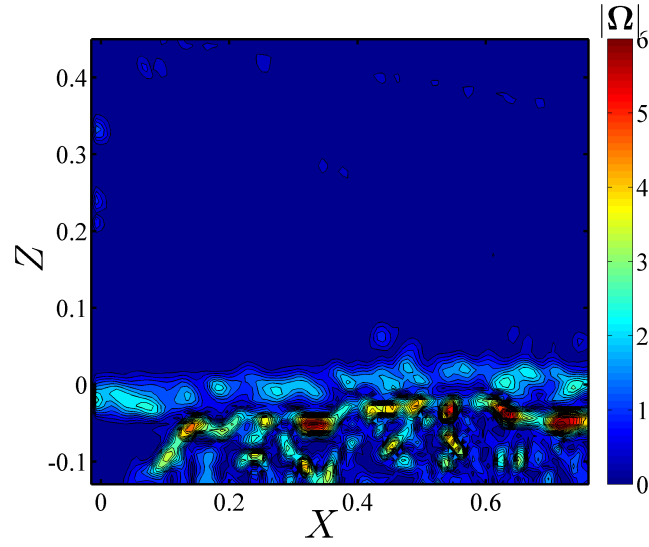


Figure 3.58: Absolute value of the non-dimensional velocity curl, $|\Omega|$, calculated for the dam-break flow at $T = 5.13$.

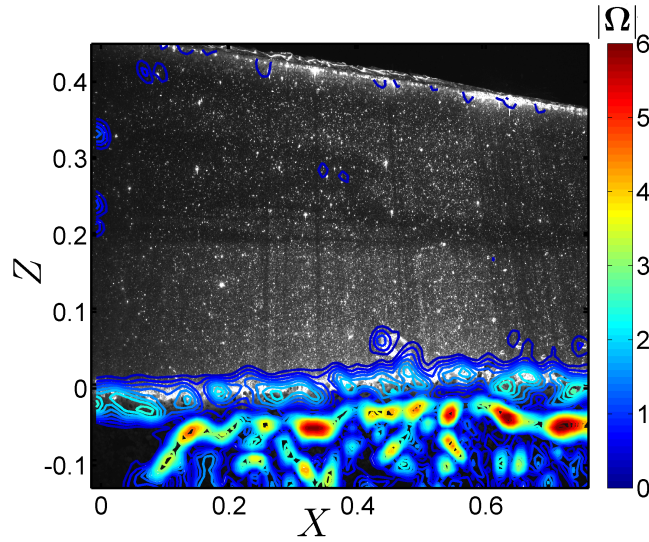


Figure 3.59: Contour map of $|\Omega|$ superimposed on the corresponding flow image for bed identification. Values of $|\Omega| = 0$ were made transparent for clarity. This method seems to be useful to estimate the sediment layer centre but not its limits.

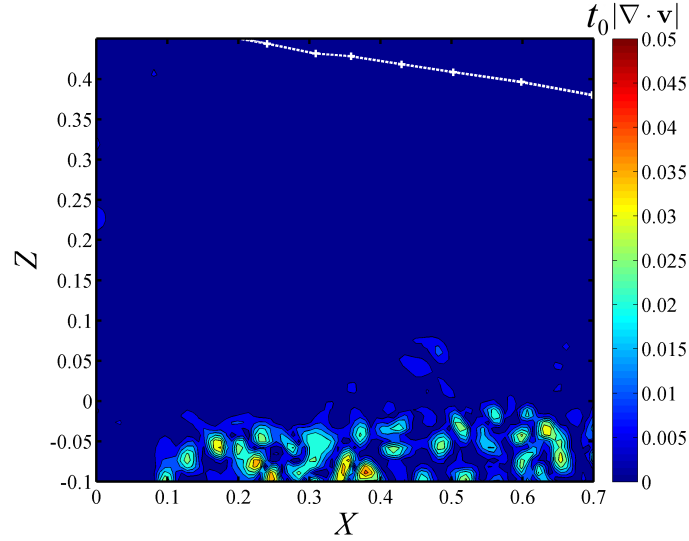


Figure 3.60: Contour map of $|t_0 \nabla \cdot \mathbf{v}|$ (white dash line depicts the free surface).

As it can be seen from Figure 3.58 the higher values of $|\Omega|$ are observed near the bed. The superposition of a flow raw image, as shown in Figure 3.59, with the computed values of $|\Omega|$ shows that there is a correspondence between the velocity curl contours and the sediment layer location. To assess the limits of the sediment layer the velocity profile, $u = u(z)$, will be used, but before, to check the velocity field consistency the absolute value of the divergence of the velocity is plotted in Figure 3.60.

The velocity curl contour plot, together with the velocity profiles for both water and sediment layers, can be used to identify and track the moving sediment layer. With this in mind it is possible to extract the vertical profiles of $|t_0 \nabla \times \mathbf{v}|$ and compare it with the velocity profile. The reason for doing this is to search for an indicator of the sediment layer limits using both the information from the velocity profile and from the curl of the velocity. Another reason to use the velocity profile information relies in the identification of spurious values in the velocity profile that may lead to

erroneous values of the vorticity as depicted in Figure 3.61, and that are therefore removed. One of the reasons for this to happen is the poor lighting conditions of the sediment bed as can be seen in Figure 3.54 (a).

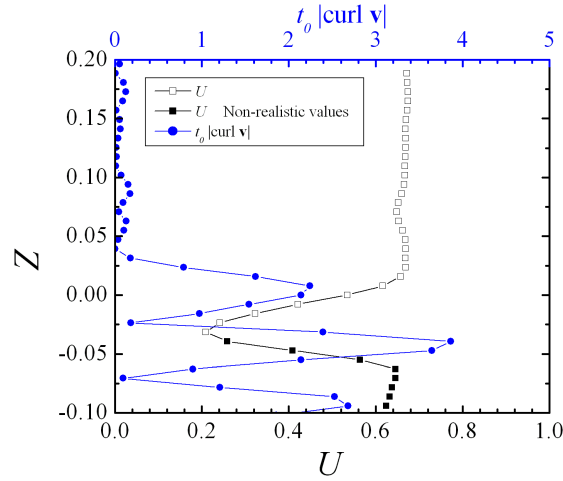


Figure 3.61: Profile of $|t_0 \nabla \times \mathbf{v}|$ and U at $X = 0.313$ and $Z < 0.2$ for emphasis in the near bed region. The velocity profile is used to identify non-physical values (filled black dots) which will be disregarded.

For three different locations, $X = 0$, $X = 0.313$ and $X = 0.6$ the non-dimensional horizontal velocity profile and the vertical profile of $t_0 |\nabla \times \mathbf{v}|$ are computed and represented in Figures 3.62 to 3.64.

It can be seen that the gradient shows a peak (P) near the bottom, and one can identify two other points, here referred as B and T , corresponding to the local minima in the vicinity of peak, P . The determined points are plotted over the corresponding flow image in Figure 3.65. As it is seen in Figure 3.65 the region enclosed by the coordinates of B and P corresponds roughly to the sediment layer. Care must be taken however in using this method. It is not assured to be universal, neither precise. However it can be use as an estimator or predictor for the sediment layer location.

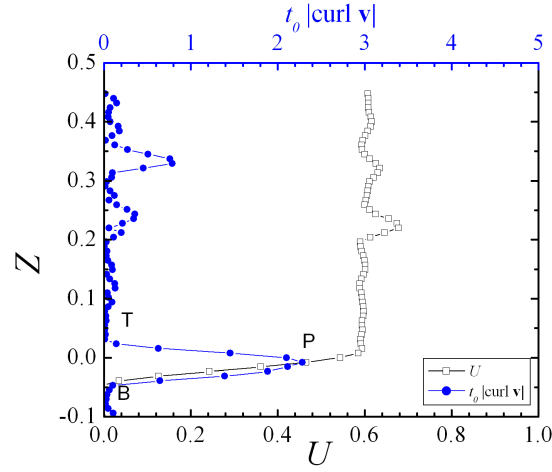


Figure 3.62: Profile of $t_0|\nabla \times \mathbf{v}|$ and U at $X = 0$ without the non-physical values.

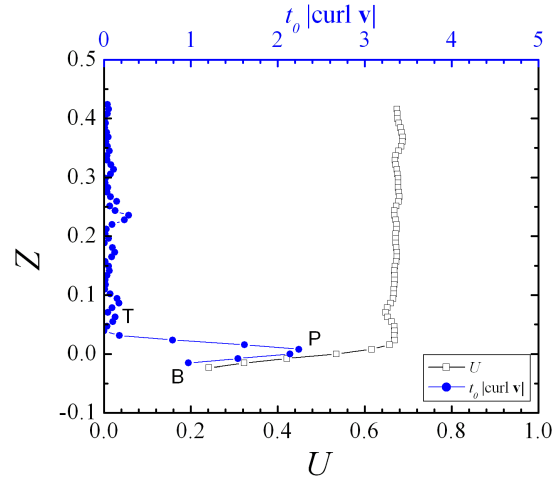


Figure 3.63: Profile of $t_0|\nabla \times \mathbf{v}|$ and U at $X = 0.313$ without the non-physical values.

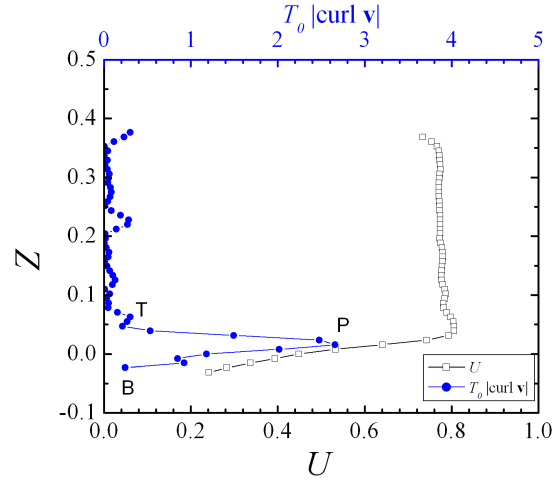


Figure 3.64: Profile of $t_0 |\nabla \times \mathbf{v}|$ and U at $X = 0.695$ without the non-physical values.

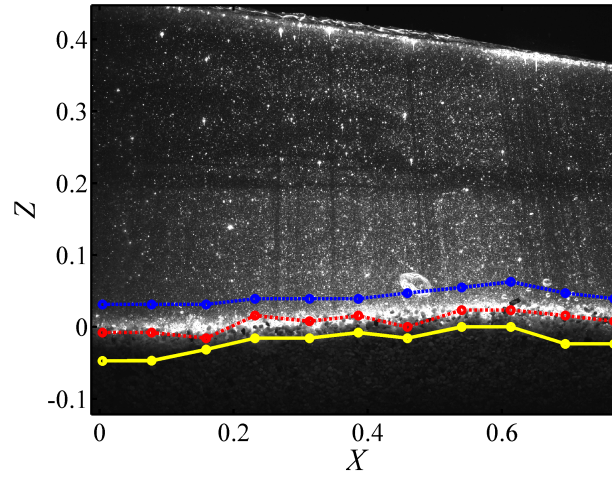


Figure 3.65: Superimposition of the points B , P and T on a flow image. Points B solid yellow line. Points P dashed red line. Points T dotted blue line.

3.2.2.2 Sediment Layer Velocity Field

Using the flow images the velocity of the sediment layer was estimated by means of the PIV. The velocity fields for the instants $T = 1.04$, $T = 2.53$ and $T = 5.13$ are depicted in Figure 3.68. It is possible to see how, for the first instants, the sediments have a significant vertical component, and again, it becomes less and less significant.

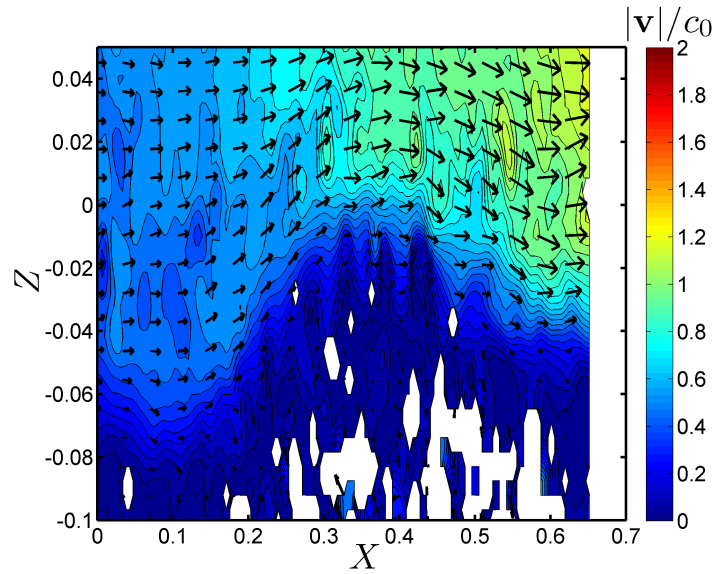


Figure 3.66: Velocity fields of the bed region for different instants at $T = 1.04$. (Only 1 out of 8 vectors represented.)

For the non-dimensional time $T = 5.13$ the velocity profiles of U and W at different locations are plotted in Figures 3.69 and 3.70. It is possible to observe that, although the vertical component is practically $W \approx 0$ (Figure 3.70), the horizontal component is not constant along the sediment layer, rather having a curve-shaped profile as depicted in Figure 3.69 indicating a shear layer behaviour.

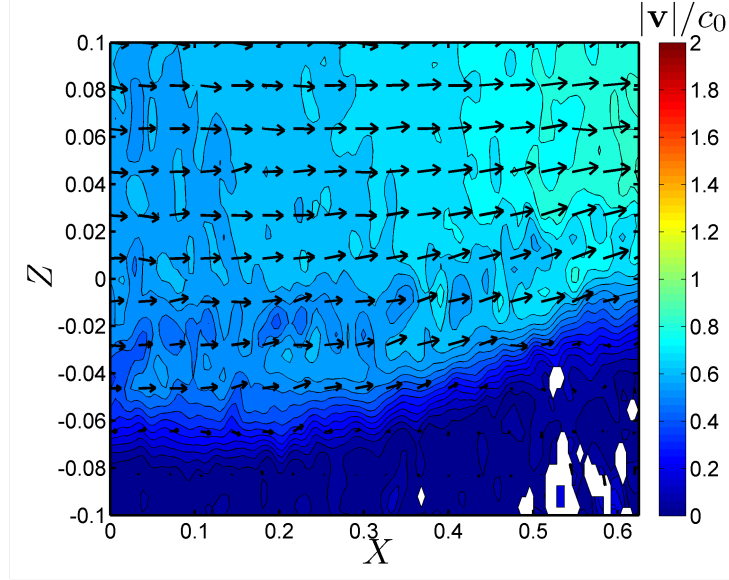


Figure 3.67: Velocity fields of the bed region for different instants at $T = 2.52$. (Only 1 out of 8 vectors represented.)

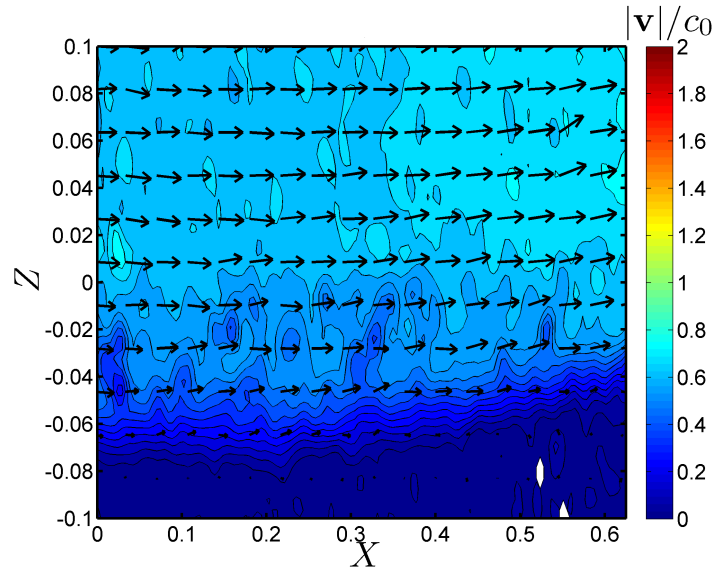


Figure 3.68: Velocity fields of the bed region for different instants at $T = 5.13$. (Only 1 out of 8 vectors represented.)

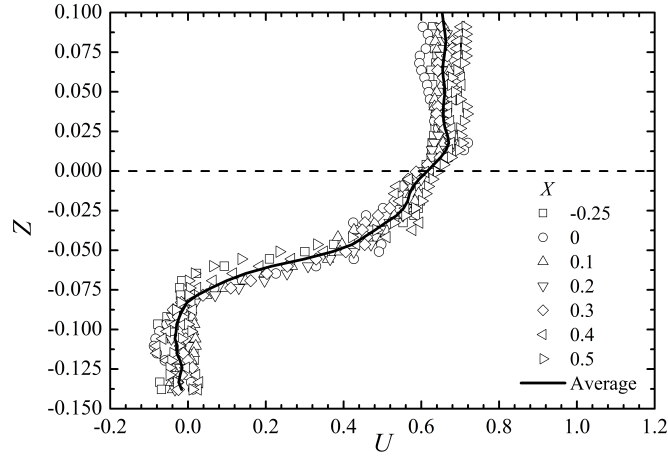


Figure 3.69: Velocity profiles, horizontal component U , measured at different sections of the bed for $T = 5.13$. Solid line is the average of all the plotted profiles.

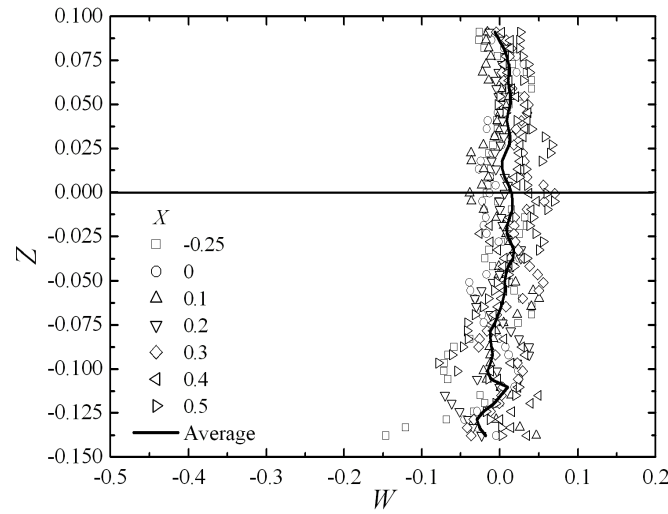


Figure 3.70: Velocity profiles, vertical component W , measured at different bed sections of the bed for $T = 5.13$. Solid line is the average of all the plotted profiles.

3.2.3 Comparison Between Numerical and Experimental Results

In this subsection a comparison between the results provided by the two-layer model presented in subsection 1.5.3 and the obtained experimental results is made. This subsection was the result of the collaboration with C. Swartenbroekx, who provided by the numerical analysis. The results here presented have been published in Swartenbroekx et al. (2011).

As presented in subsection 1.5.3 the two-layer model hypothesis concerning the velocity profiles in the water layer and sediment layer assume that both profiles are constant. The measurements show however that, if for the water layer this assumption is valid, for the sediment layer that is not the case. In the sediment layer the velocity profile tends to be linear. Furthermore, no discontinuity is found in the interface between the two-layers.

In the presented two-layer model two friction coefficients: C_{fs} and C_{fw} need to be calibrated. Since measurements of both coefficients are difficult to perform, a method to calibrate them based on solid discharge expressions (e.g Meyer-Peter and Muller (1948)) was proposed by Savary and Zech (2007) is used. Expressions like the ones of Meyer-Peter and Muller (1948) are valid for uniform stationary flow which is not the case of the dam-break flow. In the present case the PIV results are used to evaluate both solid and liquid discharges, q_w and q_s which are then compared with the discharges obtained numerically for different combinations of friction coefficients pairs, C_{fs} and C_{fw} . The method presented in section 3.2.2 is used to estimate the sediment layer height, h_s , and the flow images (e.g. Figure 3.33) provide the information about the water height, h_w . The solid and liquid discharge per unit width, q_s and q_w , are calculated respectively by:

$$q_s = \int_{z_b}^{h_s} u_s dz \quad (3.4)$$

$$q_w = \int_{h_s}^{h_w} u_w dz \quad (3.5)$$

where u_s and u_w are the velocity fields of the sediment and water layer respectively.

This model uses a finite volume discretization. In order to maintain the system hyperbolicity (Spinewine et al. (2011)) a modified first-order HLL scheme is used as proposed by Swartenbroekx et al. (2010) for the interface flux calculation. The grid size was 3 mm and the Courant-Friedrichs-Lewy number imposed was, for this case, $CFL = 0.9$. Different values for C_{fs} and C_{fw} were tested with the two-layer model. According to Swartenbroekx et al. (2011) the best agreement between experimental and numerical results was obtained with $C_{fs} = 0.06$ and $C_{fw} = 0.008$, and these were the values used for the simulations.

Three different times are used for comparison between the numerical and experimental results: $T = 1.48$, $T = 2.97$ and $T = 6.32$. The method proposed in section 3.2.2 was used to identify and determine the coordinates for the bed and transport layer. The results comparison are depicted in Figures 3.71 to 3.73. It is possible to see that for $T = 1.48$ the water level results do not agree at all, whereas for $T = 2.97$ and $T = 6.32$ the numerical results for the water level compare well with the experimental data (less than 7% difference). The discrepancies observed for $T = 1.48$ can be explained for the fact that the model is being applied to a situation where the shallow water approach is not yet valid. As presented in subsection 1.5.3, the two-layer model is a shallow-water model; and as it was seen in section 3.2.1.2, the shallow water regime starts for $T > 2.5$.

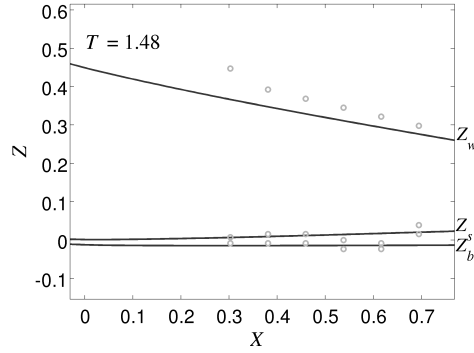


Figure 3.71: Comparison between the experimental and numerical results obtained with the 2-Layer model described in subsection 1.5.3 for $T = 1.48$. Z_w denotes the non-dimensional water level, Z_s the non-dimensional sediment layer height and Z_b the non-dimensional bed level. Dots: experimental data. Lines: numerical results.

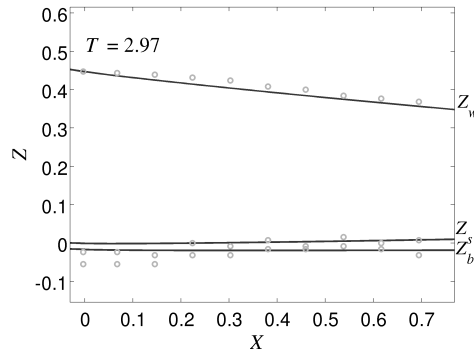


Figure 3.72: Comparison between the experimental and numerical results obtained with the 2-Layer model described in subsection 1.5.3 for $T = 2.97$. Z_w denotes the non-dimensional water level, Z_s the non-dimensional sediment layer height and Z_b the non-dimensional bed level. Dots: experimental data. Lines: numerical results.

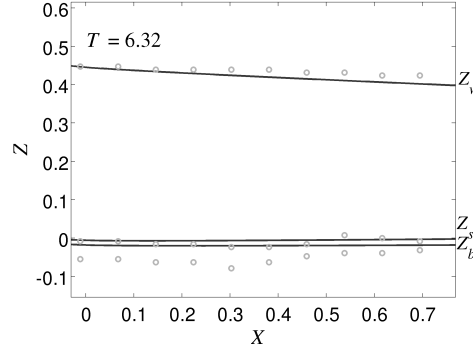


Figure 3.73: Comparison between the experimental and numerical results obtained with the 2-Layer model described in subsection 1.5.3 for $T = 6.32$. Z_w denotes the non-dimensional water level, Z_s the non-dimensional sediment layer height and Z_b the non-dimensional bed level. Dots: experimental data. Lines: numerical results.

The results for the bed level and sediment layer show that the model results don't diverge significantly from the obtained experimental data, except for the region near the gate and Z_b in Figure 3.73. From Figures 3.31 to 3.33 it is possible to see that, due to the water-sand interaction, a bed form, created by the bed erosion appears, and that for $X < 0.3$, the bed erosion is important.

3.2.4 The PVC Pellets Bed Case

Another set of measurements was made considering a bed made of PVC pellets ($h_s = 0.10$ m and an initial water height of $h_0 = 0.325$ m, see Table 2.3). As referred in section 2.5 the main reason for the use of PVC pellets was its smaller weight. On the other hand, due to its dimension, they are also easier to identify. For these measurements, besides the classical PIV measurements, the time-resolved PIV (TR-PIV) approach was used. The images were acquired by means of the fast camera Photron (see section 2.4.2), set to an acquisition frequency of 1000 images per second at 1 Mpixel

of resolution. These section presents the measurements carried out on the dam-break flow over a PVC pellets bed. In a first stage the symmetry of the flow in the channel is investigated by means of the classical PIV and in the second stage, the Time-Resolution PIV approach is used to obtain a time description of the dam-break flow over an idealised sediment bed. A study of the flow's transversal uniformity was also made using the pellets sediment bed and it is presented in Annex C.

3.2.4.1 Time Evolution Analysis of the Dam-Break Flow Over a PVC Pellets Bed

In this section the dam-break flow evolving over a PVC pellets bed is analysed using a high speed camera. This technique allows for an increased time resolution while ensuring a sufficient spatial resolution to provide a detailed analysis of the initial stages of the dam-break flow.

The classical PIV acquisition frequency of 15 Hz was not enough to obtain a proper temporal description of the flow as it was already seen in section 3.2.1. To solve this limitation, the classical PIV system was replaced by a Photron high speed camera whose properties can be found in section 2.4.2.

The fact that the a high speed camera was used introduced other changes in the set-up. A continuous, and less power laser ($P = 40 \text{ mW}$) was used insted of a pulsed laser. Its major drawback was allowing to light only flow seeded particles at $Y = -0.4W$ (i.e. 2.5 cm from the channel window closest to the camera, see Figure 2.59). Nevertheless the gain in time resolution allowed a more precise description of the dam-break flow's initial stages. Parts of this section have been presented in Aleixo (2011).

As depicted in Figure 3.74 and Figure 3.75 the high-speed camera allowed to perceive more features of the initial stages of the

dam-break, namely the water wave collapse in Figures 3.74 (e)-(f) and 3.75 (a)-(d).

Due to the finite gate speed it is possible to see in Figure 3.74 (a) and (b) that the flow starts before the complete removal of the gate, resembling a weir-type flow as described in chapter 3. The upper layer of the water column will therefore start its movement earlier. Due to the bed friction and the momentum transfer between water and sediments, the water column layer close to the bed will be slowed down; this is clearly seen in Figures 3.74 (c) and (d). The same Figures show that, due to the momentum transfer the sediments will be mobilised. The bed deformation will induce the ski jump like structure from which a water and sediment jet is launched (Figure 3.74 (e) and (f)). The jet and the collapsing wave collide as shown in Figure 3.75 contributing for the flow's energy dissipation.

Also worth mentioning is the fact that the *lobster claw* shape geometry of the water layer (Figure 3.74 (e)) will entrain air into the water stream, as can be seen in Figure 3.75 (b)-(d) in the overexposed region. Although wave breaking and air entrainment are two very important phenomena (Lamarre and Melville, 1991), they will not be here analysed in depth for the case of the dam-break flow.

It is important to notice the bed deformation with the dam-break wave propagation which is in this case clearly amplified when compared to the sand bed case. The deformed bed geometry induces the appearance of the upward jet as depicted in Figure 3.74 (e). This was not observed in the measurements made over the sand bed (Figures 3.31-3.33) due to the fact that sand is heavier and more compacted than the PVC pellets. The water layer coalescence mechanism is a significant difference between the two sedimentologic cases. For the sand bed case, the upper water layer collides ($Z > 0.15$) with the inferior water layer in the interval $0.77 < T < 0.81$. For the pellets case, due to the formation of

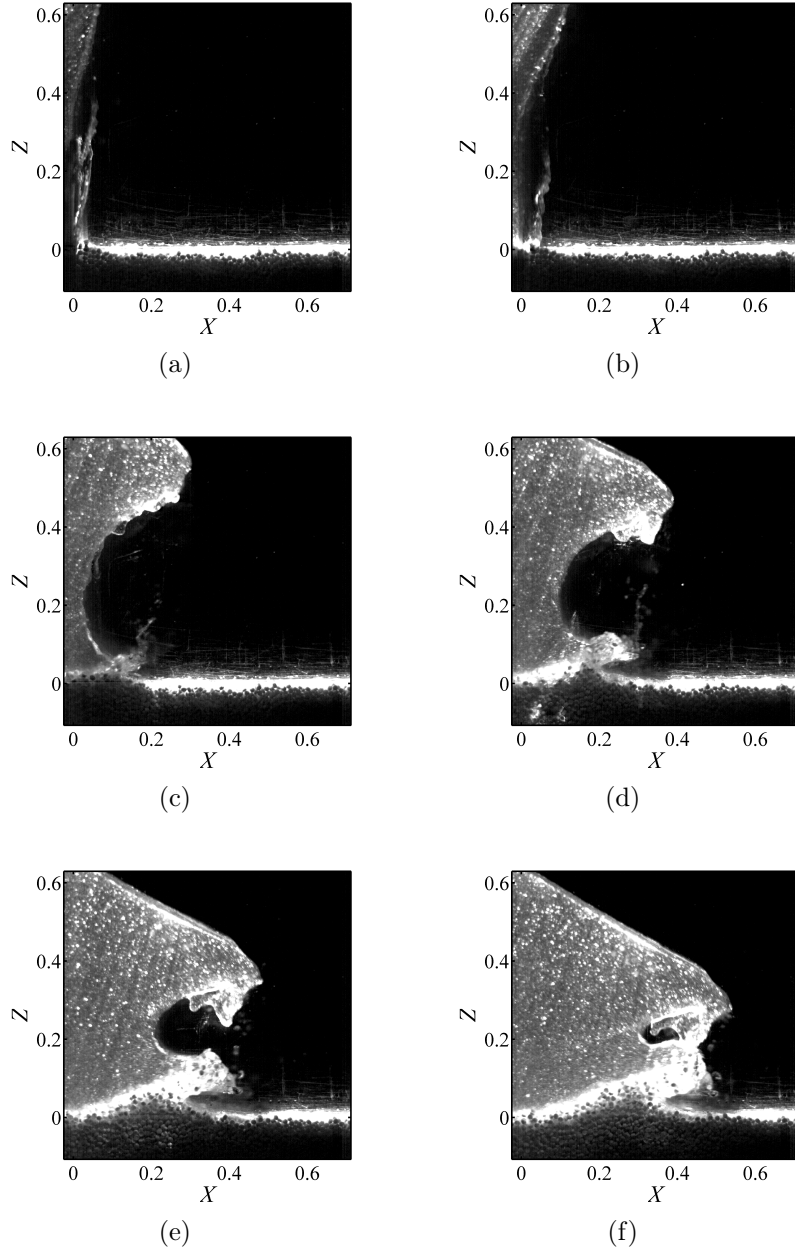


Figure 3.74: Raw images and respective velocity field for the very first instants of the dam-break flow. (a) $T = -0.1$, (b) $T = 0$, (c) $T = 0.28$, (d) $T = 0.41$, (e) $T = 0.55$, (f) $T = 0.63$. (Experiment h32.5-hsP10-D3.85-BW-run02).

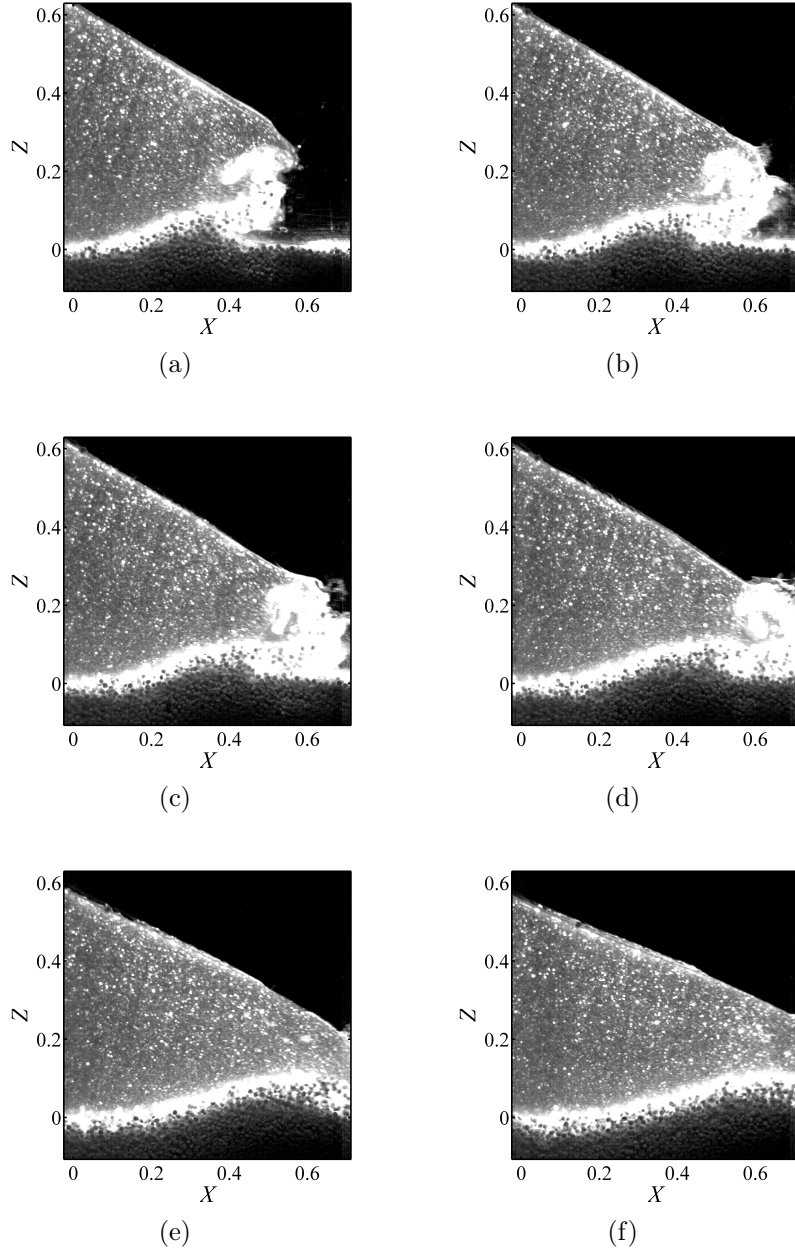


Figure 3.75: Raw images and respective velocity field for the very first instants of the dam-break flow. (a) $T = 0.69$, (b) $T = 0.74$, (c) $T = 0.77$, (d) $T = 0.82$, (e) $T = 0.96$ and (f) $T = 1.10$. (Experiment h32.5-hsP10-D3.85-BW-run04). (Continuation of Figure 3.74)

the upward jet the coalescence of upper and inferior water layers occurs in the interval $0.69 < T < 0.77$. While in the sand bed case one can see clearly that the near bed water layer ($Z < 0.10$) moves along the horizontal direction (Figure 3.35), in the pellets case an ascending movement of the water induced by the bed deformation is observed (Figure 3.76 and 3.77).

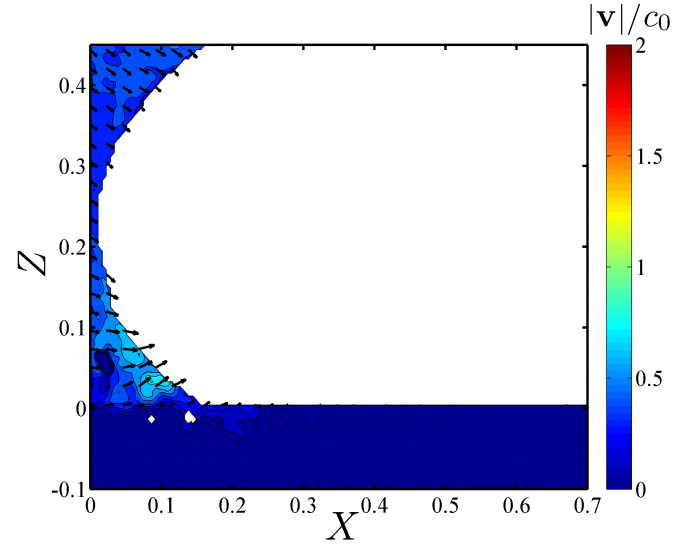
Figures 3.76 to 3.80 depict the velocity field of the dam-break flow over a pellets sediment bed from $T = 0$ to $T = 1.10$. The data was processed by means of PIV and for each depicted velocity field, 7 images were used resulting in an integration time of $\Delta T = 0.033$. The PIV processing used a minimum interrogation window of $16 \text{ pixels} \times 16 \text{ pixels}$ resulting in a spatial resolution of 1.8 mm, enough for describing the early stages of the dam-break flow.

After the dam-break wave collapse the water surface evolves more smoothly as it can be seen in Figures 3.75 (e) and (f) corresponding to $T = 0.96$ and $T = 1.10$ respectively. It is here investigated the possibility of obtaining flow average values considering a larger set of flow images. Instead of only 7 images, a set of 51 images is considered for the analysis to be performed for $T > 1.10$. The time $T_i = 1.5$ was chosen as test case, and the corresponding integration time was $\Delta T = 0.28$ since 51 images were considered. This means that the mean velocity determination is made in the time interval:

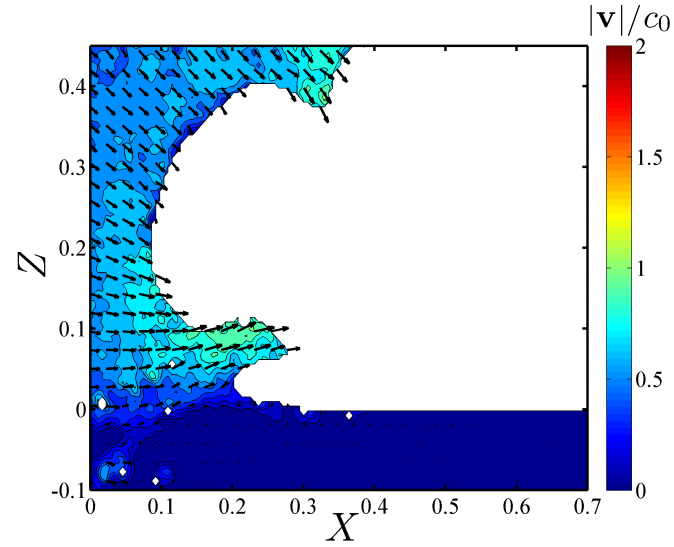
$$T = T_i \pm \frac{\Delta T}{2} = 1.5 \pm 0.14 \quad (3.6)$$

Figure 3.81 depicts the flow images at the extremities of the time interval defined in equation (3.6).

It can be seen that there is a variation both in the water surface and bed geometry. It is also possible to observe in the images that the luminosity is not always uniform and some vertical patterns appear in the images. This problem was addressed during the PIV

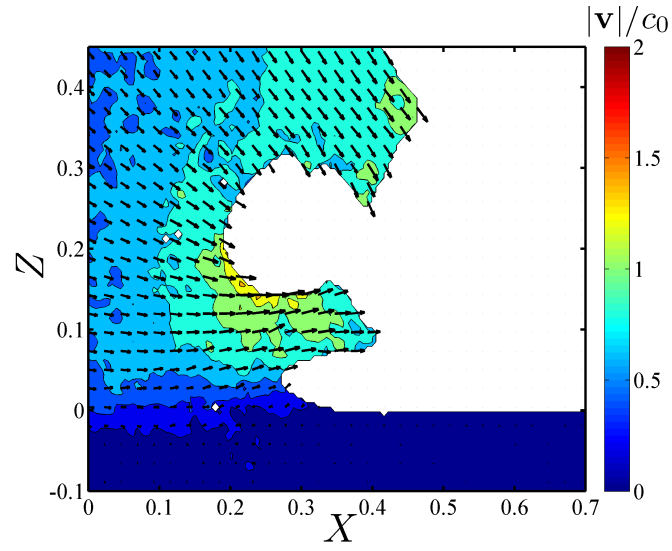


(a)

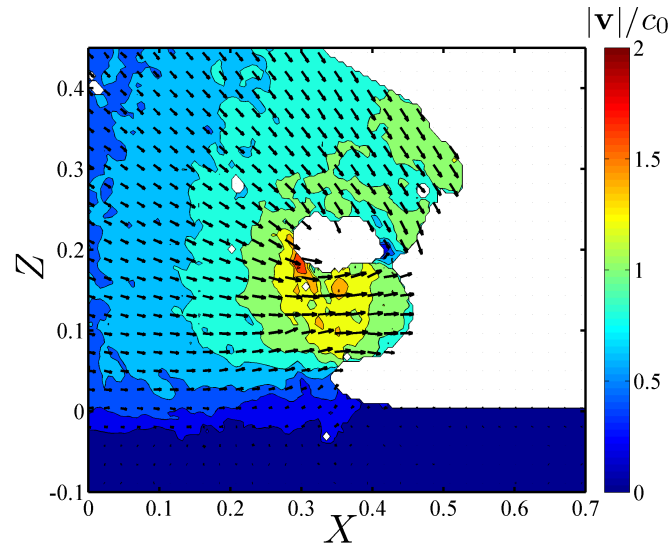


(b)

Figure 3.76: Velocity field for the very first instants of the dam-break flow. (a) $T = -0.1$, (b) $T = 0$. Only 1 out of 4 vectors depicted.

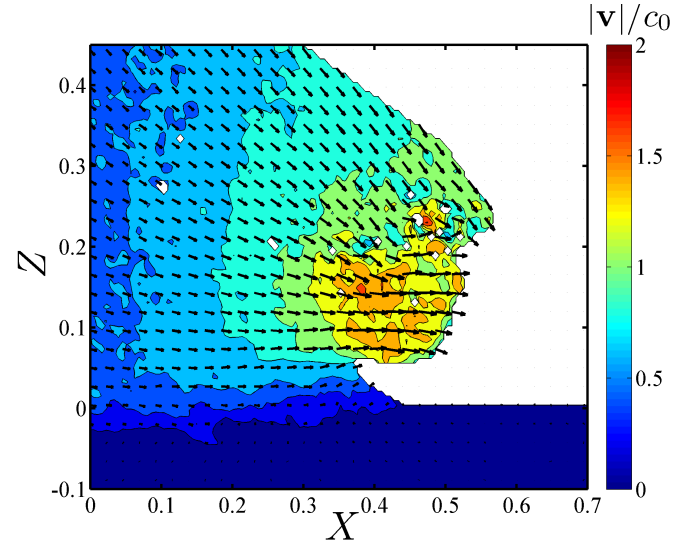


(a)

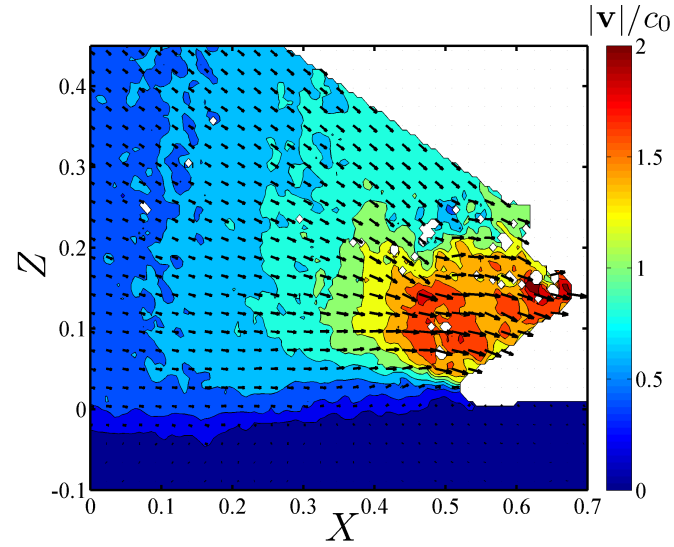


(b)

Figure 3.77: Velocity field for the very first instants of the dam-break flow. (a) $T = 0.28$, (b) $T = 0.41$. Only 1 out of 4 vectors depicted.

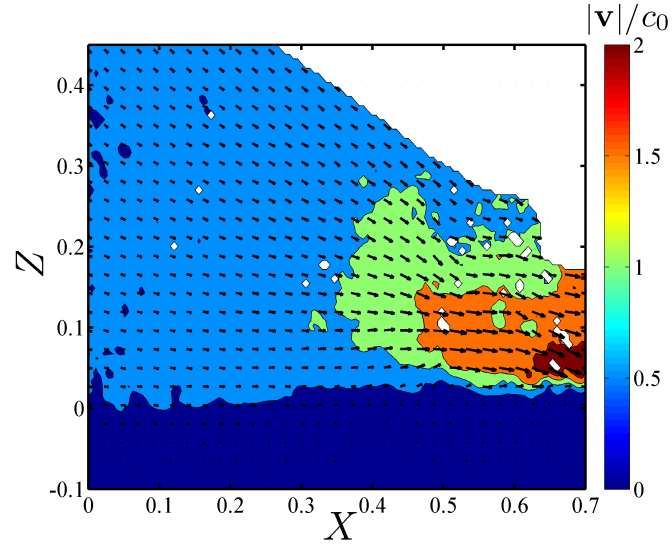


(a)

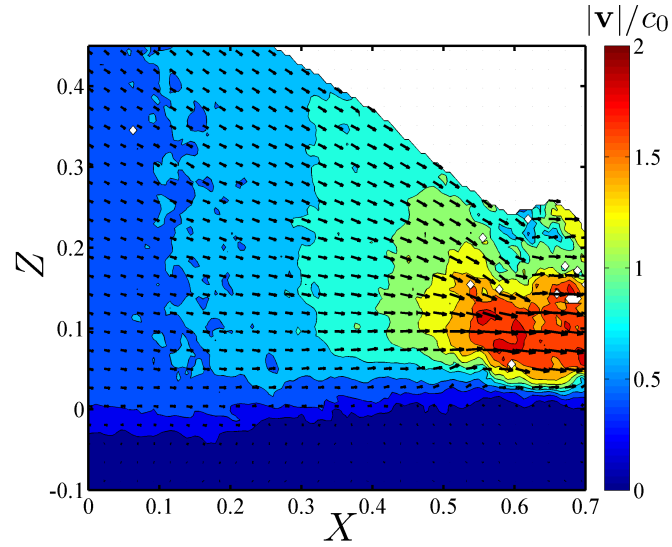


(b)

Figure 3.78: Velocity field for the very first instants of the dam-break flow. (a) $T = 0.55$, (b) $T = 0.63$. Only 1 out of 4 vectors depicted.

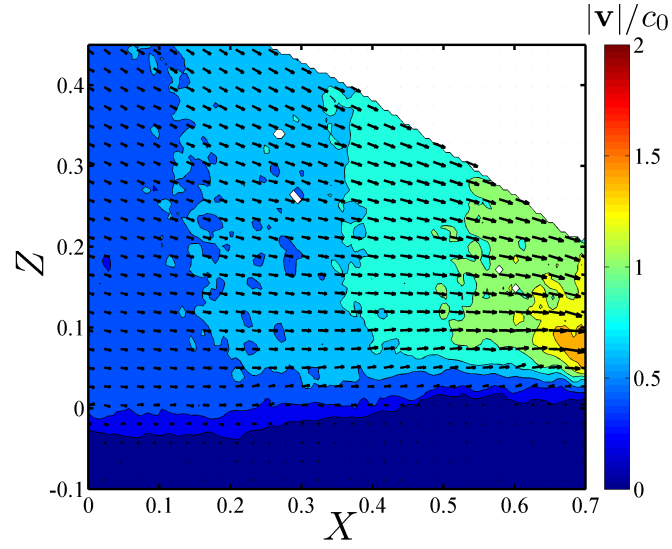


(a)

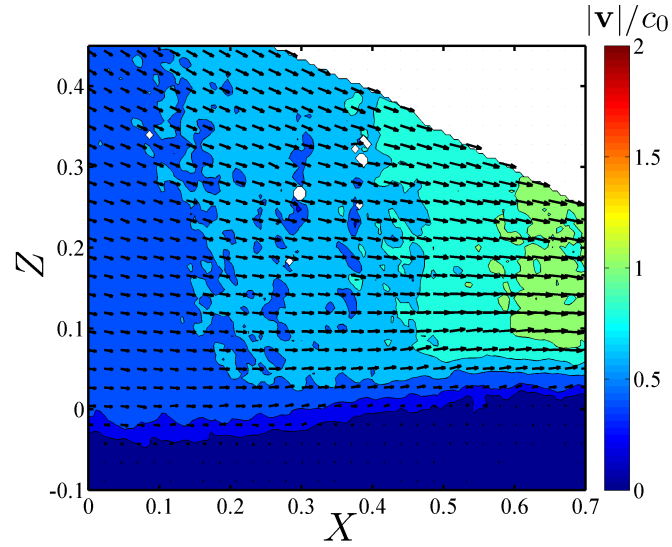


(b)

Figure 3.79: Velocity field for the very first instants of the dam-break flow. (a) $T = 0.69$, (b) $T = 0.74$. Only 1 out of 4 vectors depicted.



(a)



(b)

Figure 3.80: Velocity field for the very first instants of the dam-break flow. (a) $T = 0.96$ and (b) $T = 1.10$. Only 1 out of 4 vectors depicted.

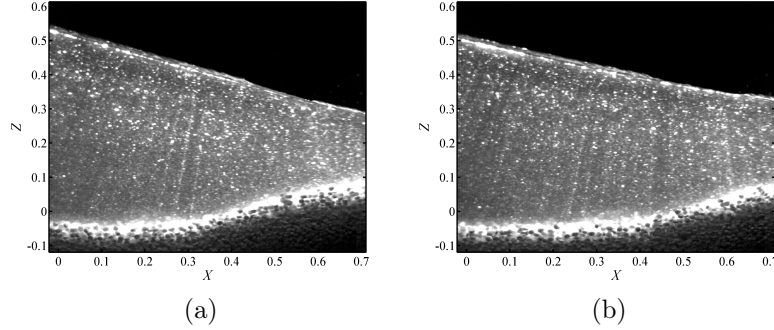


Figure 3.81: Raw images of the flow corresponding to (a) $T=1.5-0.14$ and (b) $T=1.5+0.14$.

processing by means of a routine to uniformise the image's background. The results obtained for the mean flow are depicted in Figures 3.82 and 3.83. The non-dimensional standard-deviations of velocity, $\sigma_U = \sigma_u/c_0$ and $\sigma_W = \sigma_w/c_0$ are depicted in Figures 3.84 and 3.85.

Profiles of U , W , σ_U and σ_W between $X = 0$ and $X = 0.5$ are depicted in Figures 3.86 to 3.89. Similar features to the ones observed in the sand bed case are observed. In the water layer the horizontal component is practically constant, whereas in the sediment layer the velocity profile appears to be linear. The vertical component is essentially negative, except in the bed region for $X \geq 0.3$. Also, the horizontal component is essentially positive. The visible bed form is a reminiscence of the flow-bed interaction during the initial stages; it induces a vertical upward motion and is thus responsible for the positive values of W . Standard deviations profiles plotted in Figures 3.88 and 3.89 show that for $X < 0.5$ $\sigma_U \approx \sigma_W$. Otherwise, $\sigma_U > \sigma_W$.

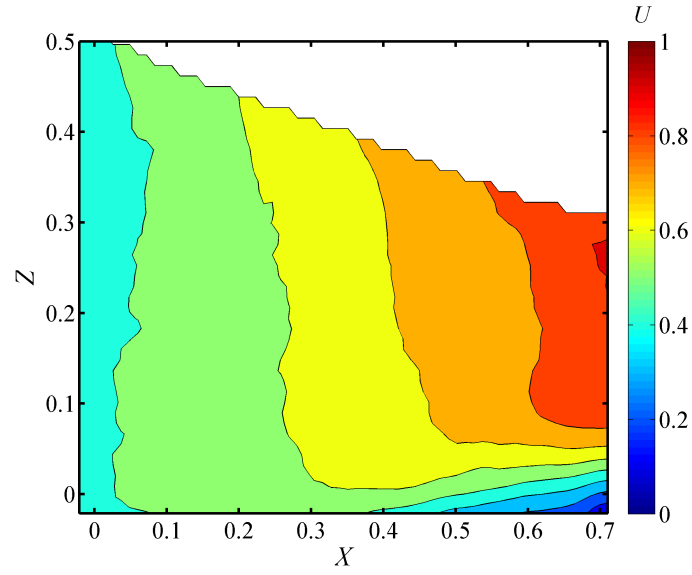


Figure 3.82: Mean values for the horizontal component of velocity, U , for $T = 1.5 \pm 0.14$.

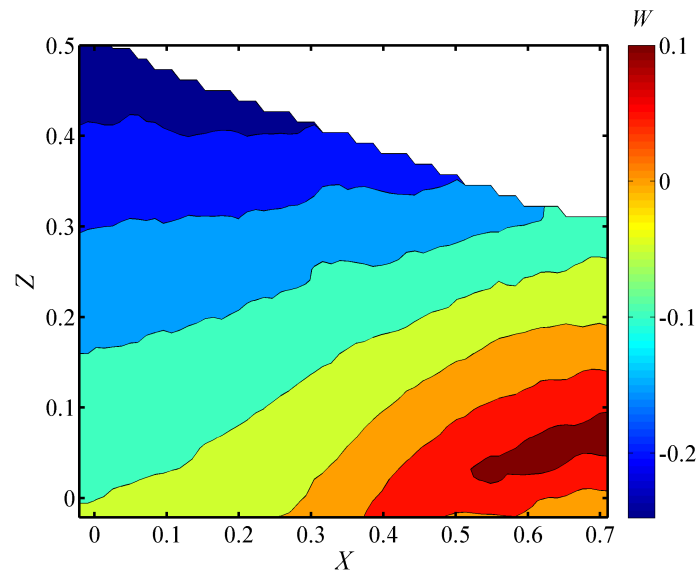


Figure 3.83: Mean values of the vertical velocity component, W , for $T = 1.5 \pm 0.14$.

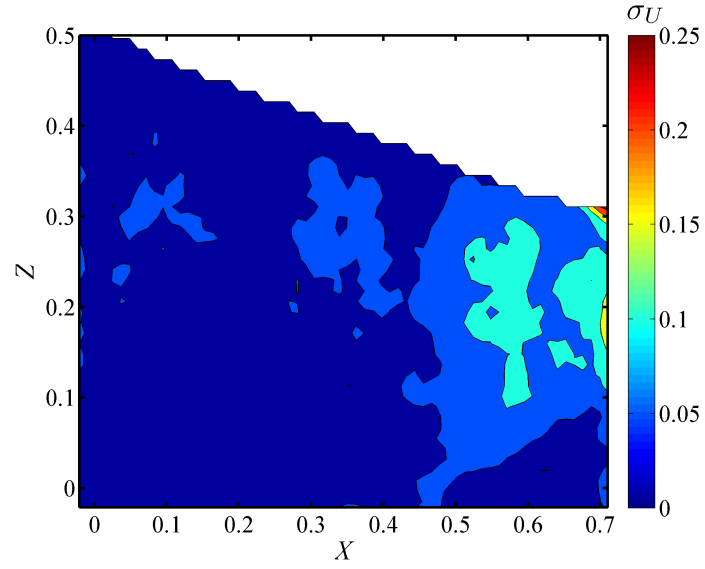


Figure 3.84: Standard deviation of the horizontal velocity component, $\sigma_U = \sigma_u/c_0$, for $T = 1.5 \pm 0.14$ (number of samples 50).

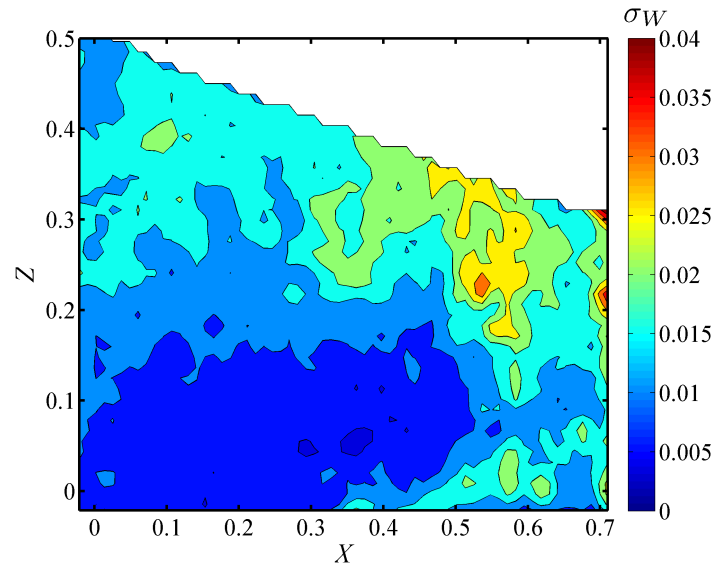


Figure 3.85: Standard deviation of the vertical velocity component, $\sigma_W = \sigma_w/c_0$, for $T = 1.5 \pm 0.14$ (number of samples 50).

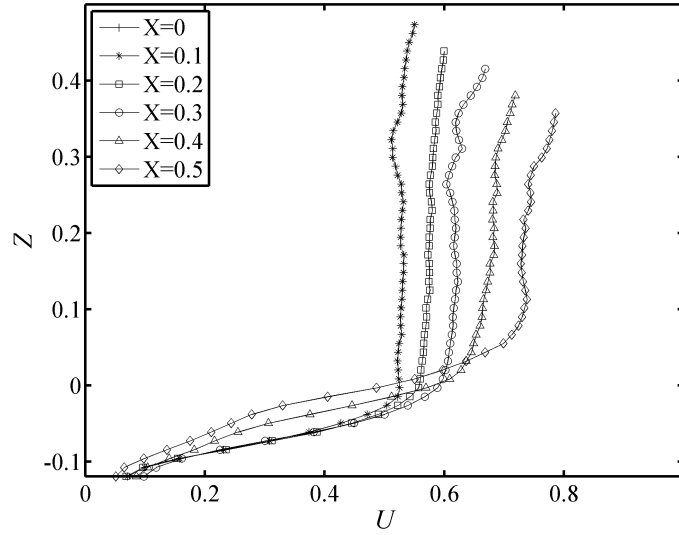


Figure 3.86: Horizontal velocity profiles measured between $X = 0$ and $X = 0.5$.

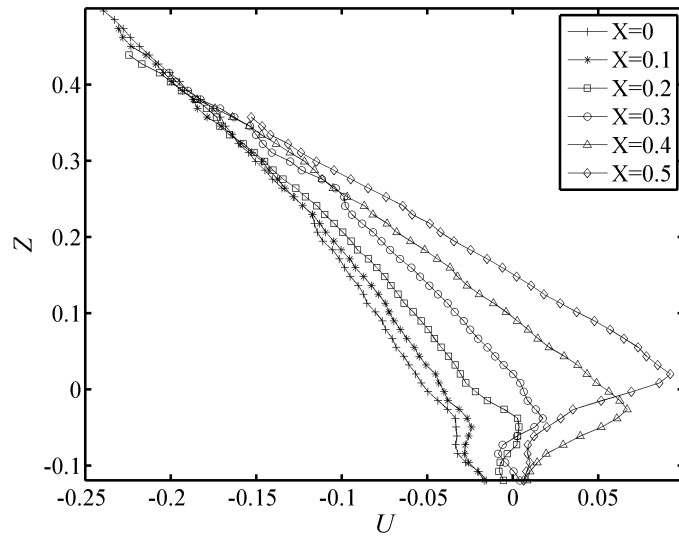


Figure 3.87: Vertical velocity profiles measured between $X = 0$ and $X = 0.5$.

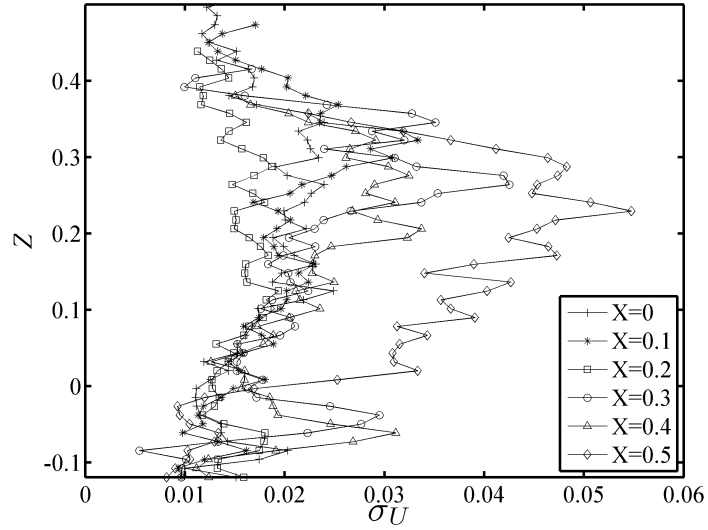


Figure 3.88: Standard deviation of the horizontal velocity measured between $X = 0$ and $X = 0.5$ (number of samples 50).

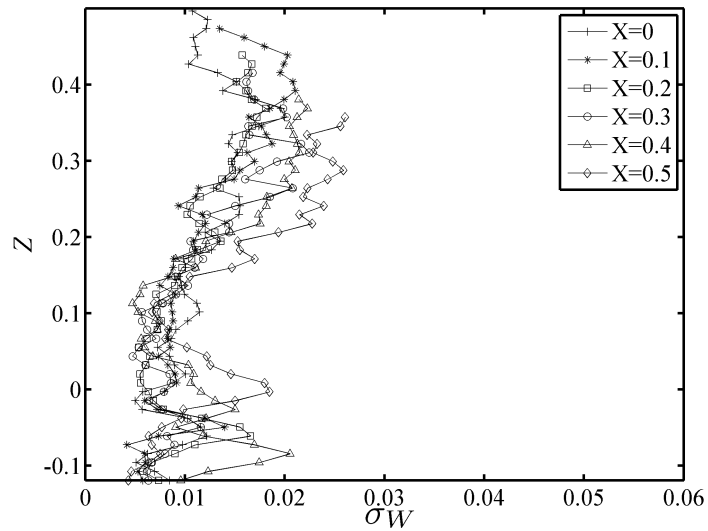


Figure 3.89: Standard deviation of the vertical velocity measured between $X = 0$ and $X = 0.5$ (number of samples 50).

3.3 Analysis of the Scale Effects

In physical modelling it becomes important to analyse the scale effects and to relate the experiments made at small scale in the laboratory, like the ones presented in section 2.5, with the reality. Classical works, like the ones by Sédov (1977) and Yalin (1971) present the foundations for physical modelling. A recent publication from ASCE (American Society of Civil Engineers) presents an extended approach to physical modelling considering problems involving loose-boundaries, air-water flows, debris and others (Etema et al., 2000).

For the specific case of the dam-break flow Lauber and Hager (1998b) have found that, for the hydrodynamic dam-break flow in a prismatic rectangular channel, the Froude similitude holds and scale effects can be neglected provided that:

$$h_0 \geq 0.30 \text{ m} \quad (3.7)$$

Soares-Frazão (2007) presented a comparison of the experiments for the dam-break over a triangular sill using two initial conditions $h_0 = 0.111 \text{ m}$ and $h_0 = 0.74 \text{ m}$. Soares-Frazão (2007) verified that by using the Froude similitude scaling variables, the results for the water level evolution from both experiments collapsed in the same curve, as depicted in Figure 3.90. It is thus possible to assume that the experiments presented in this chapter can be related to larger scale experiments by Froude scaling.

For the hydrodynamic case two different water heights were considered, $h_0 = 0.325 \text{ m}$ and $h_0 = 0.40 \text{ m}$, both obeying equation (3.7). In Figures 3.91 to 3.94 the comparison of the dam-break flow water height at different time instants using Froude similitude to scale the physical variables. It is possible to see that, for $T < 1.10$ there are significant differences between the free surface of both cases, but for $T > 1.10$, as the flow approaches the shallow-

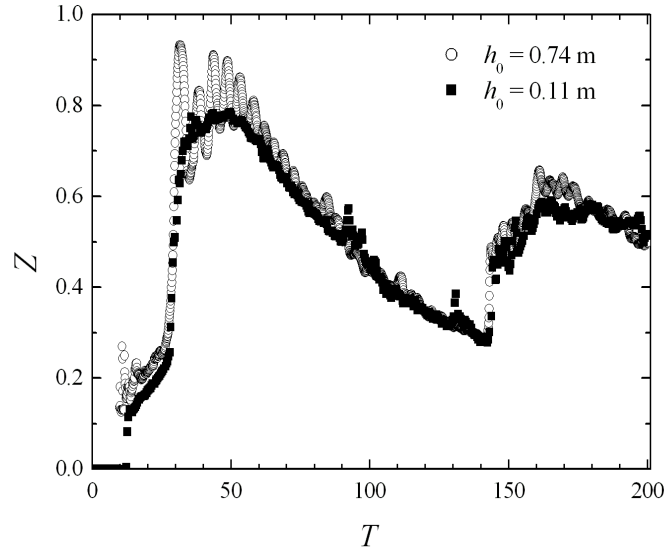


Figure 3.90: Experimental results for a dam-break flow over a bump obtained considering two initial water heights $h_0 = 0.11$ m and $h_0 = 0.74$ m, published in Soares-Frazão (2007).

water regime, the differences become more negligible, as the case for $T = 2.5$ which is the determined limit for which the shallow water hypothesis starts to be valid.

As seen in section 3.1.3 for the two different initial reservoir heads: $h_0 = 0.325$ m and $h_0 = 0.40$ m conditions, the obtained velocity results compared quite well between them and with the Ritter (1892) solution.

For the sedimentologic dam-break flow an analysis is here presented. Different profiles from the different runs, chosen for the same coordinate $X^* = x/(c_0 t)$, were plotted together to verify if the obtained profiles were self-similar. The results are plotted in Figures 3.95 to 3.98. It is possible to see that near the gate the profiles can be assumed to be self-similar, whereas, with increasing X^* , there is a deviation from one of the runs, which is the one cor-

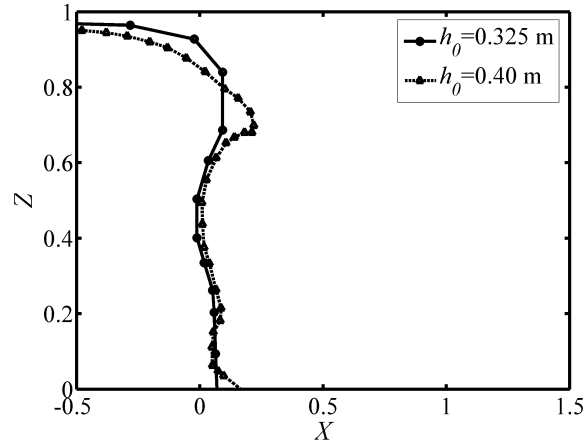


Figure 3.91: Water surface evolution for the hydrodynamic case, for two different initial conditions $h_0 = 0.325$ m and $h_0 = 0.40$ m at $T = 0$.

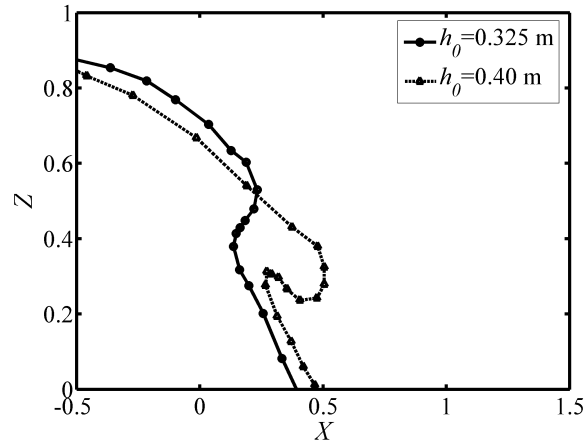


Figure 3.92: Water surface evolution for the hydrodynamic case, for two different initial conditions $h_0 = 0.325$ m and $h_0 = 0.40$ m at $T = 0.55$.

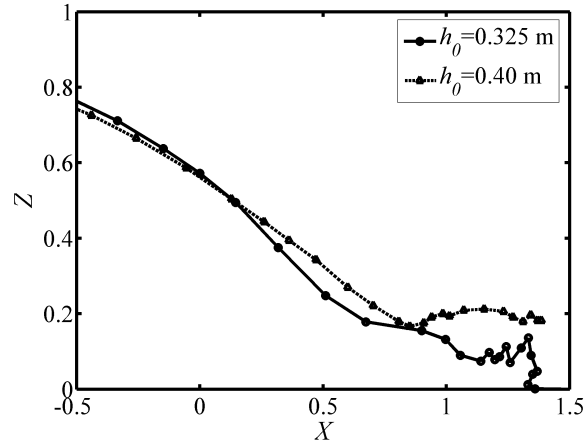


Figure 3.93: Water surface evolution for the hydrodynamic case, for two different initial conditions $h_0 = 0.325$ m and $h_0 = 0.40$ m at $T = 1.10$.

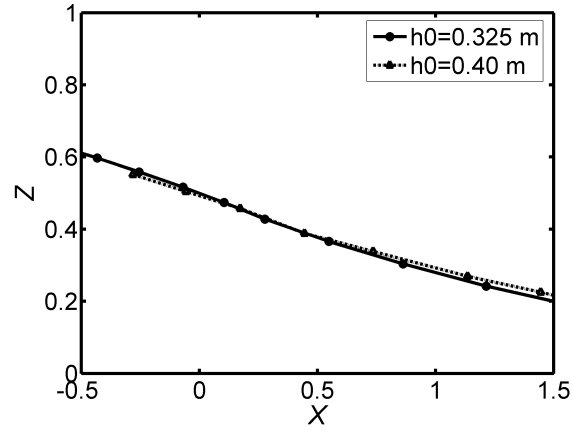


Figure 3.94: Water surface evolution for the hydrodynamic case, for two different initial conditions $h_0 = 0.325$ m and $h_0 = 0.40$ m at $T = 2.5$.

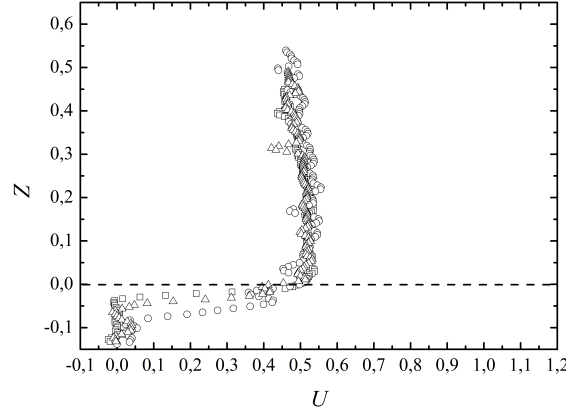


Figure 3.95: Velocity profiles of U at $X^* = 0$ considering the sedimentologic case with sand and $h_0 = 0.325$ m. Three different runs considered.

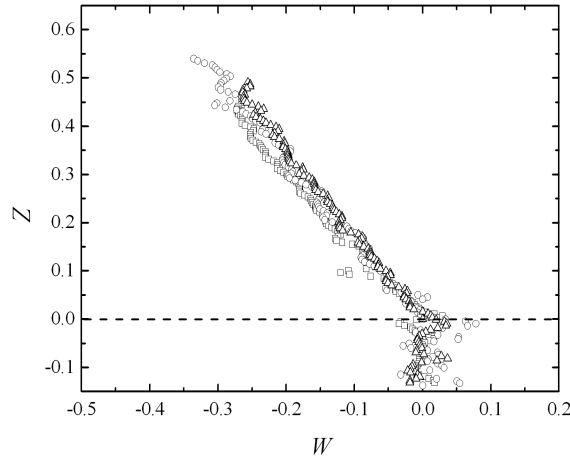


Figure 3.96: Velocity profiles of W at $X^* = 0$ considering the sedimentologic case with sand and $h_0 = 0.325$ m. Three different runs considered.

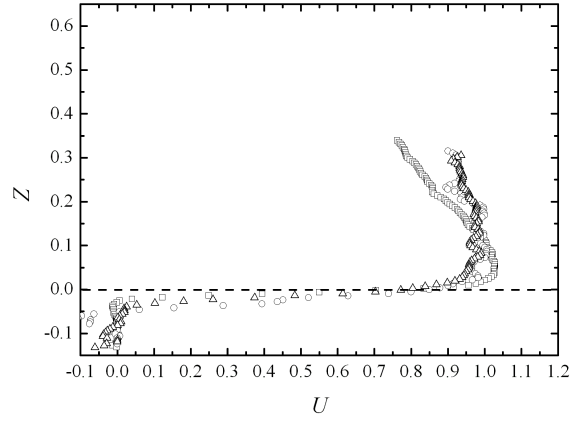


Figure 3.97: Velocity profiles of U at $X^* = 0.5$ considering the sedimentologic case with sand and $h_0 = 0.325$ m. Three different runs considered.

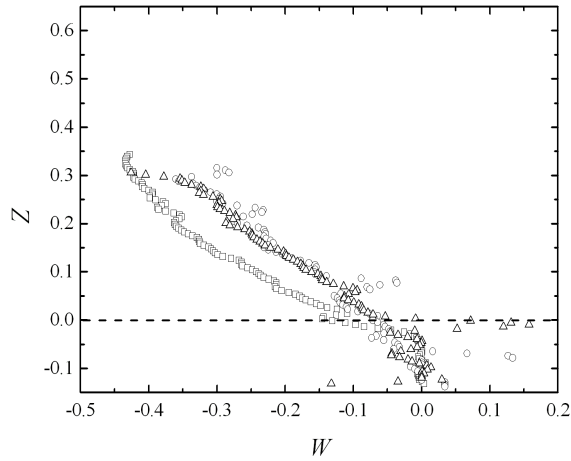


Figure 3.98: Velocity profiles of W at $X^* = 0.5$ considering the sedimentologic case with sand and $h_0 = 0.325$ m. Three different runs considered.

responding to $X^* = 0.5$, that is, $T = 0.81$ and $X = 0.405$, where, as it can be seen in Figure 3.37, there is still the effect of the dam-break wave breaking.

3.4 The Measurement Techniques Compared

This section presents the comparison between the PIV and PTV approach. As referred the PTV algorithm used was developed for granular flows (Capart et al., 2002), and its application to an hydrodynamic dam-break flow has to be verified. The comparison between the two techniques was made for $T = 5$ using the flow images of Figure 2.25. To do so the Lavision Davis 7.2 PIV software is used. The PIV processing applied to the present dam-break flow images used an iterative deformation window algorithm. The initial size of the interrogation windows was 256×256 pixels and the final size was 32×32 pixels. An overlap of 50% between the interrogation windows was imposed, giving roughly the same area per window as the area per bin used. This procedure was applied to the set of images corresponding to the interval $[T - 5\Delta T, T + 5\Delta T] = [4.73, 5.27]$. The corresponding mean velocity field is depicted in Figure 3.99. In Figure 3.100 the signal to noise ratio colourmap of the PIV measurement, as defined in equation (2.15), section 2.2.2, is presented. As it was then referred (subsection 2.2.2), and following the PIV manual (Lavision, 2007) values above 1.3 are considered a good signal-to-noise ratio, and as it can be seen by the results depicted in Figure 3.100 the obtained PIV results have a high signal-to-noise ratio.

In order to compare the two methods, velocity profiles at three different locations were plotted and are presented in Figures 3.101 and 3.102. As it can be seen they agree quite well for the region

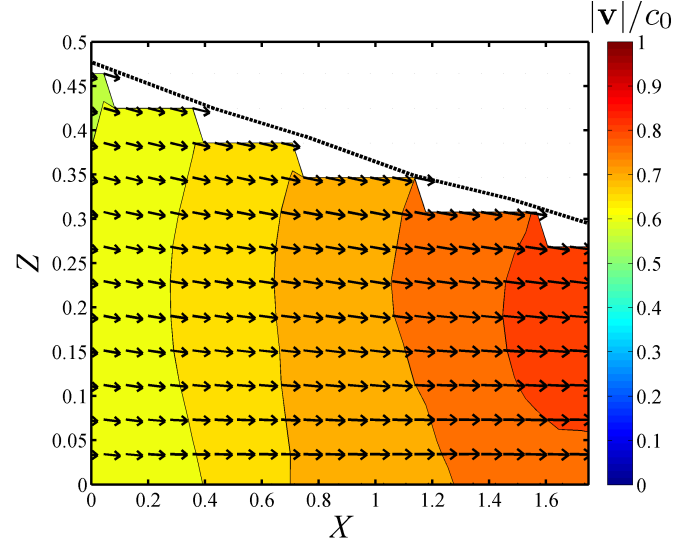


Figure 3.99: Mean velocity field obtained with PIV for $T = 5$.

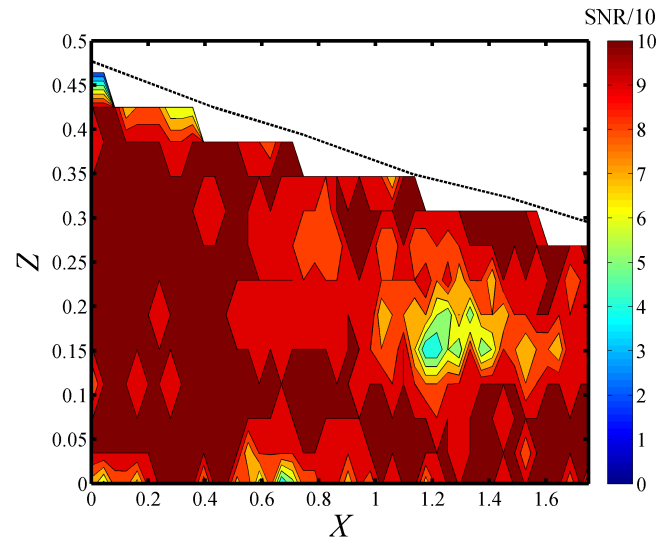


Figure 3.100: Average signal-to-noise ratio for the PIV data processing at $T = 5$.

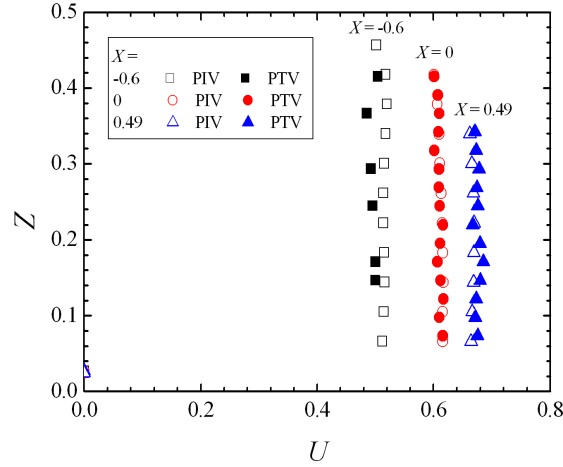


Figure 3.101: Profiles of the horizontal component, U , (obtained with the PTV method and their comparison with PIV at different sections of the flow.

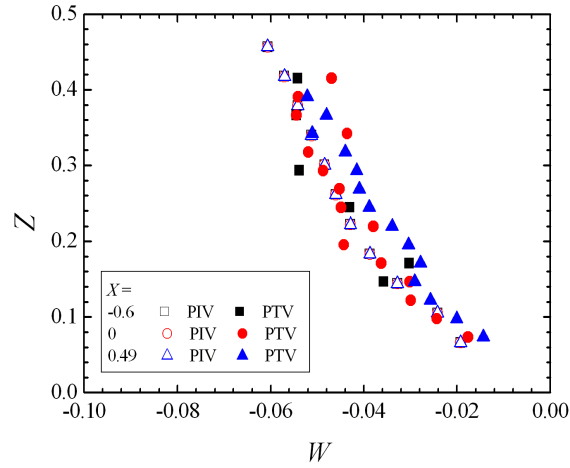


Figure 3.102: Profiles of the vertical component, W obtained with the PTV method and their comparison with PIV at different sections of the flow.

upstream of the gate, which is the one with more seeding. At the gate section, the PIV gave unrealistic results probably due to the poor seeding.

The presented results have shown that PIV and PTV hold the same results for different conditions. The limitations of the PIV and PTV for low density and high density seeding concentration are well known. In the literature combined approaches between PIV and PTV can be found (e.g. Takehara et al. (2000); Stitou and Riethmuller (2001); Bastiaans et al. (2002)) where the goal is to extend the resolution of PIV by means of the PTV approach. The velocity field determined by PIV can be used as input into the PTV in order to be used as a predictor to determine possible particle's displacements in scarce seeding regions, where the determination of velocity would be made by PTV.

Chapter 4

Statistical Analysis of the Dam-Break Flow

*Two roads diverged in a wood, and I -
I took the one less travelled by*
Frost (1921)

As it was seen in Chapters 1 and 3 the dam-break flow is an example of a severe transient flow, i.e., a flow that exhibits a strong time dependence. In this case it becomes difficult, if not impossible, to use the common methods used for stationary flow. This chapter presents a statistical analysis of the dam-break flow. It starts first by presenting the statistical tools used for a stationary flow case and proceeds to the application of the ensemble statistics to this type of flows. The ensemble average concept is compared with a moving time average. Finally, with the averaged velocity fields an analysis of the boundary-layer in the dam-break flow is presented.

4.1 The Stationary Flow Case

In a stationary open channel flow, i.e., a flow where the macro-scale variables like the water height and flow rate have a time scale T_M

much larger than the time scale of the fluctuations, T_f :

$$T_M \gg T_f \quad (4.1)$$

where the fluctuations can be observed when accurately measuring a generic variable ϕ with enough time resolution, Δt_a . To comply with the Shannon theorem $\Delta t_a > T_f/2$. Using the Reynolds decomposition the flow variables can be written using the Reynolds averaging (Reynolds, 1895) as:

$$\phi = \bar{\phi} + \phi' \quad (4.2)$$

where ϕ is the instantaneous value of a generic flow variable, (e.g. u , w , etc), $\bar{\phi}$ is the time-averaged value of the generic variable and ϕ' it's the fluctuation. Figure 4.1 (a) illustrates the Reynolds decomposition stated by equation (4.2) for the horizontal component u . The first order moment, the mean, is, for a stationary flow given by:

$$\bar{\phi} = \frac{1}{t_2 - t_1} \int_{t_1}^{t_2} \phi dt \quad (4.3)$$

For a stationary flow the interval $(t_2 - t_1)$ can be taken as large as possible, that is $(t_2 - t_1) \gg T_f$. For the discrete set of N values, one has:

$$\bar{\phi} = \frac{1}{N} \sum_{i=1}^N \phi_i \quad (4.4)$$

In Figure 4.1 (b) a case where the relation (4.1) is no longer valid. In this case equation (4.3) can be used, provided that

$$T_f < (t_2 - t_1) \ll T_M \quad (4.5)$$

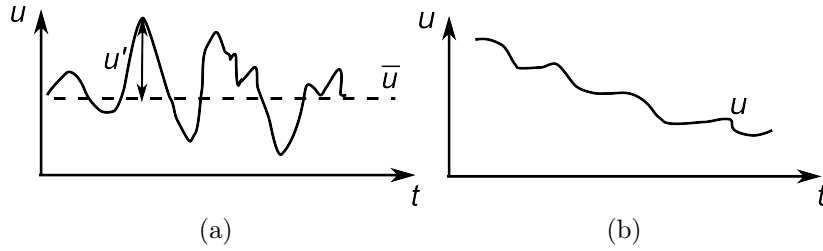


Figure 4.1: (a) The Reynolds decomposition for a stationary case is here illustrated. The mean value is time independent provided that the time scale of the macro variables (e.g flow rate) are much larger than the fluctuation time scales. (b) Depicts a case where the transient character of the flow is important, that is, where equation (4.1) is no longer valid.

The second order moment, the variance of ϕ , σ_ϕ^2 , can be expressed as:

$$\sigma_\phi^2 = \frac{1}{t_2 - t_1} \int_{t_1}^{t_2} (\phi - \bar{\phi})^2 dt \quad (4.6)$$

For the discrete case one has:

$$\sigma_\phi^2 = \frac{1}{N-1} \sum_{i=1}^N (\phi_i - \bar{\phi})^2 \quad (4.7)$$

The standard deviation is the square root of the variance, therefore, for the discrete case:

$$\sigma_\phi = \sqrt{\frac{1}{N-1} \sum_{i=1}^N (\phi_i - \bar{\phi})^2} \quad (4.8)$$

It is also possible to define the covariance as

$$\overline{u'w'} = \frac{1}{N} \sum_{i=1}^N (u_i - \bar{u}) (w_i - \bar{w}) \quad (4.9)$$

This equation is linked to the Reynolds stress $\tau_R = -\rho \overline{u'w'}$ which is an important parameter in turbulent flows. It can be used to determine, for instance, the shear stress at the bottom (Tennekes and Lumley, 1972).

Equations (4.3) to (4.9) are valid for stationary flows. In this case the interval $t_2 - t_1$, or the discrete number of samples, N , can be made as large as possible. The question concerning how many samples are needed in order to obtain a meaningful average value has been raised before. For purposes of obtaining an average of a generic physical variable Bell (2001) suggests as a rule of thumb a minimum number of samples between 4 and 10 values. For the particular case of velocity measurements in turbulent flows, Yanta and Smith (1973) made an investigation about the sample lengths required for turbulence measurements. According to those authors the sample length was found to be a function of turbulence intensity. Following the study of Yanta and Smith (1973), Hitching and Lewis (1999) proposed the following expression for the sample size, N :

$$N = 4 \left(\frac{\mu_u}{\mu_u - \alpha_u} \right)^2 \frac{\sqrt{u'^2}}{\bar{u}} \quad (4.10)$$

where μ_u is the mean of the population, α_u is the mean of the sample, $\sqrt{u'^2}/\bar{u}$ is the turbulence intensity, and according to Hitching and Lewis (1999) the constant in the right hand side of is a function of the chosen confidence level (95% for the case of equation (4.10)). Bates and Hughes (1976, 1977) for turbulent pipe in a flow proposed that $N \geq 3500$ samples.

The number of samples is also important to ensure a small sta-

tistical error. Following Lumley (1979), Durst et al. (1996) used the following expression:

$$\frac{\varepsilon_\phi}{\bar{\phi}} = \sqrt{\frac{2}{N}} \frac{\sigma_\phi}{\bar{\phi}} \quad (4.11)$$

where ε_ϕ is the error associated to the standard derivation of ϕ and N the number of independent samples. Durst et al. (1996) assumed $\varepsilon_\phi/\bar{\phi} = 0.3\%$, and $\sigma_\phi/\bar{\phi} = 40\%$ for which equation (4.11) holds $N > 3.5 \times 10^4$.

For the case of the Reynolds stresses measurements Hitching and Lewis (1999), following Yanta and Smith (1973) and Bates and Hughes (1976) proposed a value $N \geq 3000$.

With these values of N and considering an acquisition frequency of 1 kHz, it would be needed a) 3.5 seconds to obtain 3500 data points as proposed by Bates and Hughes (1976) or b) 35 seconds to obtain the required data as proposed by Durst et al. (1996). This later is about 2.3 times the duration of the tested dam-break flows. As a consequence, in order to obtain such large value of samples a larger acquisition frequency is needed. And this value is often limited by the measurement equipment characteristics.

4.2 The Transient Flow Case: The Dam-Break

. In the case of the transient flow equations one falls in the situation depicted in Figure 4.1 (b) where the mean value is no longer time independent and therefore in 4.3 to 4.7 care must be taken in choosing the time interval $t_2 - t_1$. A small interval will not hold enough data for meaningful statistics and a large interval will smooth out the transient effect of the flow.

One way to deal with the transient character of the flow is by introducing the concept of ensemble averaging (Kundu et al., 2011; Bendat and Piersol, 2011). This concept implies the repetition of the same experiment in the same conditions until a set of meaningful data is obtained. The statistics are then performed along the set of experiments as exemplified in Figure 4.2 for the horizontal component u . For a generic variable, ϕ one has:

$$\hat{\phi}(t_e) = \frac{1}{N} \sum_{j=1}^N \phi^j(t_e) \quad (4.12)$$

where $\hat{\phi}$ denotes the ensemble average of ϕ and $\phi^j(t_e)$ is the j -th record of the variable ϕ for a given time t_e (Figure 4.2), and N should be made as large as possible.

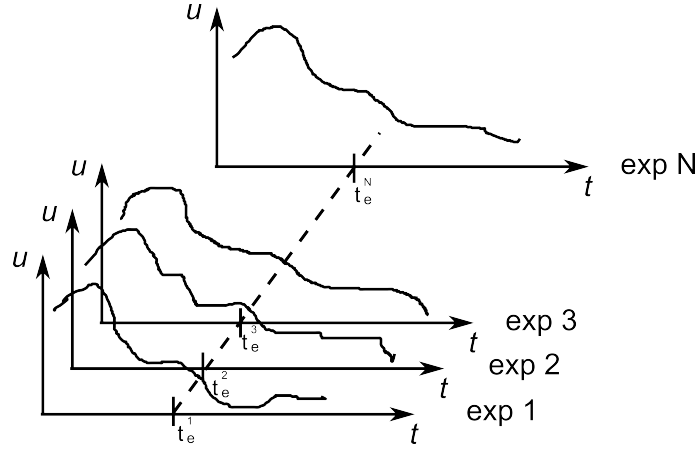


Figure 4.2: The concept of ensemble average is here illustrated. N experiments are made for the same initial conditions.

Obtaining an ensemble of similar experiments is not always possible due to the nature of the experiment, e.g., dam-break testing at full scale. However, if the acquisition time is small enough compared with the time scales of the phenomenon, it is possible to define an interval for which the computation of flow statistics is possible.

This is depicted in Figure 4.3 where the acquisition time scale, Δt_a is much smaller than the time duration of the phenomenon $\Delta t_c = (t_2 - t_1)$:

$$\Delta t_a \ll \Delta t_c \quad (4.13)$$

The average value of a variable ϕ would then be:

$$\tilde{\phi} = \frac{1}{t_2 - t_1} \int_{t_1}^{t_2} \phi dt \quad \text{and } t_2 - t_1 = \Delta t_c \gg \Delta t_a \quad (4.14)$$

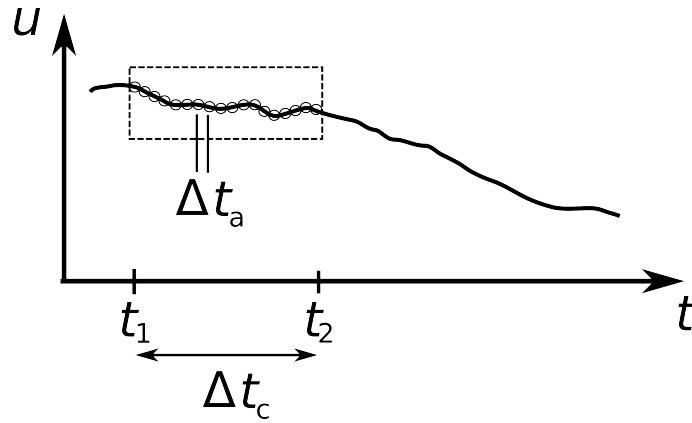


Figure 4.3: Illustration of a time averaging in a transient signal considering the relation given by equation (4.13).

The ensemble average method (equation 4.12) is here compared with the method of time averaging in a smaller interval (equation 4.14). Four points of the domain were considered for this analysis and are depicted in Figure 4.4, which also depicts the time evolution of the free surface at 4 different times. Their coordinates are presented in Table 4.1. The experiments used are referred by the code: h32.5-hs0-D0-BPP-run01 to h32.5-hs0-D0-BPP-run07 (See section 2.5 for details).

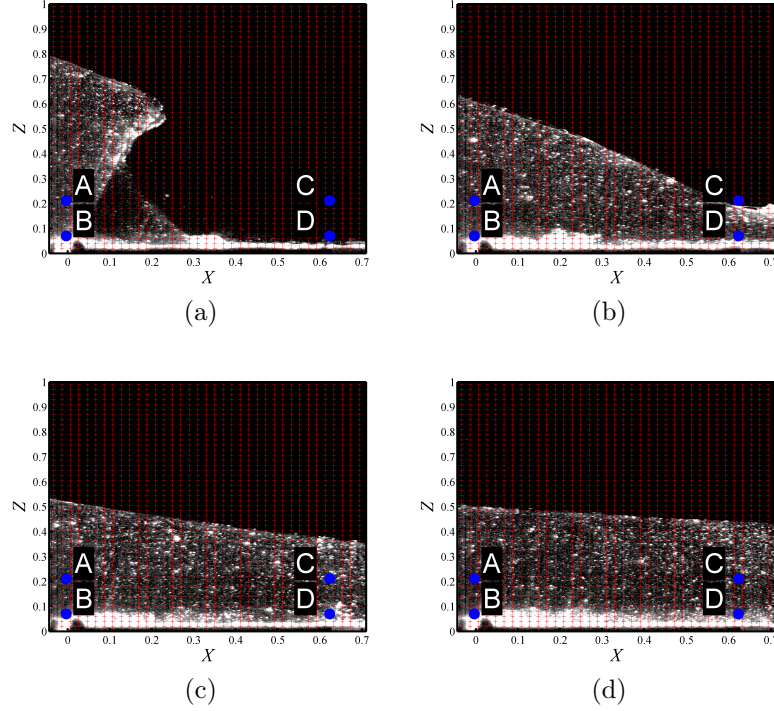


Figure 4.4: Location of the four chosen points for the statistical analysis and their location relative to the water surface in different time instants (a) $T = 0.5$, (b) $T = 1$, (c) $T = 2.5$ and (d) $T = 7.47$. The gate is located at $X = 0$ and the flow region was highlighted by means of enhancing the image contrast.

Table 4.1: Coordinates of the chosen points for analysis: dimensional and non-dimensional values.

Point	x (m)	z (m)	X	Z
A	0	0.071	0	0.22
B	0	0.021	0	0.062
C	0.200	0.071	0.62	0.22
D	0.200	0.021	0.62	0.062

The time series for the horizontal and vertical velocity components for one of the experiments are depicted in Figure 4.5 for Point A and D.

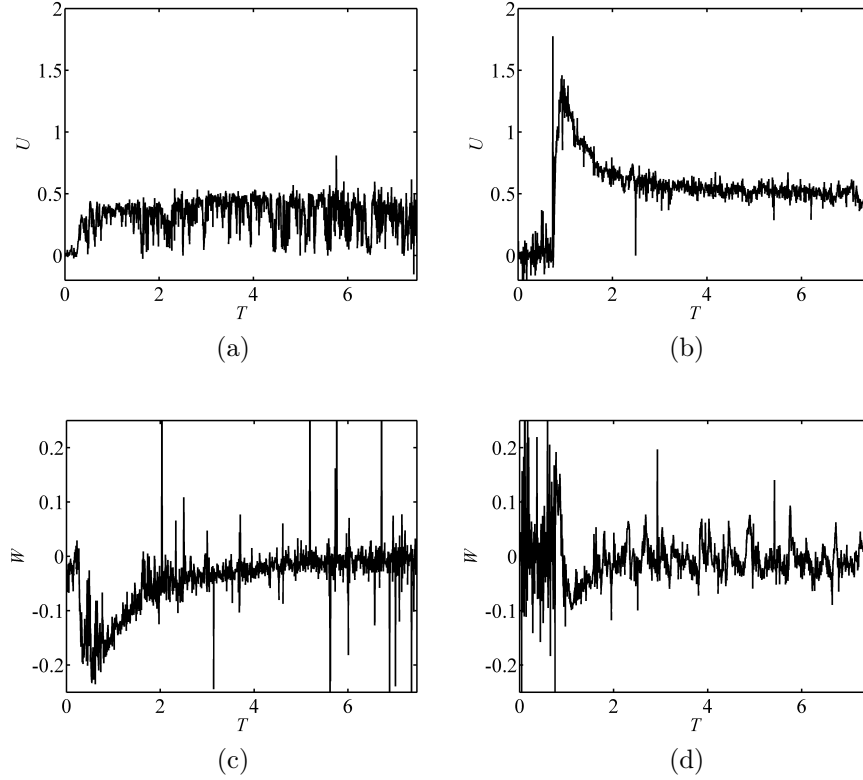


Figure 4.5: Time series of the velocity components: (a) $U = u/c_0$ at Point A. (b) $U = u/c_0$ at Point D. (c) $W = w/c_0$ at Point A. (d) $W = w/c_0$ at Point D.

Choosing point D as example, the corresponding time series of the horizontal component of velocity, $U = u/c_0$ is shown for four successful runs (of a total of 7) in Figures 4.6 (a) and (b) and 4.7 (a) and (b). Figure 4.8 depicts the ensemble average made with (a) 2 and (b) 3 runs whereas Figure 4.9 (a) and (b) depicts respectively the ensemble average obtained with 4 and 7 experiments. Since the fluctuations of each run are not correlated, averaging in the ensemble of the data set will naturally filter out the random noise. In Figure 4.10 the ensemble average of all measured experiments is compared with a moving time-average made in one of the runs

(experiment 1) using 10 and 20 data points as span, i.e., $\Delta t_c = (t_2 - t_1) = 0.01$ s and $\Delta t_c = (t_2 - t_1) = 0.02$ s respectively. For each case $\Delta t_c > \Delta t_a$. In Figure 4.10 it is possible to observe the moving average smoothing effect compared to the ensemble average of seven runs.

From Figure 4.5 to Figure 4.10 it is possible to observe the following:

- a:** There is some sharp noise peaks in the measurements since the PIV filtering was kept to a minimum.
- b:** In point D a sharp transient is observed due to the passage of the dam-break wave at $T = 0.69$.
- c:** After the dam-break wave crossing, (in case of point D, for $T > 2.5$), the time series of the velocity suggests that $T_M \gg T_f$ in the interval $[2.5, 7.47]$, and that flow statistics can be calculated by means of equation (4.4) and (4.7) using a much larger interval.

This remarks will be addressed in the next section with a detailed analysis of one of the runs.

4.2.1 Time Averaging of the Dam-break Flow Velocity Field

The experiment h32.5hs0D0BPPrun01 is here considered as an analysis example. Noise appears often in measurements, and the particular case is no exception, as can be seen by the spikes identified in Figure 4.11. Methods and algorithms to filter noise have been proposed and can be found in the literature (e.g. Goring and Nikora (2007); Glisson (1985)).

To filter the noise out of the data a method proposed by Tukey (1977) is here applied. This filter is hereafter referred as Tukey 53H,

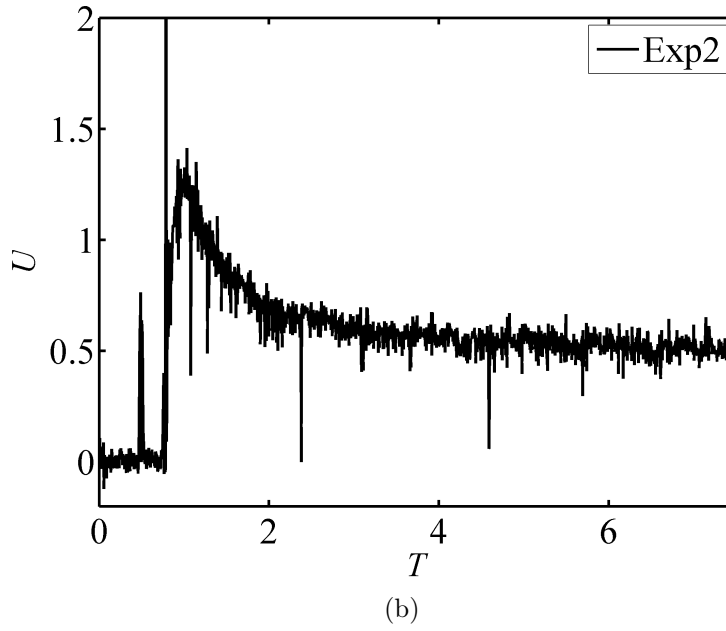
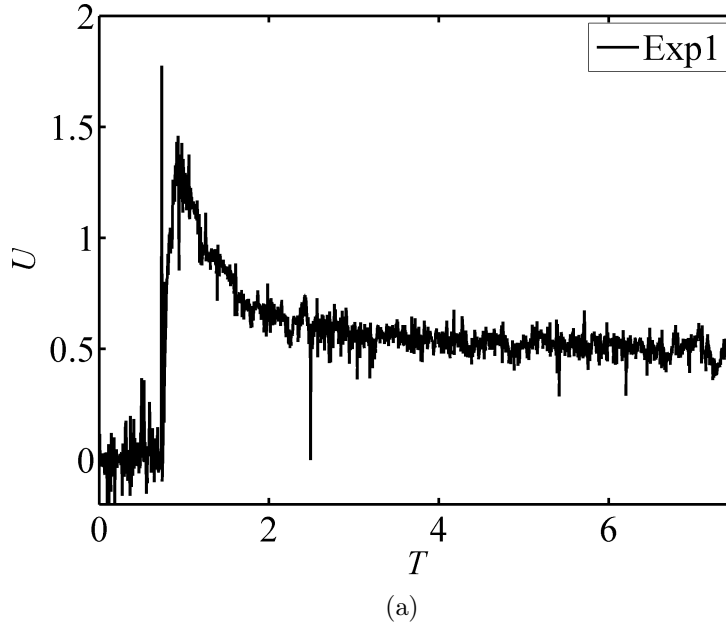


Figure 4.6: Time series for the horizontal velocity component U in point D for (a) experiment 1. (b) experiment 2. (Experiments: h32.5-hs0-D0-BPP-run01 and h32.5-hs0-D0-BPP-run02).

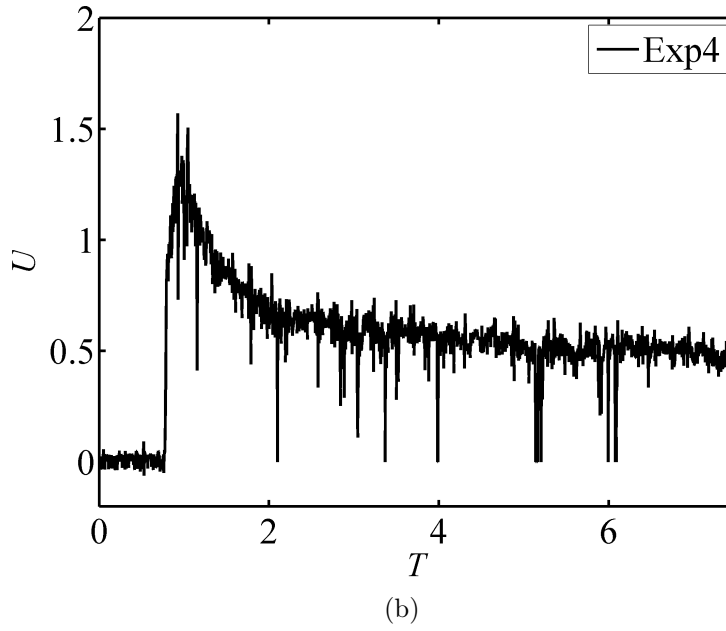
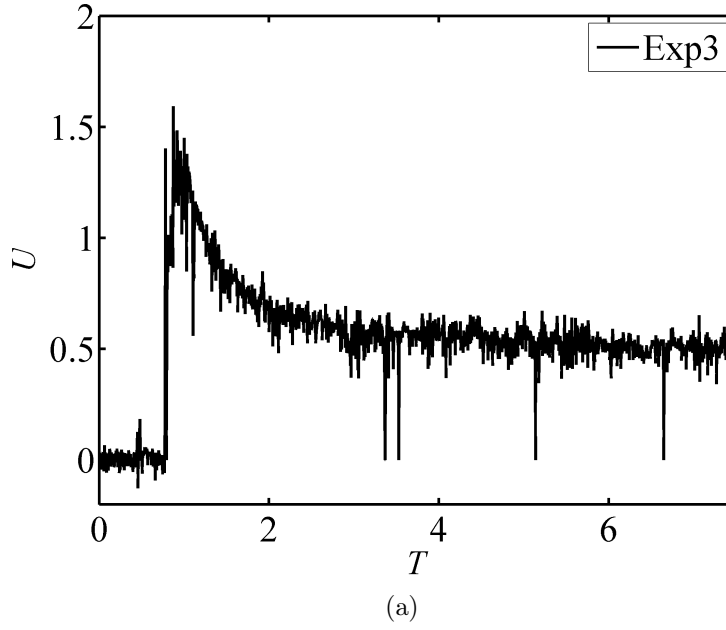


Figure 4.7: Time series for the horizontal velocity component U in point D for (a) experiment 3. (b) experiment 4. (Experiments: h32.5-hs0-D0-BPP-run03 and h32.5-hs0-D0-BPP-run04).

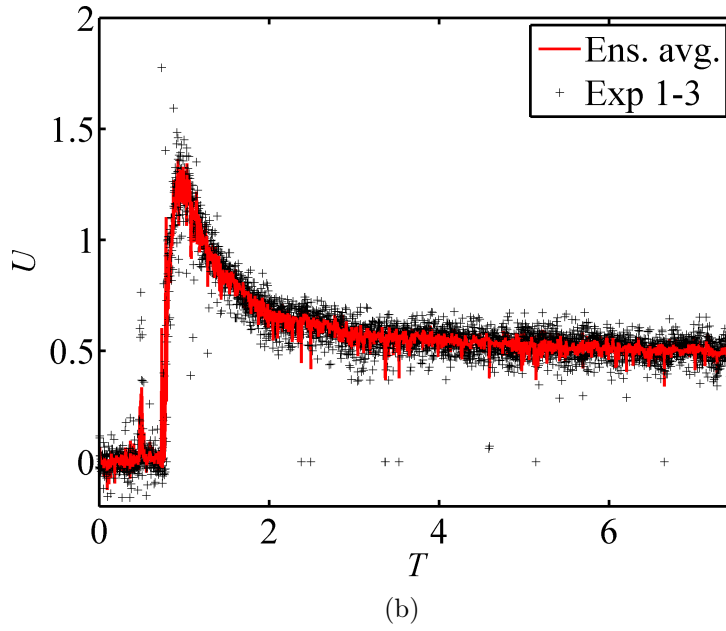
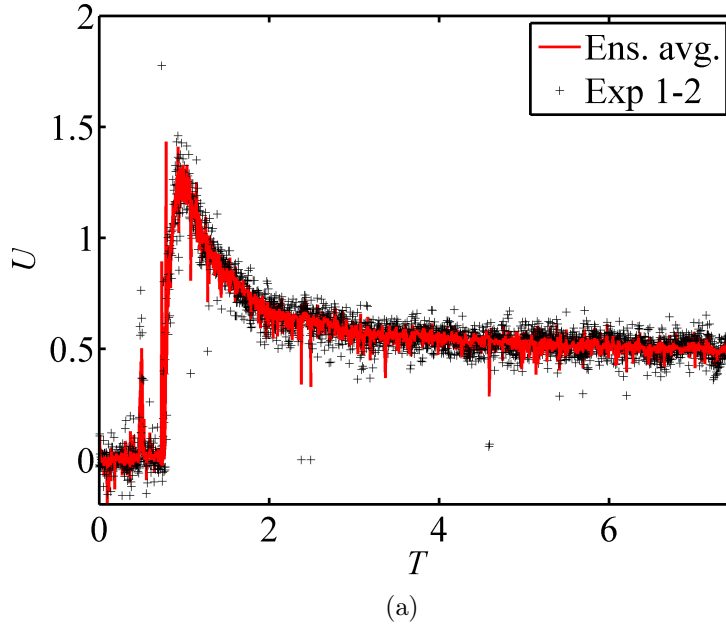


Figure 4.8: Ensemble averages considering (a) 2 experiments: h32.5-hs0-D0-BPP-run01 and h32.5-hs0-D0-BPP-run02 (b) 3 experiments: h32.5-hs0-D0-BPP-run01 to h32.5-hs0-D0-BPP-run03.

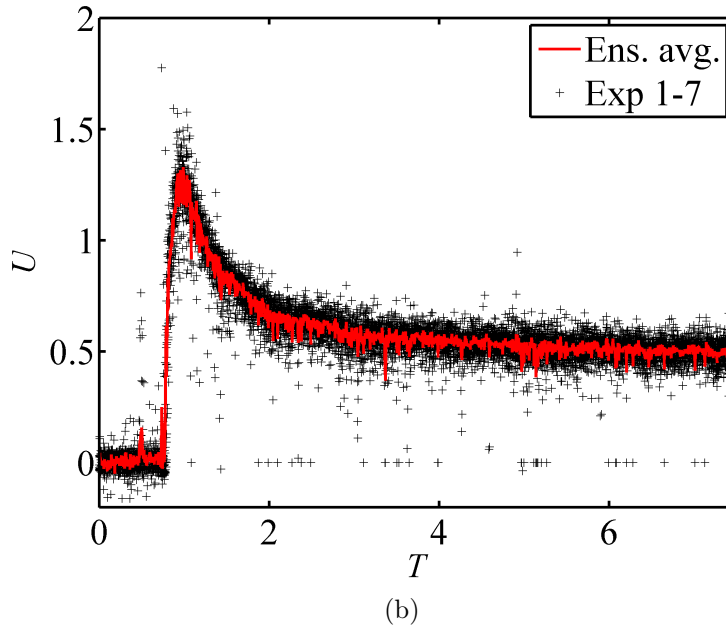
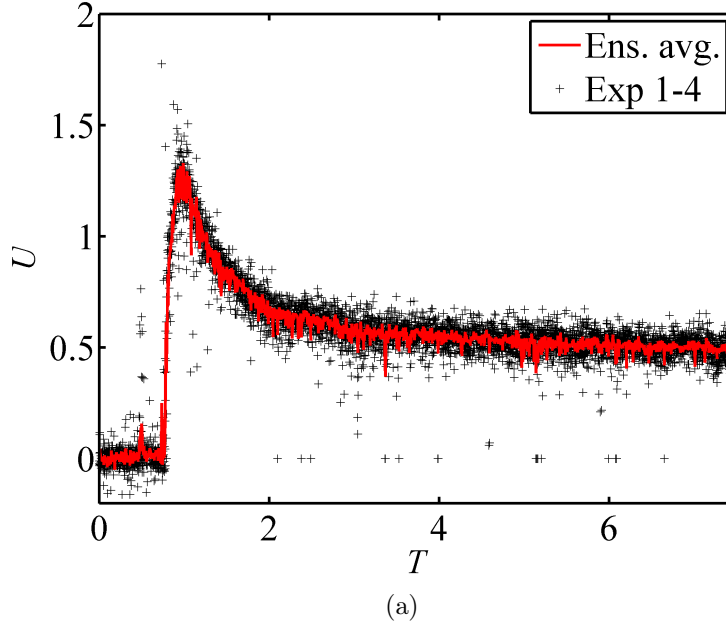


Figure 4.9: Ensemble averages considering (a) 4 experiments: h32.5-hs0-D0-BPP-run01 to h32.5-hs0-D0-BPP-run04 and (b) comparison between the ensemble average (whole data) and application of equation 4.14 to experiment 1 with $t_2 - t_1 = 0.01$ s, that is, considering 10 data points.

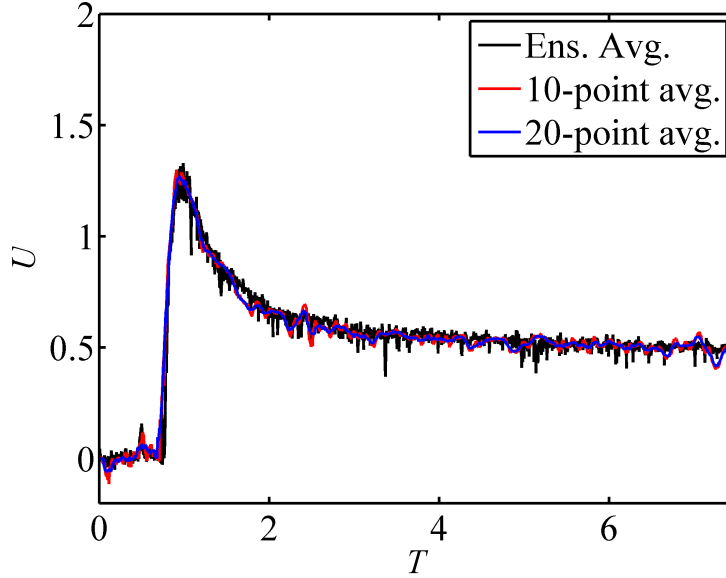


Figure 4.10: Comparison between the ensemble average (7 runs) and application of equation 4.14 to experiment 1 with $t_2 - t_1 = 0.01$ s, and $t_2 - t_1 = 0.02$ s that is, considering 10 and 20 data points respectively.

(Tukey, 1977). The starting point of this despiking method is to consider that the median is a robust estimator of the mean in order to generate a smooth sequence that can be subtracted from the original signal Goring and Nikora (2007). A complete description of this method is presented in Annex D. One of the requirements of this method is to have a defined constant mean value. This is difficult to obtain for the case of a transient flow. For the present case, after the passing of the dam-break wave ($T > 2.5$), one has, for the case depicted in Figure 4.11, the instantaneous values of U oscillating around a constant value. Figure 4.12 (a) and (b) depicts the distribution function of U and W measured in point D considering the interval $[0.69, 7.46]$, i.e., since the beginning of the passing dam-break wave to the end and Figure 4.13 represents the distribution function of U and W only for $T > 2.5$. It is possible to

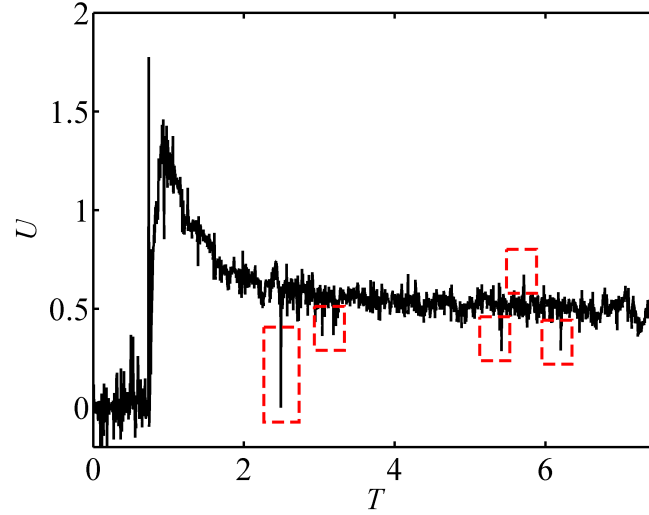


Figure 4.11: Identification of possible spikes in the signal of the vertical component U measured at point D .

observe that for this last case, the distribution of U and W seems to follow a gaussian curve. However, by means of the χ^2 test this hypothesis (gaussian distribution) was rejected.

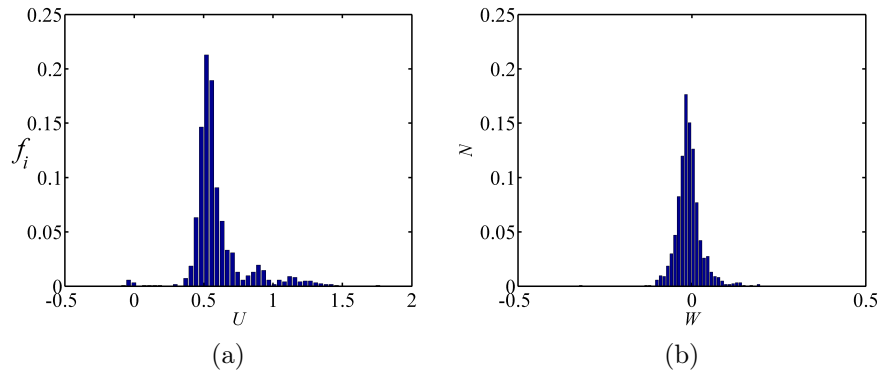


Figure 4.12: Velocity distribution of (a) U and (b) W for point D considering the complete measurement interval $\Delta T = [0.69, 7.46]$. f_i is the relative frequency per bin.

It is possible to extend this reasoning to the other points of the

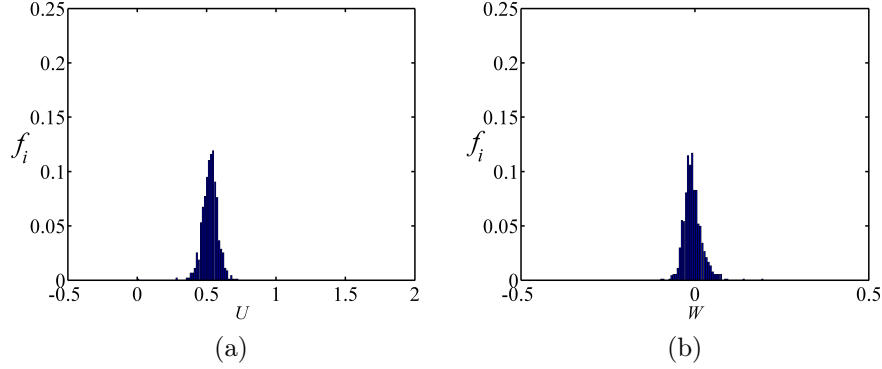


Figure 4.13: Velocity distribution of (a) U and (b) W for point D considering $T > 2.5$, f_i is the relative frequency per bin.

measurement domain, where after the passing of the dam-break wave, a constant mean value can be defined in a certain time interval, $\Delta T_c = \Delta t_c/t_0$. In this interval, ΔT_c , is then possible to apply the Tukey 53 H method to filter the spikes from the original signal. An example of the filtering made by the Tukey 53H method is presented in Figures 4.14 and 4.15 for both horizontal and vertical components respectively.

The fact that during a certain interval ΔT_c one assumes that the mean value is constant was the motivation to proceed to determine the time averages of the dam-break flow in that referred interval. For the present case: $2.5 < \Delta T_c < 7.47$. The flow field corresponding to the extremities of this interval is depicted in Figures 4.16 and 4.17. Several features are visible: a small recirculation zone immediately after the gate section ($0 < X < 0.2$, induced by the gate waterproofing system (Figure 4.18). For $T = 2.5$ at $X \approx 0.6$, in the near wall region, i.e., $0 < Z < 0.1$ the velocity vectors are slightly directed upwards (Figure 4.19, feature that does not occur for $T = 7.47$. For both cases, near the gate region defined by $0.05 < Z < 0.3$ and $-0.05 < X < 0.05$ the velocities are much smaller than what should be expected.

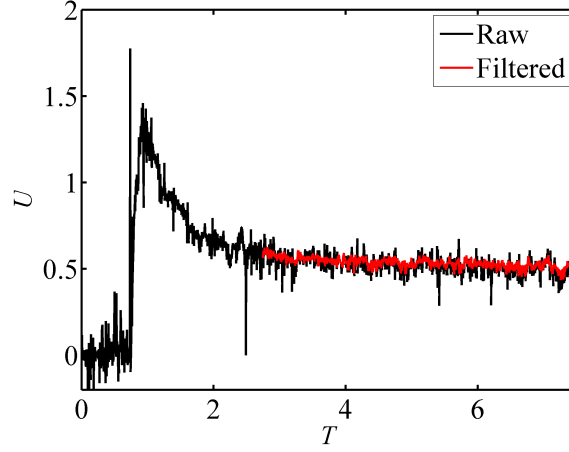


Figure 4.14: Application of Tukey 53H despiking method to the velocity component U measured at point D : (experiment h32.5-hs0-D0-BPP-run01).

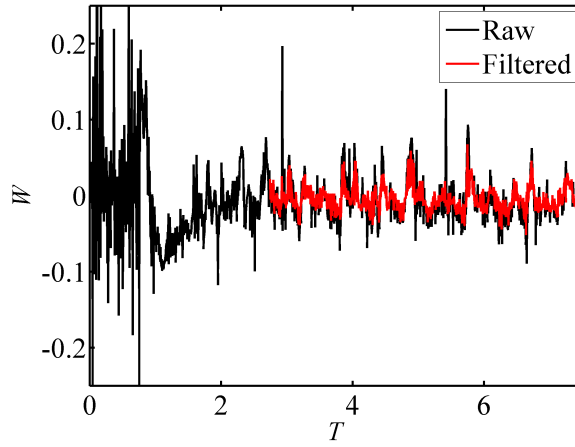


Figure 4.15: Application of Tukey 53H despiking method to the velocity W , measured at point D . (Experiment h32.5-hs0-D0-BPP-run01).

In this time interval ΔT_c one has a reasonable number of samples N . For the present case $400 < N < 950$, and although smaller than the values usually taken in permanent flow ($N \geq 3500$ see section 4.1). From the filtered signal, time statistics were defined

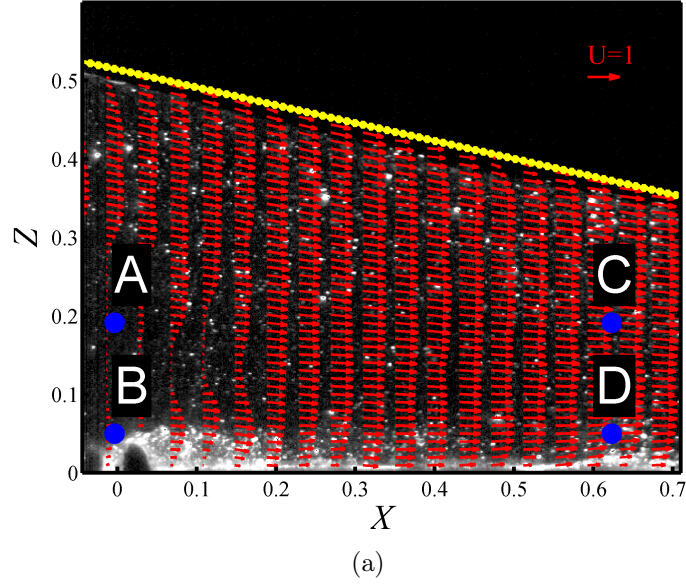


Figure 4.16: Flow field measured for $T = 2.5$. (Experiment h32.5hs0D0BPPrun01).

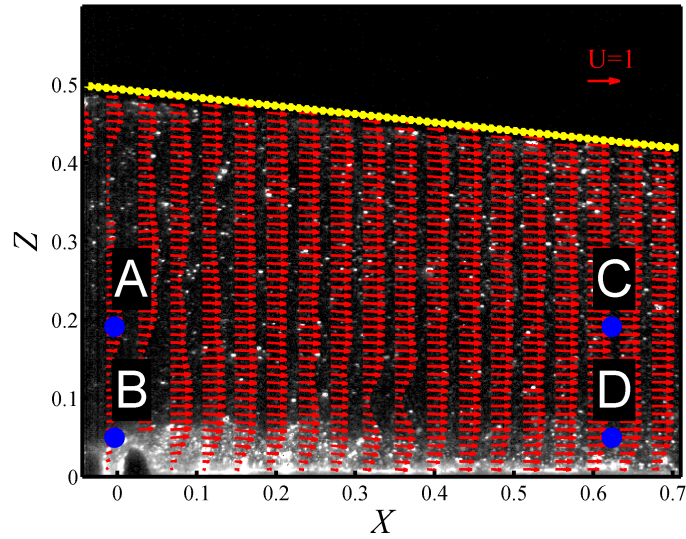


Figure 4.17: Flow field measured for $T = 7.47$. (Experiment h32.5hs0D0BPPrun01).

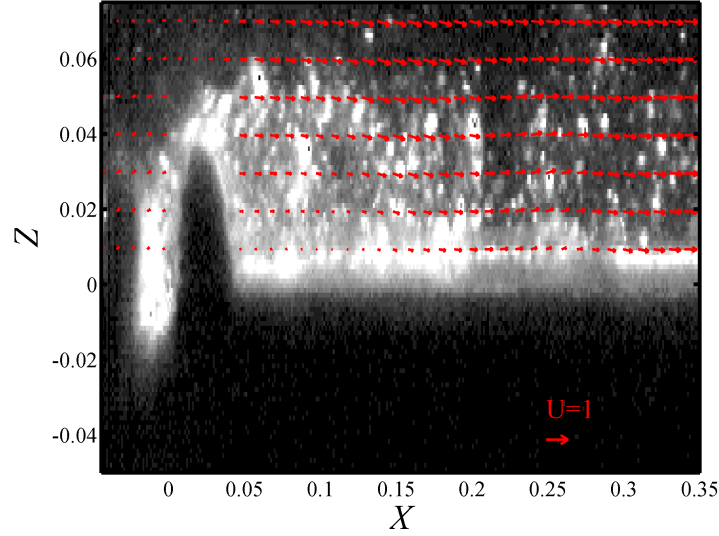


Figure 4.18: Close-up of the near-wall region at gate the gate section for $T = 2.5$. The gate waterproofing system (plasticine) disturbs the flow. (Experiment h32.5hs0D0BPPrun01).

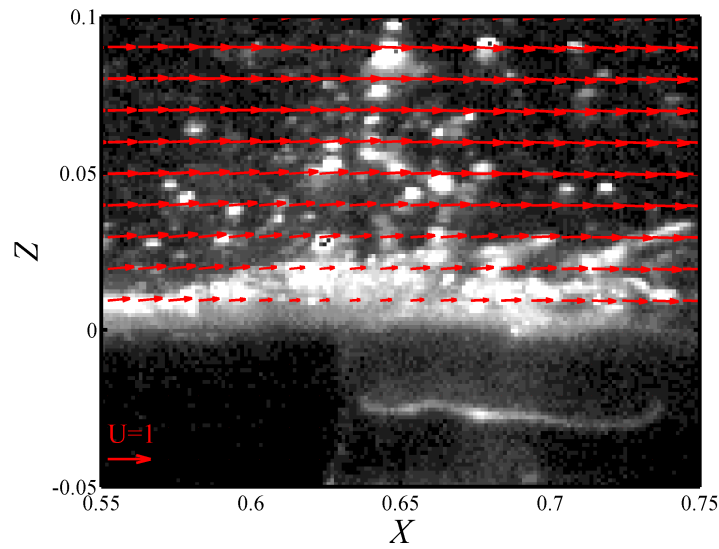


Figure 4.19: Close-up of the near-wall region at $0.55 < X < 0.75$ and $0 < Z < 0.1$ for $T = 2.5$. (Experiment h32.5hs0D0BPPrun01).

using equations (4.4) and (4.8). The time averaged flow field in the time interval $2.5 < \Delta T_c < 7.47$ is depicted in Figure 4.20. The standard-deviation for horizontal and vertical components is depicted in Figures 4.21 and 4.22. The cross correlation coefficient can be computed from equation (4.9) and it is given by:

$$R_{uw} = \frac{\overline{u'w'}}{\sigma_u \sigma_w} \quad (4.15)$$

and it is shown in Figure 4.23. For representation purposes, and in every depicted case, the free surface was considered to be the free surface at the $T = 7.47$.

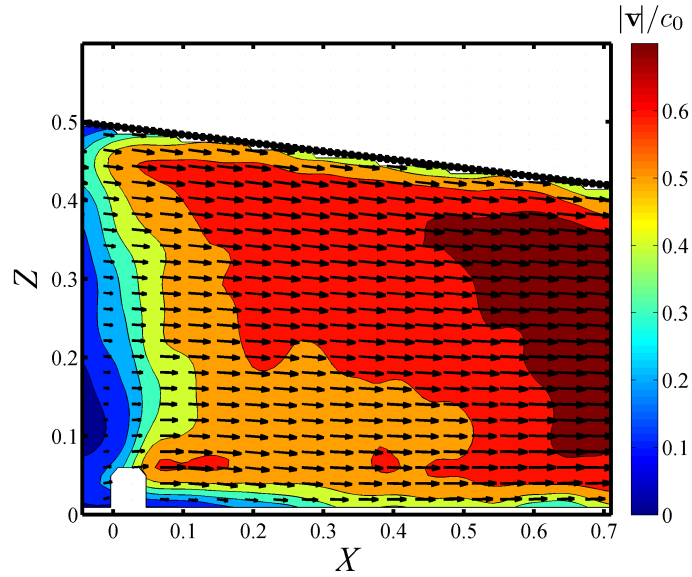


Figure 4.20: Time average of the velocity field in the interval $\Delta T_c = [2.5, 7.47]$. (Experiment h32.5hs0D0BPPrun07; for clarity only 1 vector out of 4 depicted in the horizontal direction and 1 out of 2 in the vertical direction).

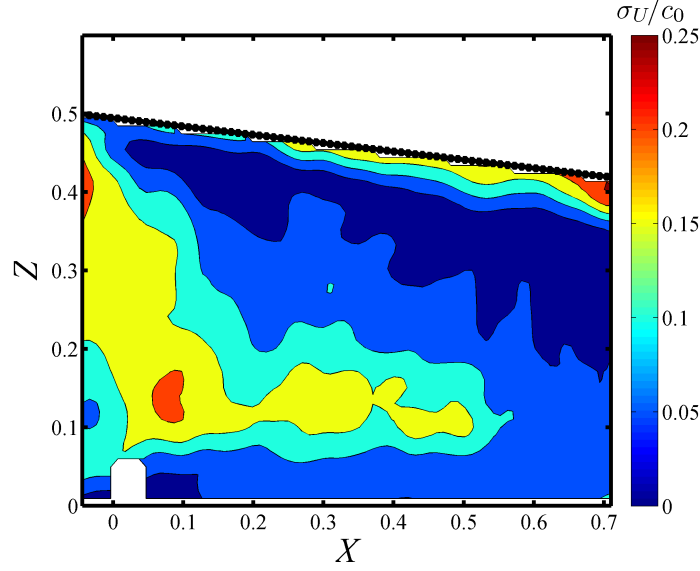


Figure 4.21: Standard deviation of the horizontal component of the velocity, $\sigma_U = \sigma_u/c_0$, computed in the interval $\Delta T_c = [2.5, 7.47]$. (Experiment h32.5hs0D0BPPrun07).

4.3 Ensemble Averages Over Time Averages

With the seven runs, ensemble averages were performed. From the flow images six sections, Sections S1 to S6, are chosen for which a detailed statistical analysis in the near bed region is made. The coordinates of the different sections are indicated in Table 4.2.

Table 4.2: Sections where the velocity profiles were obtained from experiments h32.5-hs0-D0-BPP-run01-h32.5-hs0-D0-BPP-run07 .

Section	S1	S2	S3	S4	S5	S6
X	0	0.077	0.15	0.30	0.45	0.60

With the existing data set the ensemble statistics of the measured variables, $\bar{u}$, $\bar{w}$, σ_u , σ_w were computed. The goals for this procedure were the following: a) to smooth out outliers and b) to

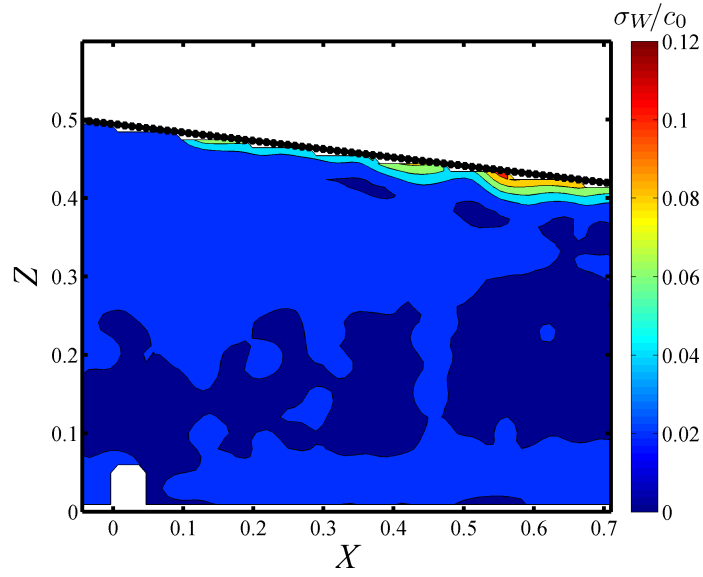


Figure 4.22: Standard deviation of the vertical component of the velocity, $\sigma_W = \sigma_w/c_0$, computed in the interval $\Delta T_c = [2.5, 7.47]$. (Experiment h32.5hs0D0BPPrun07).

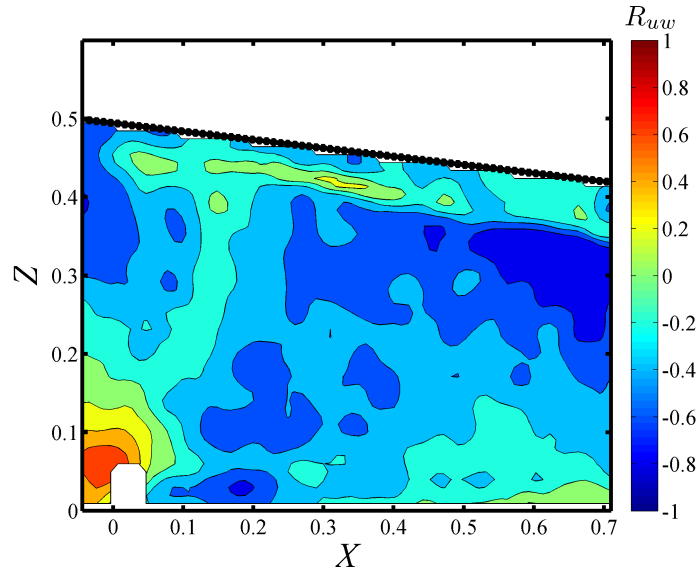
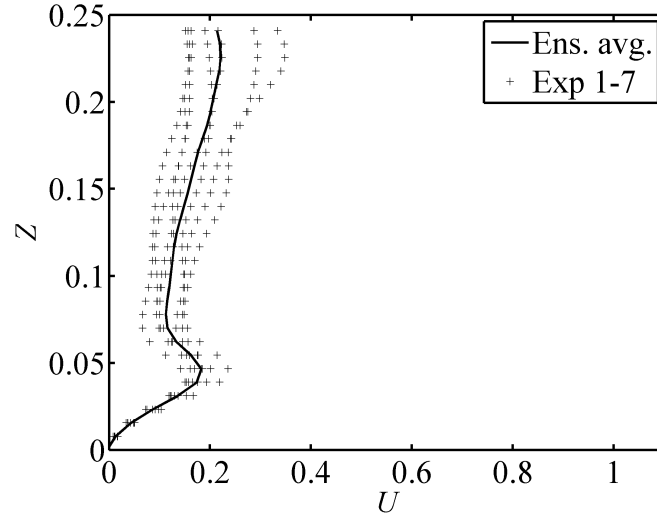
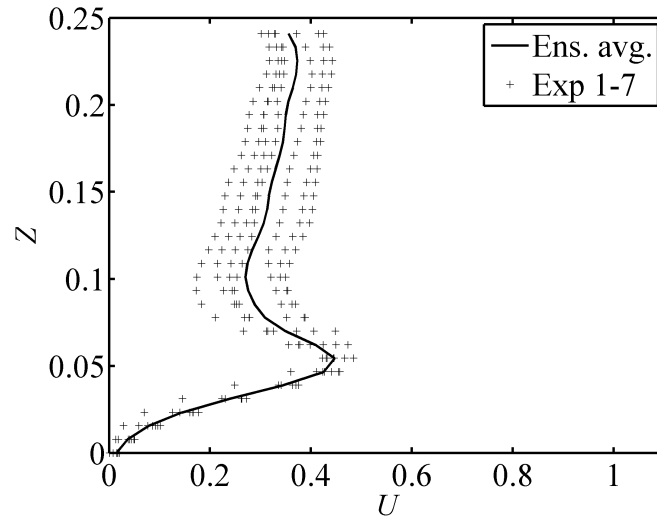


Figure 4.23: Cross correlation R_{uw} in the interval $\Delta T_c = [2.5, 7.47]$. (Experiment h32.5hs0D0BPPrun01).

check if the obtained profiles revealed some consistency. The results are presented in Figures 4.24 to 4.29.

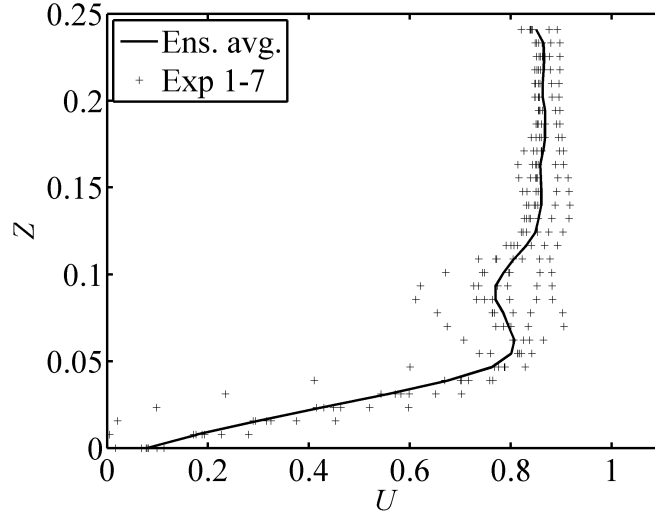


(a)

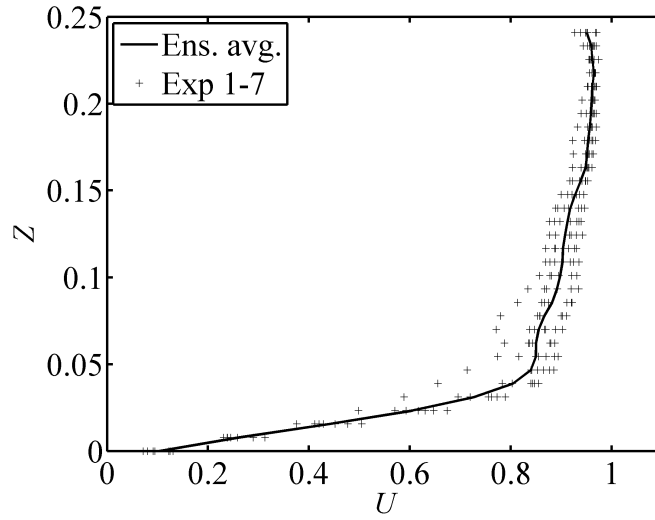


(b)

Figure 4.24: Comparison between mean values of U for each experiment (dots) and ensemble average (solid line) at (a) $X = 0$ and (b) $X = 0.077$.

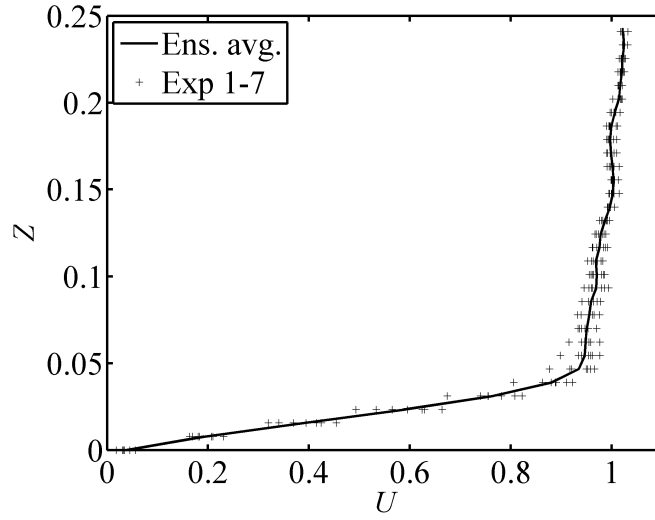


(a)

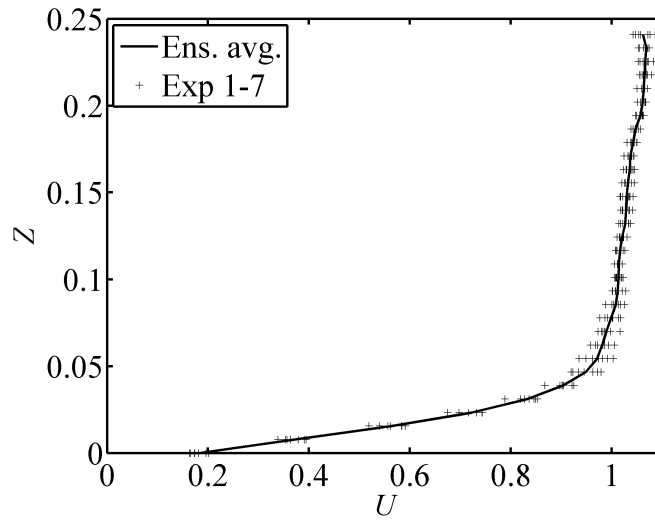


(b)

Figure 4.25: Comparison between mean values of U for each experiment (dots) and ensemble average (solid line) at (a) $X = 0.15$ and (b) $X = 0.30$.

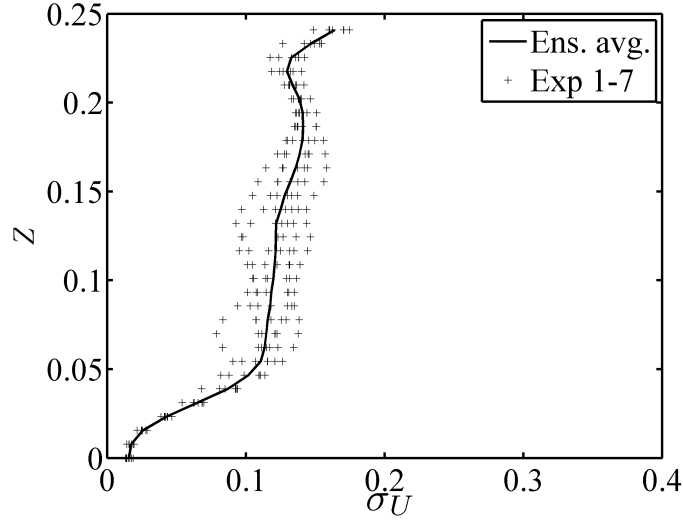


(a)

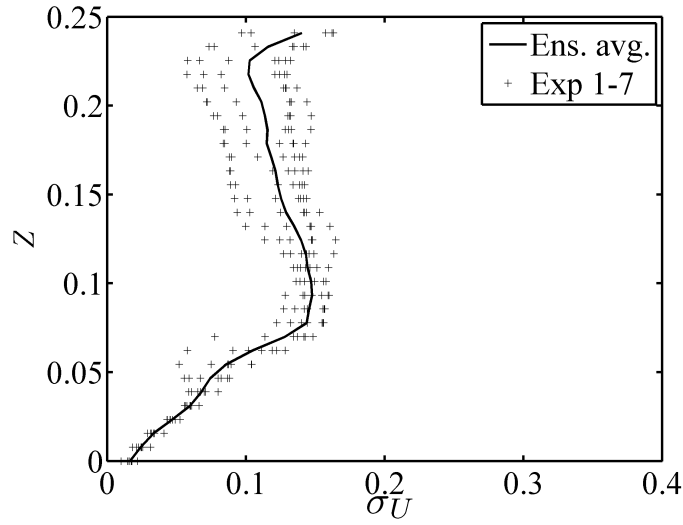


(b)

Figure 4.26: Comparison between mean values of U for each experiment (dots) and ensemble average (solid line) at (a) $X = 0.45$ and (b) $X = 0.60$.

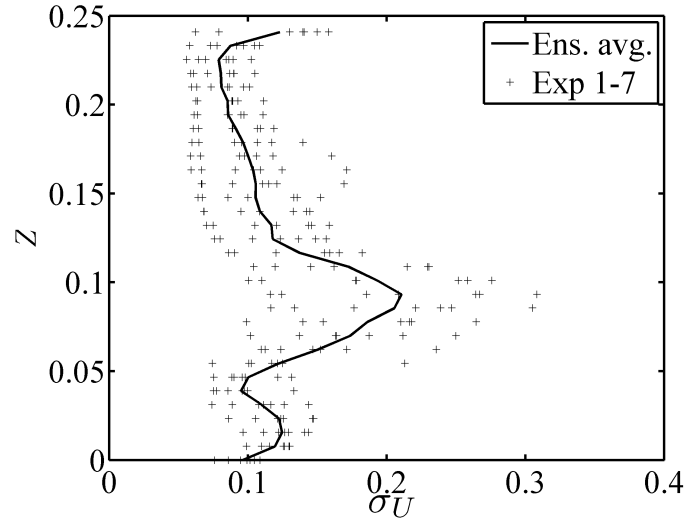


(a)

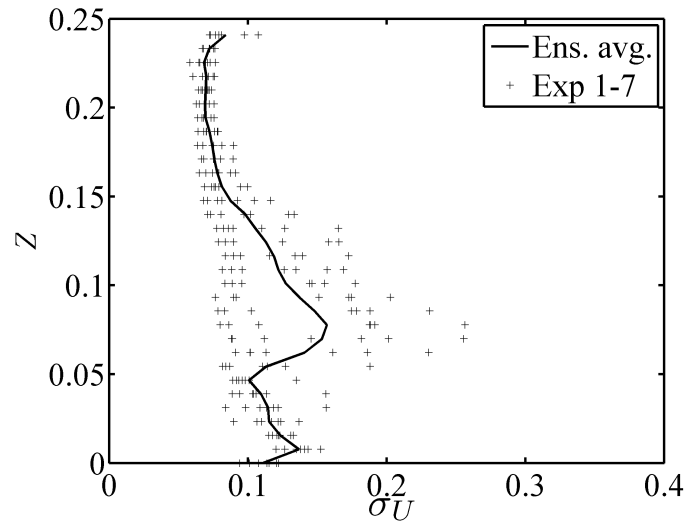


(b)

Figure 4.27: Comparison between the standard deviations, σ_U for each measurement (dots) and ensemble averaged standard deviations (solid line) at a) $X = 0$ and (b) $X = 0.077$.

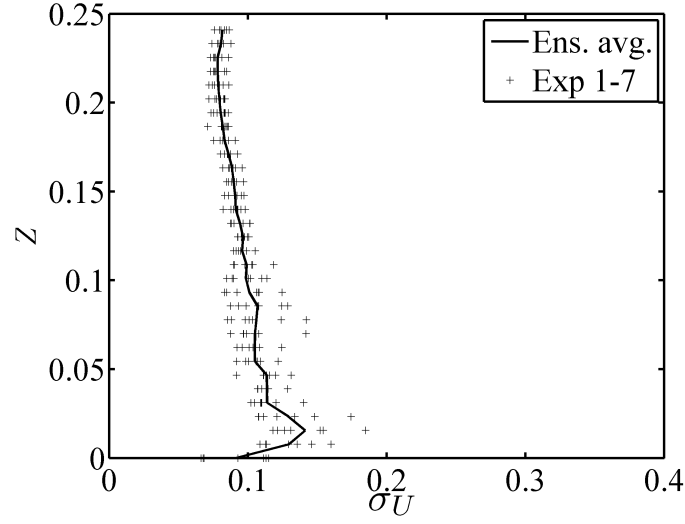


(a)

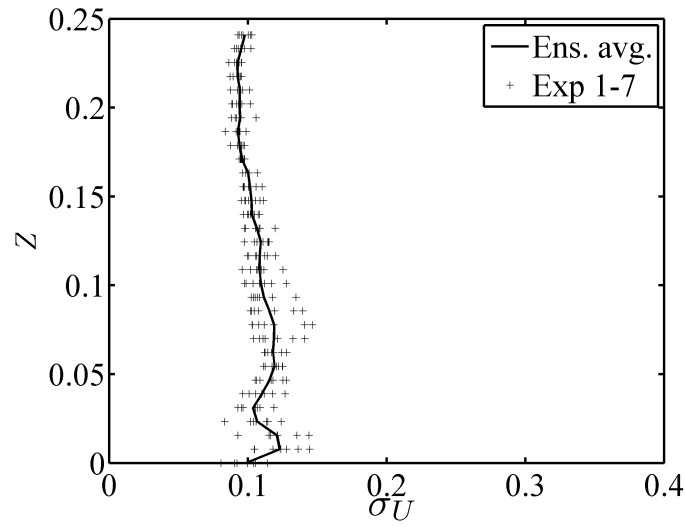


(b)

Figure 4.28: Comparison between the standard deviations, σ_U for each measurement (dots) and ensemble averaged standard deviations (solid line) at $X = 0.15$ and $X = 0.30$.

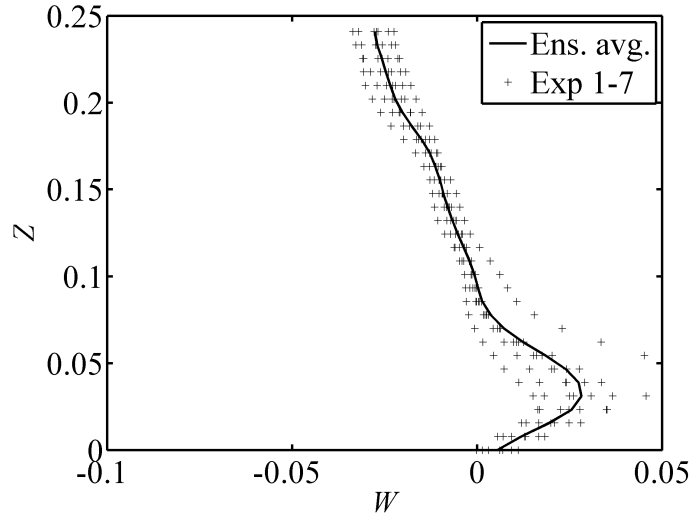


(a)

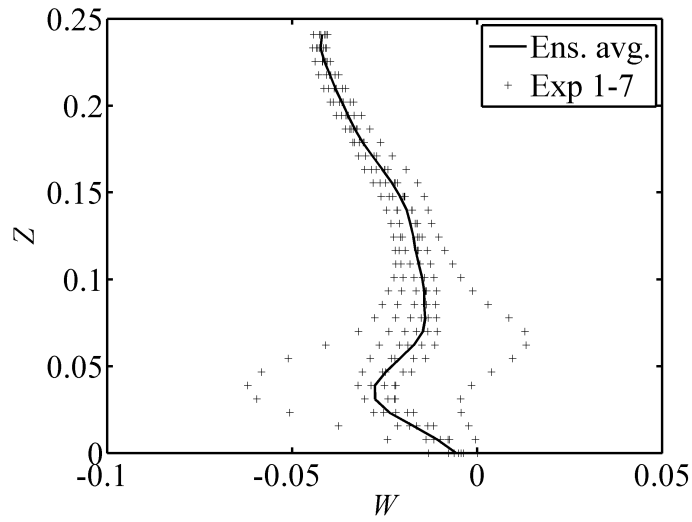


(b)

Figure 4.29: Comparison between the standard deviations, σ_U for each measurement (dots) and ensemble averaged standard deviations (solid line) at (a) $X = 0.45$ and (b) $X = 0.60$.

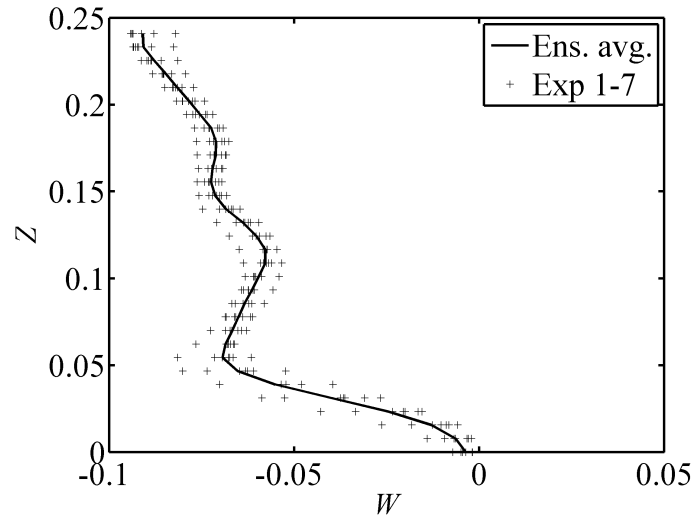


(a)

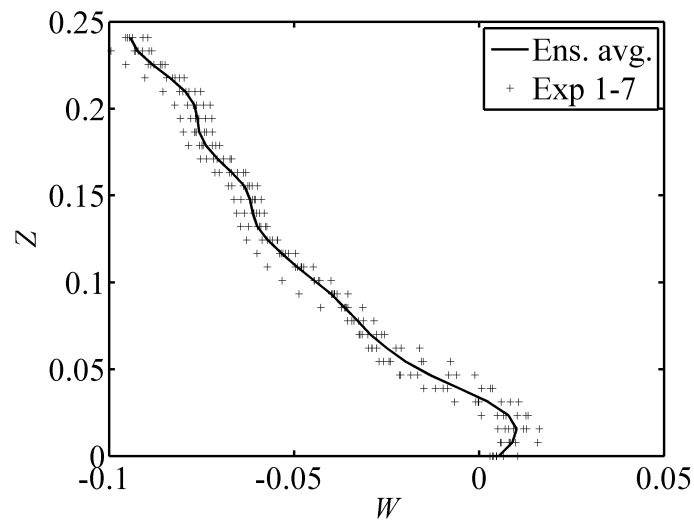


(b)

Figure 4.30: Comparison between mean values of W for each experiment (dots) and ensemble average (solid line) U at (a) $X = 0$ and (b) $X = 0.077$.

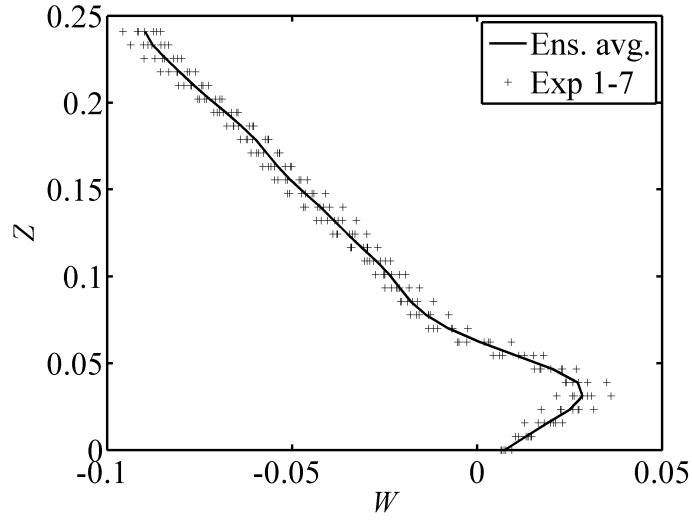


(a)

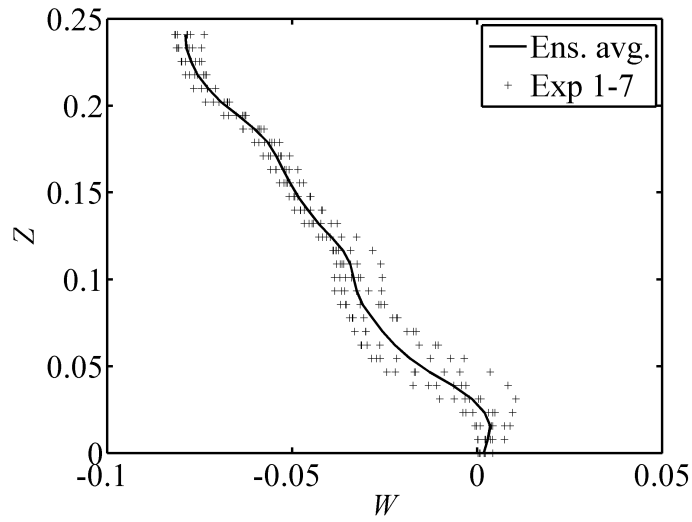


(b)

Figure 4.31: Comparison between mean values of W for each experiment (dots) and ensemble average (solid line) at (a) $X = 0.15$ and (b) $X = 0.30$.



(a)



(b)

Figure 4.32: Comparison between mean values of W for each experiment (dots) and ensemble average (solid line) at (a) $X = 0$ and (b) $X = 0.60$.

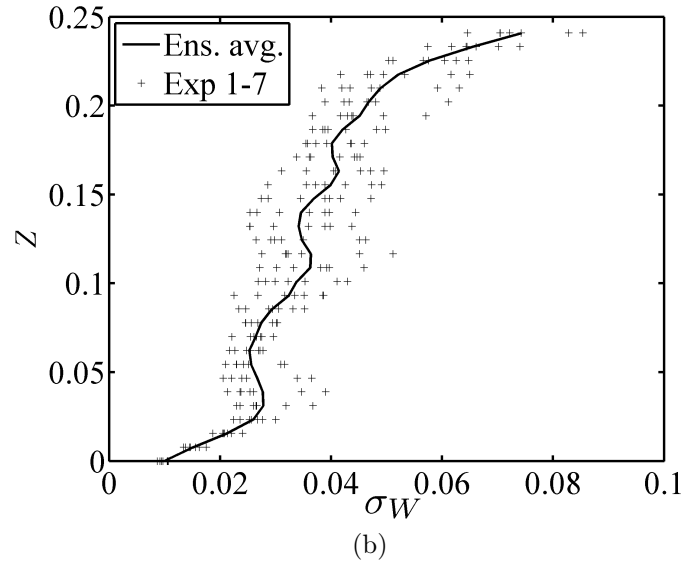
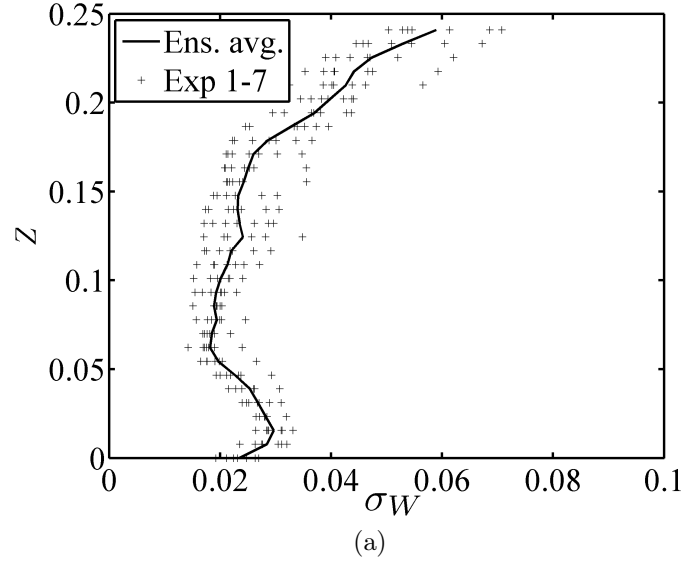
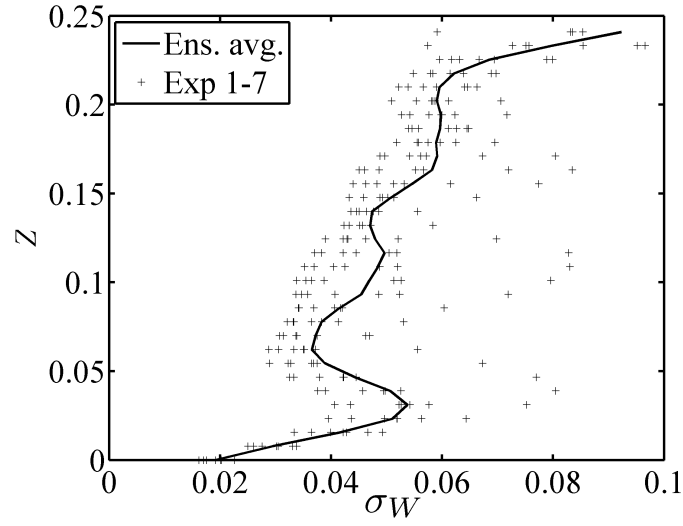
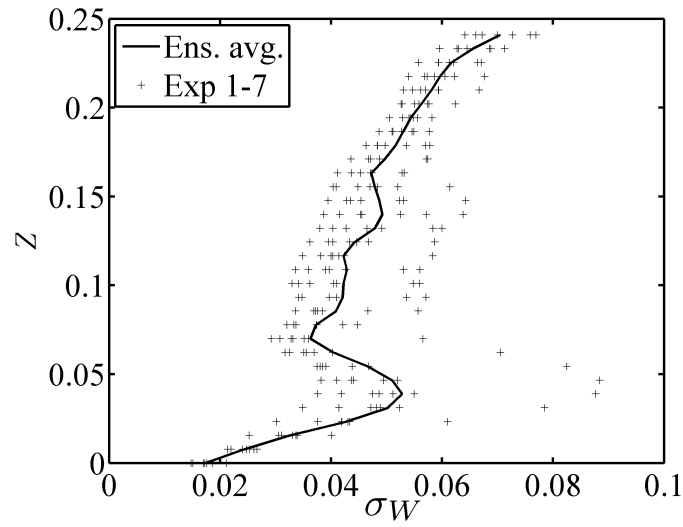


Figure 4.33: Comparison between the standard deviations, σ_W for each measurement (dots) and ensemble averaged standard deviations (solid line) at (a) $X = 0$ and (b) $X = 0.077$.



(a)



(b)

Figure 4.34: Comparison between the standard deviations, σ_W for each measurement (dots) and ensemble averaged standard deviations (solid line) at (a) $X = 0.15$ and (b) $X = 0.30$.

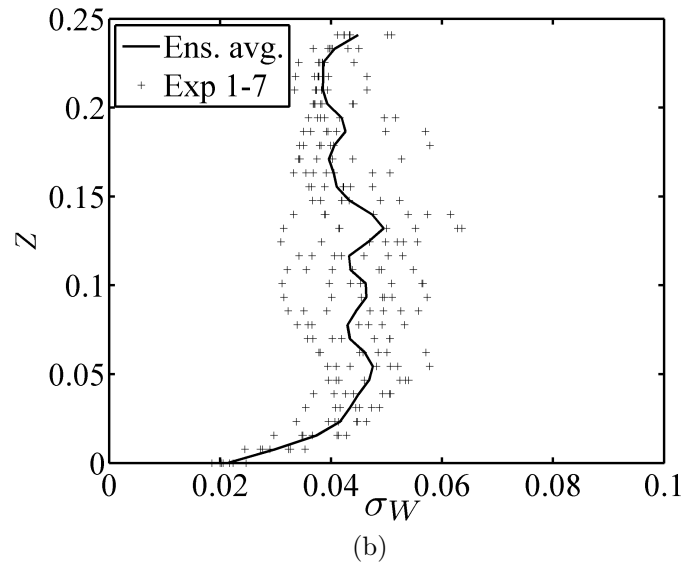
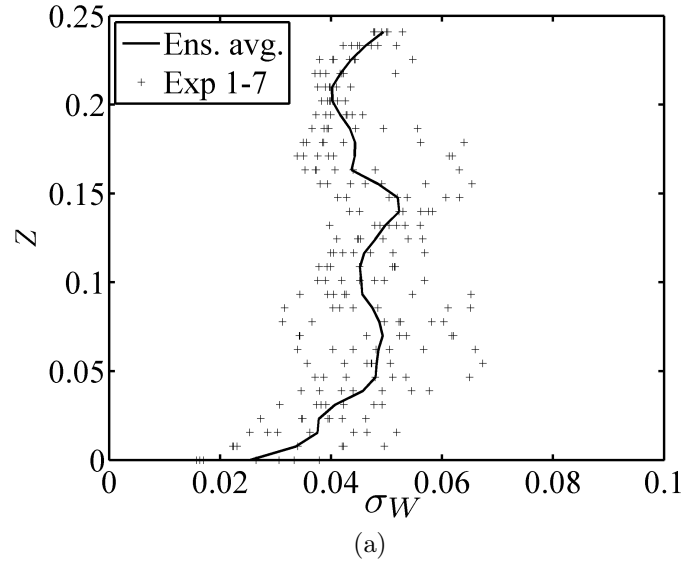


Figure 4.35: Comparison between the standard deviations, σ_W for each measurement (dots) and ensemble averaged standard deviations (solid line) at (a) $X = 0.45$ and (b) $X = 0.60$.

4.4 UVP Velocity Profiles Analysis

This section presents the experiments made in the dam-break channel concerning the determination of the velocity profiles with the UVP technique. It is presented after the section where the ensemble averages since the obtained velocity profiles are ensemble-averaged. Three set of experiments were used one for each considered measurement section: S0, S1 and S2. S0 is located in the upstream reservoir and S1 and S2 in the test section. Their coordinates are indicated in Table 4.3. The goals of this section are the assessment of the UVP as a measuring technique for the early stages of the dam-break flow and the accurate determination of the velocity profiles for the computation of the shear stress by means of the methods proposed in section 2.3.

To obtain the profiles of the horizontal component $U = u/c_0$ with increased spatial resolution the UVP technique was used. A UVP transducer with a frequency of 4 MHz was placed under the channel's bottom in an adapted designed support, as referred in subsection 2.2.8. In section S0 a spatial resolution of 0.74 mm was used and the maximum measurement height above the bed was set at 0.86 m. The sampling frequency was 111 Hz. In Sections S1 and S2 due to the characteristics of the flow (high transient) each measurement control volume was set at 0.74 mm and in order to maximize its sampling frequency the measurement profile height was limited to 0.068 m above the transducer tip divided in 91 measuring points. With this configuration a sampling frequency of 250 Hz (acquisition time of 4 ms) was used. The measurements were therefore limited to the dam-break flow bottom region.

The results here shown refer to the ensemble average of all the experiments made in the same conditions (see section 2.5 for experiments details). Table 4.3 resumes the three different locations along the channel's centreline and the maximum height above the

channel's fixed smooth bed that was measured with the UVP.

Table 4.3: Location of the two sections for UVP measurements and maximum heights scanned.

Section	x (m)	X	$h_{\max}$	$H_{\max}$	Experiment
S0	-0.65	-2	0.086	0.27	h32.5-hs0-D0-BWG
S1	0.30	0.92	0.068	0.2	h32.5-hs0-D0-BP-S1
S2	0.80	2.46	0.068	0.2	h32.5-hs0-D0-BP-S2

For section S0, Figure 4.36 illustrates the time evolution of the velocity profiles between $0 \leq T \leq 14$. The vertical axis depicts the non-dimensional height above the bed whereas the abscissa represents the non-dimensional time T . This representation allows one to have a global perspective on how U evolves with time. For $T < 1$ one can see that the negative wave has not reach section S0 yet. The flow is practically at rest. Negative values are measured due essentially to noise. For $T > 1$ the velocity starts increasing due to flow's acceleration. A region in the coordinates space $Z - T$ of the figure 4.36, comprised between $5 < T < 8$, shows a velocity peak that seems to propagate from the higher values of Z to the lower ones. This effect is put into evidence in Figure 4.37, where the time series of velocity at different heights is plotted as function of T .

As it was observed with the imaging techniques, it would be expected that the velocity would increase monotonically up to a maximum value, and then would progressively decay (e.g. Figure 4.6). The observed peaks in Figure 4.37 are a feature related to the UVP functioning, due to the free surface time evolution. This UVP feature affects the velocity profiles for the considered time instants as it can be seen in Figure 4.38. However, for $8 < T < 14$ the measured velocity profiles are no longer affected. For $T = 14$, however, the upper part seems once again affected.

For the section S1 the same processing was applied. Figure 4.40 depicts the time evolution of the velocity profiles between

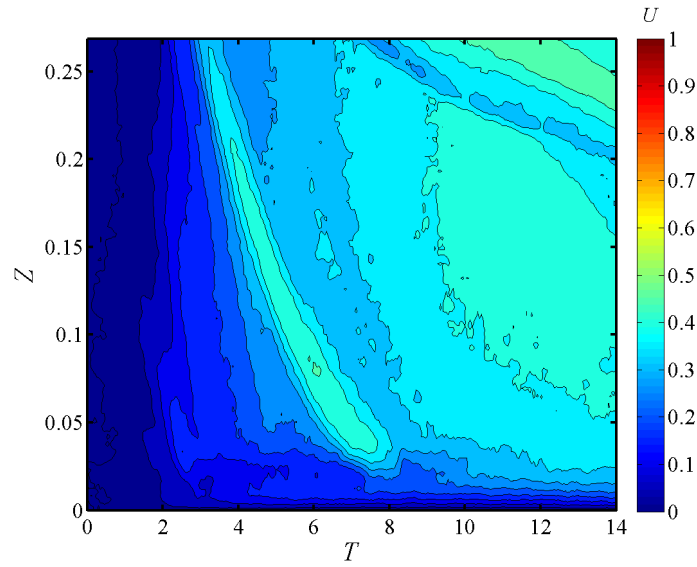


Figure 4.36: Colour map of the velocity profile in $X = -2$.

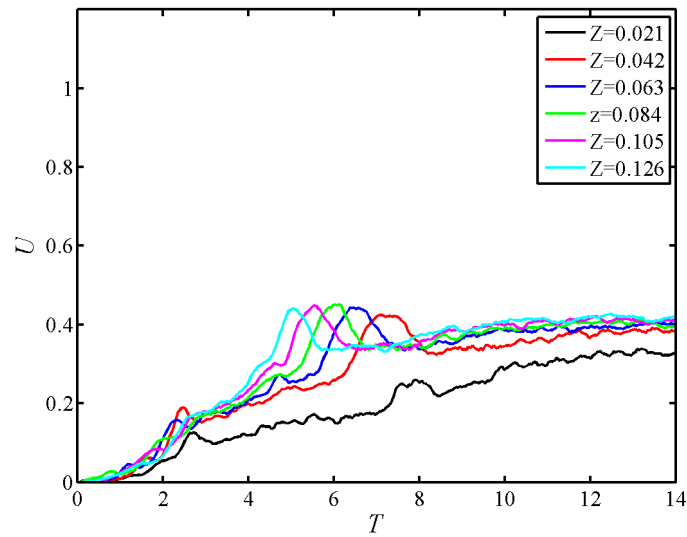


Figure 4.37: Velocity time series for different heights at $X = -2$.

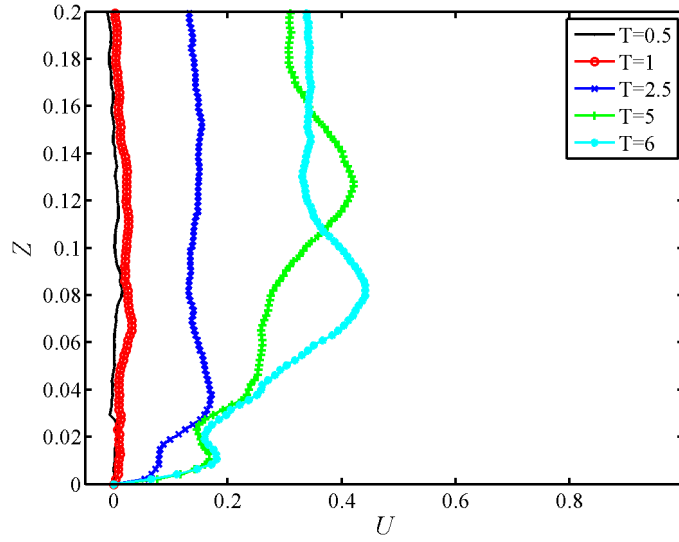


Figure 4.38: Measured U velocity profiles in section $X = -2$ at different times: $T = 0.5$, $T = 1$, $T = 2.5$, $T = 5$ and $T = 6$.

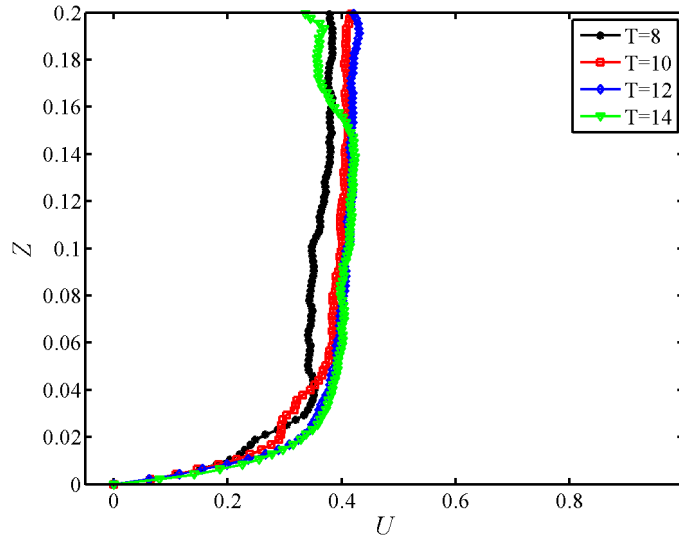


Figure 4.39: Measured U velocity profiles in section $X = -2$ at different times: $T = 8$, $T = 10$, $T = 12$ and $T = 14$.

$0 < T < 14$. Before the dam-break wave arrival, no velocity is measured. When the dam-break wave arrives the UVP starts measuring, but due to rapid change of conditions from air to water, erroneous measurements appear for $Z > 0.05$ and from Figure 4.41 it is possible to see that the UVP takes some time to adapt to the flow conditions, for $Z = 0.126$ one has $T \approx 4.5$. The behaviour depicted in Figure 4.41 is somehow similar to the behaviour observed in Figure 4.6. For $T > 5$ due to the complete immersion of the UVP control volumes (Figures 4.42 and 4.43) the velocity profiles become coincident.

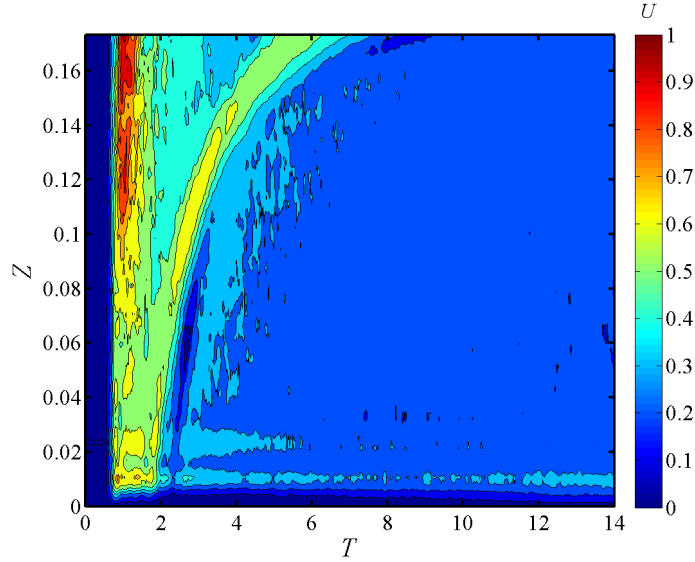


Figure 4.40: Colour map of the velocity profile in $X = 0.92$.

For section S2 the same reasoning is valid, see Figure 4.44 where the time evolution of the velocity profiles are depicted. Long duration oscillations are visible in the time series (Figure 4.45). For $Z = 0.12$, the oscillations stops at $T \approx 12$ ($T \approx 4.5$ in section S1). And in the measured velocity profiles (Figures 4.46 and 4.46) one can still observe oscillations for $T = 8$ and $T = 10$.

From these observed features, and when compared with the

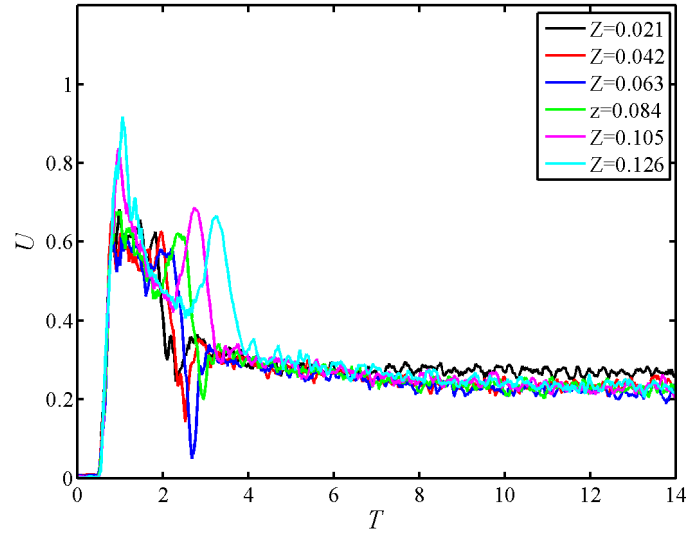


Figure 4.41: Velocity time series for different heights at $X = 0.92$.

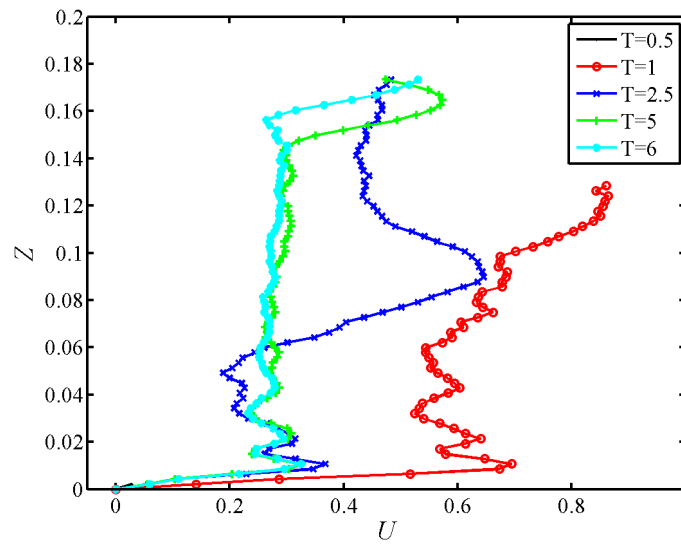


Figure 4.42: Measured U velocity profiles in section $X = 0.92$ at different times: $T = 0.5$, $T = 1$, $T = 2.5$, $T = 5$ and $T = 6$.

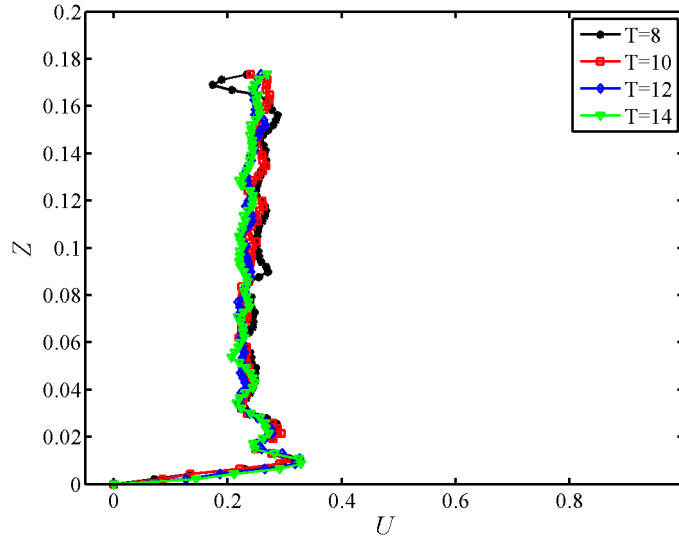


Figure 4.43: Measured U velocity profiles in section $X = 0.92$ at different times: $T = 8$, $T = 10$, $T = 12$ and $T = 14$.

UVP behaviour in the case of permanent flow, the following conclusions were drawn:

1. If possible the upper boundary of the profile should be defined, that is, for the UVP system to work in proper conditions a fixed boundary (bed or slowly changing free surface) should be imposed.
2. If possible, all the control volumes should stay immersed, not the case for the initial instants.
3. Despite its good time resolution, the UVP requires some time to adapt to the regions with water flow and without.
4. This characteristics show that to study the initial instants of the dam-break flow by means of a UVP is not necessarily a good option.

However, the measurements made in the reservoir ($X = 2$) for

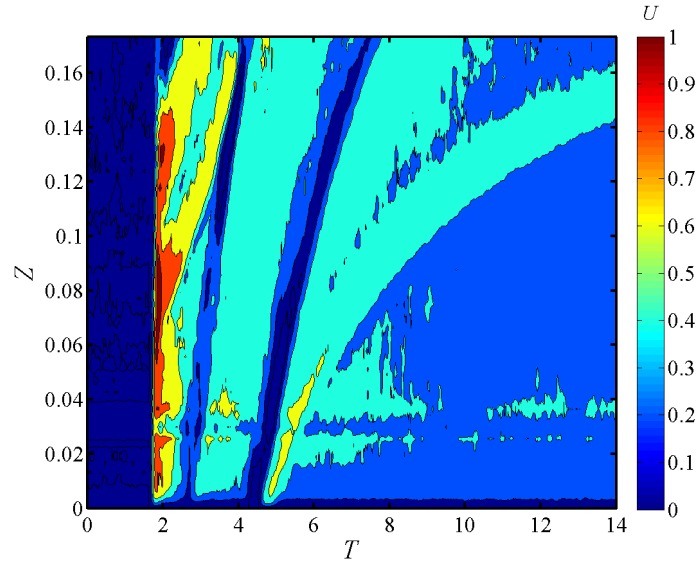


Figure 4.44: Colour map of the velocity profile in $X = 2.46$.

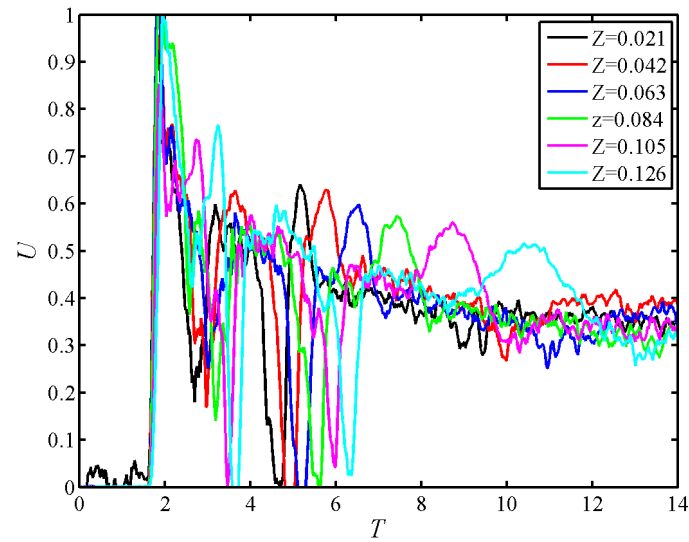


Figure 4.45: Velocity time series for different heights at $X = 2.46$.

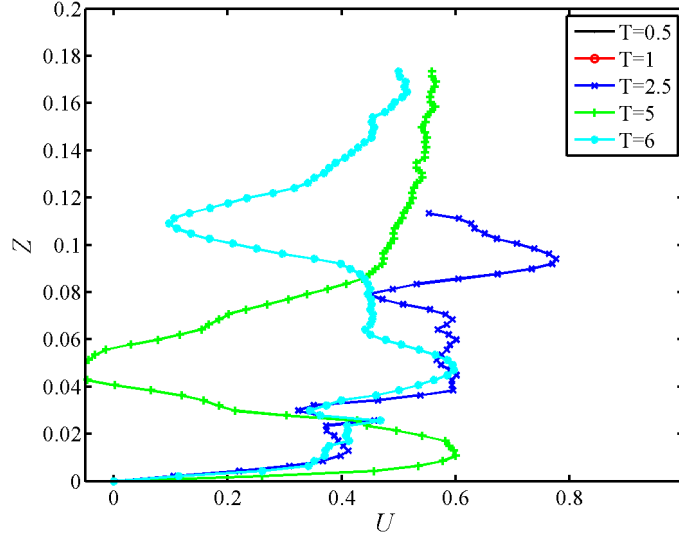


Figure 4.46: Measured U velocity profiles in section $X = 2.46$ at different times: $T = 0.5$, $T = 1$, $T = 2.5$, $T = 5$ and $T = 6$.

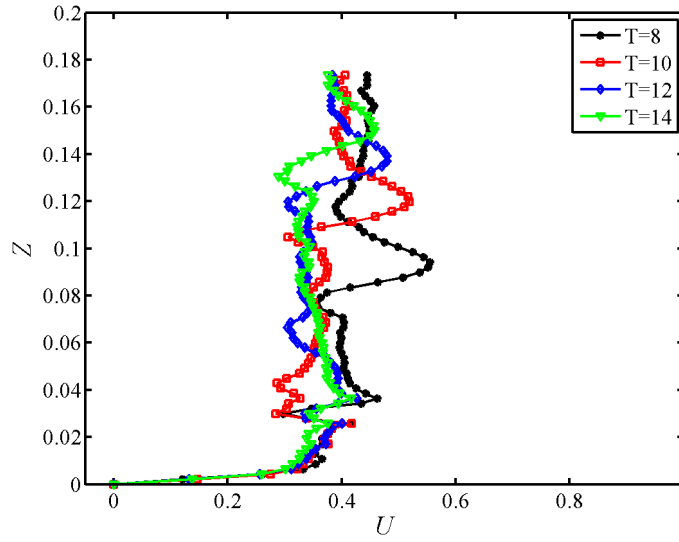


Figure 4.47: Measured U velocity profiles in section $X = 2.46$ at different times: $T = 8$, $T = 10$, $T = 12$ and $T = 14$.

larger times can be analysed. An analysis is here presented in order to determine the shear stress for the late times: $T = 10$, $T = 12$ and $T = 14$. These profiles were chosen due to their , as seen in Figure 4.39. For these times the boundary layer parameters were computed (see Annex E). For the considered times one has $3.9 \times 10^3 < \text{Re}_{\delta^*} < 5.6 \times 10^3$.

Following the ideas presented in section 2.3, the velocity profiles are adimensionalised by U_δ , the velocity at $z = \delta$, and δ is the boundary layer height (height for which $u(\delta) = 0.99U_o$) respectively and depicted in Figure 4.48.

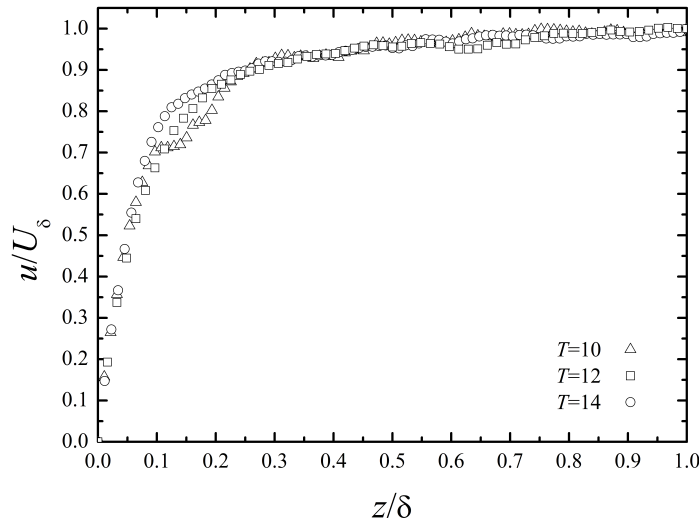


Figure 4.48: Non dimensional velocity profiles for $X = -2$ and $T = 10, 12, 14$.

From equation (2.34) one can write:

$$\frac{u}{U_\delta} = \frac{u_*}{U_\delta \kappa} \ln \left(\frac{z}{\delta} \frac{\delta u_*}{\nu} \right) + B \quad (4.16)$$

and by the logarithm properties (eq. 4.16) can be written as:

$$\frac{u}{U_\delta} = \frac{u_*}{U_\delta \kappa} \ln \left(\frac{z}{\delta} \right) + B' \quad (4.17)$$

where

$$B' = \frac{u_*}{U_\delta \kappa} \ln \left(\frac{\delta u_*}{\nu} \right) + B \quad (4.18)$$

For graphical purposes the natural logarithm in equation (4.17) is here changed for base-10 logarithm, and with $\kappa = 0.37$ one has:

$$\frac{u}{U_\delta} = 6.22 \frac{u_*}{U_\delta} \log \left(\frac{z}{\delta} \right) + B' \quad (4.19)$$

In a semi-logarithmic plot (4.19) represents a straight line:

$$\frac{u}{U_\delta} = m \log \left(\frac{z}{\delta} \right) + B'. \quad (4.20)$$

Where the slope m is given by:

$$m = 6.22 \frac{u_*}{U_\delta}. \quad (4.21)$$

Which is a function of the shear velocity $u_* = \sqrt{\tau_w / \rho_w}$, and therefore of the shear stress τ_w (see equation (2.31)).

In Figure 4.49 the velocity profiles for $X = -2$ and the above mentioned instants are plotted in semi-logarithmic coordinates (base 10). It is possible to see that for $z/\delta < 0.1$ a linear region appears. In that region a straight is fitted to the experimental data. The obtained parameters for equation (4.21) are indicated in Table 4.4.

The obtained results for u_* and τ_w need a comparison with other sources. Due to the simple geometry of the problem a numerical analysis of the problem by means of a Computational Fluid Dynamics software should be made in the future to compare the numerical

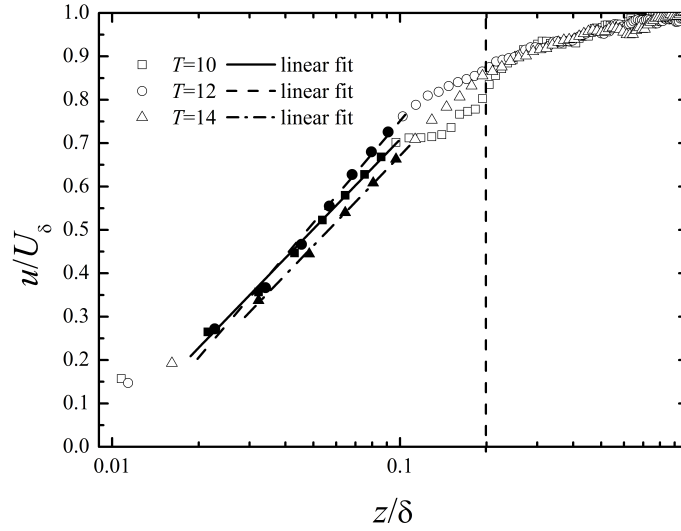


Figure 4.49: Non dimensional velocity profiles for $X = -2$ and $T = 10, 12, 14$ in semi-logarithmic coordinates. Filled points used for the linear fit. Vertical dashed line indicates $z/\delta = 0.2$ the maximum value for which the log law is valid (Graf and Altinakar, 2001). Note that the logarithm is in base 10.

Table 4.4: Obtained parameters for equation (4.19).

T	m	B'	R^2	U_δ (m/s)	u_* (m/s)	τ_w (N/m ²)
10	0.687	1.396	0.998	0.737	0.0814	6.61
12	0.779	1.530	0.996	0.757	0.0948	8.97
14	0.690	1.361	0.999	0.747	0.0828	6.85

results with the experimental ones. On the other hand, other measurement techniques should also be tested to directly measure the shear stress.

Chapter 5

Conclusions

The fundamental things apply, as time goes by
"As Time Goes By" (Herman Hupfel)

To write down the conclusions of this thesis it is helpful to recall the objectives initially proposed in the section 1.2 (Objectives of the thesis):

- To analyse the initial instants of the dam-break flow, not only from the point of view of water surface evolution but also from the velocity field point of view.
- To investigate the hypothesis behind a two-layer shallow water numerical model regarding the velocity field of both sediment and water layer.
- To lay down a methodology concerning the statistical analysis of the dam-break flow.
- To apply alternative measurement techniques to the case of the dam-break flow, namely the UVP velocity profile meter.

Regarding the first objective, the initial stages were analysed by means of imaging techniques. This allowed to describe those stages

and compare it with the work by Pohle (1952) and Estrade (1967). As seen in section 3.1.1 the use of imaging techniques allowed to describe the trajectories of seeded particles in the flow. It was seen that the model proposed by Pohle (1952) agrees well with the measured results for $T < 0.71$ which is in close agreement with the time proposed by Estrade (1967).

The study of the initial instants of the dam-break flow allowed as well to investigate the role of the gate that simulates the dam-break flow in the flow itself. It was demonstrated that the criterion proposed by Lauber and Hager (1998a) for upward moving gates was not necessarily valid for downward moving ones. See Appendix A for more information.

The gate effect is of fundamental importance for the first stages analysis. The presence of sediments can jam the door and lead, from run to run, to different removal times, as it was seen when investigating the 2D characteristics of the flow (see annex C).

The analysis of the hydrodynamic dam-break flow in terms of velocity field allowed also to establish a time criterion to define at which point the flow starts to be within the St.-Venant conditions. This time was found to be $T \approx 2.5$. This value was found to be approximately the same for the sedimentologic case. More investigation on this subject is required to determine the reason why these values are so close.

A part of the experiment made to determine the velocity field used the PTV algorithm initially developed by Capart et al. (2002). A binning method was proposed and implemented allowing to structure the obtained velocity field. A criterion for the bins size was proposed. This criterion is based on the particles displacement between consecutive frames.

The repeatability of the dam-break flow, proven for the water height evolution by Zech et al. (2008), was also shown for the velocity field. With this property it is possible to complete a set of data

by repeating the dam-break experiments in the same conditions.

The sedimentologic dam-break flow, using as sediments sand, allowed to investigate the hypothesis regarding the shallow water two-layer model developed by Capart and Young (2002), Spinewine (2005) and Savary (2007). It was possible to concluded that:

- The velocity profile in the sediment layer is not constant but linear.
- The velocity profile in the water layer is constant (potential).
- The simultaneously measured velocity profiles allowed also to check for the continuity of the velocity profile between the different layers.

The numerical model can now be further improved based on the findings of this study for the velocity field; and in the results obtained by Spinewine et al. (2011a,b) regarding the concentration profiles.

The measurements carried on the sediment layer allowed also to proposed a method to estimate its limits. Taking advantage that the moving sediment layer behaves like a shear layer, this method is based on the determination of the curl of the velocity. It has the advantage of avoiding complex image processing tasks. The results show that this method can be used as a predictor, i.e., as a first step for a more detailed analysis. Numerical model predictions of the sediment layer and experimental results were found to be in close agreement.

Another type of sedimentologic dam-break flow was considered using bed material PVC pellets. The use of a high speed camera allowed for the time description of the initial stages of the dam-break flow, with a time resolution of 1 ms. The spatial resolution was enough to observe at the grain scale the fluid-bed interaction.

Time Resolved PIV was successfully used to obtain the quantitative time description of the water wave collapse. The water wave collapse and consequent energy dissipation still have to be studied in more detail using models from Maritime Hydraulics, for example. With this set of measurements a region of the dam-break flow was found where statistical variables were calculated.

With the available measurement techniques it was possible to do a statistical analysis the dam-break flow with some limitations. To a set of experiments, the ensemble statistics tools were applied. The first idea was the start of a turbulent description of a transient flow like the dam-break flow. It was verified that the time scale of the dam-break flow tested is not enough to obtain a sufficiently large set of data where higher order moments would converge. From the data analysis it was, however, possible to conclude that, after the incoming dam-break wave passes, it is possible to identify a time interval where the flow behaves as a quasi-stationary flow. In that interval statistical variables of the dam-break flow were calculated.

Finally, from the initially proposed objectives the UVP was tested in the conditions of a dam-break flow. It was verified that although it features fast acquisition frequency and good spatial resolution, the instrument does not respond as fast as it would be needed to measure the incoming wave. On the other hand, by comparison with measurements carried on permanent flow, it was verified that having all the UVP acoustic beam immersed and with fixed or slowly moving boundaries would greatly improve the measurements. UVP measurements made in the reservoir section of the channel showed that a logarithmic region exists in the velocity profile for $T \geq 10$.

Overall, the experiments made also allowed also obtaining a data base that has been used to validate models by other researchers namely Canelas et al. (2012), Swartenbroekx et al. (2011).

Ideas to Explore in the Future

In the future the following subjects should be investigated as a follow up of this thesis.

The gate removal process could be re-analysed considering:

1. A systematic comparison between upward and downward driven gates.
2. A gate with an adjustable time removal circuit.
3. An hybrid upward and downward mechanism.

In the present case only the complete removal of the gate was considered; however, more complex situations can be simulated by considering, for example, an incomplete gate removal, as depicted in Figure 5.1.

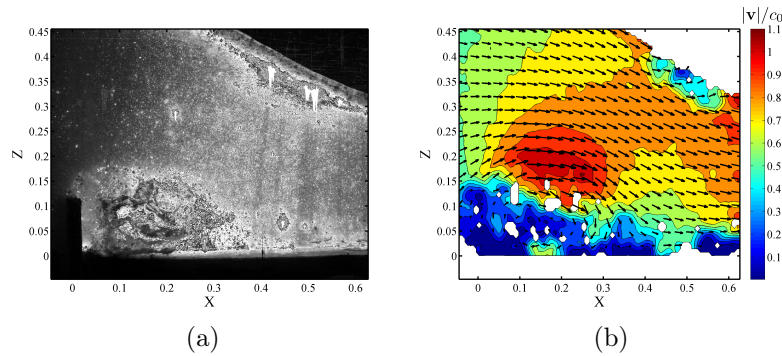


Figure 5.1: Flow pattern and velocity field for a non completed removed gate, $h_0 = 0.325$ m and $T = 2.23$. Final gate height, $Z = 0.12$.

Another point to look at is the bed rugosity. For the hydrodynamic case only smooth beds were tested. It is possible to repeat the same measurements considering a different rigid bottom with different rugosities (e.g. sand paper).

The determination of the shear stress for smooth and rough beds should be tried by using an adequate device, namely a floating element balance or an optical probe as described in Carvalho et al. (2010).

The test of more initial conditions, water height and sediment heights, should also be considered.

Appendix A

Ideas About the Gate Removal

As it was seen in section 2.1.1, by the Lauber and Hager (1998b) criterion the experimental case defined by the initial reservoir head, $h_0 = 0.40$ m, was considered instantaneous, whereas the measurements shown that is not the case. An attempt is here made to derive an alternative criterion.

As it was seen in section 1.5.2, Pohle (1952) showed that when the gate is rapidly removed, the free surface particles near the gate will move as a free fall body. Assuming that the no-slip condition does not hold and letting the particle position be given by (Galilei, 1632):

$$z_p = z_0 - \frac{1}{2}gt^2 \quad (\text{A.1})$$

where z_p is the particle's coordinate, g is the gravity acceleration. As seen in Figures 2.6 the gate moves with a constant acceleration and can be assumed that the gate position in time is given by:

$$z_g = z_{g0} - \frac{1}{2}at^2 \quad (\text{A.2})$$

where z_g is the gate tip position, z_{g0} is the initial gate tip position and a the gate acceleration. Considering an initial height of h_0 the gate removal time is given by:

$$t_{gr} = \sqrt{\frac{2h_0}{a}} \quad (\text{A.3})$$

At this time the uppermost particle's position is then

$$z_p = h_0 - \frac{1}{2}g\frac{2h_0}{a} \quad (\text{A.4})$$

that can be rewritten as

$$h_0 - z_p = \frac{g^2}{2a}t_0^2 \quad (\text{A.5})$$

where $t_0 = \sqrt{h_0/g}$. From this model it is required an infinite gate acceleration to have $z_p = h_0$. Practically, it is possible to assume a small variation in z_p , and still consider the dam removal instantaneous. Let

$$h_0 - z_p = \frac{1}{2}gt_r^2 \quad (\text{A.6})$$

where t_r is the time needed to a particle attend the position $h_0 - z_p$. Dividing equation (A.6) by h_0 and letting $\varpi = (h_0 - z_p)/h_0$ one has

$$\varpi = \frac{1}{2} \left(\frac{t_r}{t_0} \right)^2 \quad (\text{A.7})$$

and $0 \leq \varpi \leq 1$ is here defined as the admissible percentage of variation in the reservoir's water height to still consider the dam break as instantaneous. From equation (A.7)

$$t_r = \sqrt{2\varpi}t_0 \quad (\text{A.8})$$

For $\varpi = 1$ the criterion proposed by Lauber and Hager (1998a) (eq. 2.1) is found. It is important to define the ϖ parameter. From Figure 2.7 one sees that for $h_0 = 0.40$ m, $Z = z/h_0 \approx 0.75$, which corresponds to a significant variation of the water height, but for $h_0 = 0.325$ m, $Z \approx 95\%$, leading to a much smaller variation. From this last case it is here adopted the value $\varpi = 0.05$, and the removal times from equation (A.8) are:

$$t_r = 0.057 \text{ s} \quad \text{for } h_0 = 0.325 \text{ m} \quad (\text{A.9})$$

$$t_r = 0.064 \text{ s} \quad \text{for } h_0 = 0.40 \text{ m} \quad (\text{A.10})$$

To better compare the measured removal times with the ones proposed by Lauber and Hager (1998a) and by equation (A.8) all these results are presented in Table A.1. The proposed criterion is more restrictive than the one proposed by Lauber and Hager (1998a). If for $h_0 = 0.325$ m it is possible to accept as instantaneous, since the removal times are in the same order of magnitude as the value proposed by equation (A.9). For $h_0 = 0.40$ m that is no longer the case, since the measured removal time is about twice the value indicated by equation (A.10).

Table A.1: Comparison between the measured removal times for the two initial conditions, the Lauber and Hager (1998a) criterion and equation (A.8) with $\varpi = 0.05$.

h_0 (m)	t_r (ms)	Lauber & Hager (ms)	$\sqrt{2\varpi}t_0$ (ms)
0.325	70	257	57
0.40	120	285	64

Appendix B

Binning and Interpolation

This Annex is adapted from Aleixo et al. (2010).

Two times are here considered $T = 1$ and $T = 7$. In order to obtain better statistics a time interval centred in the chosen non-dimensional time instants was used. This non-dimensional time interval was defined for the near field to be $\Delta T = 0.275$ corresponding to a sampling of five images, and $\Delta T = 0.604$ corresponding to 11 images sampling for the far field case.

For the near field case a sensibility analysis for the bin size was performed. The bins sizes were defined by equation (2.6) where $n = 1, 2$ and 3 .

In Table B.1 the different parameters are expressed for a characteristic velocity of $u = 13.68$ pixels per time unit.

Table B.1: Obtained bin size parameters for $T = 1$.

n	Δx	# bins
1	14	57×57
2	28	28×28
3	41	19×19

In Figure B.1 the corresponding non-structured velocity field is presented.

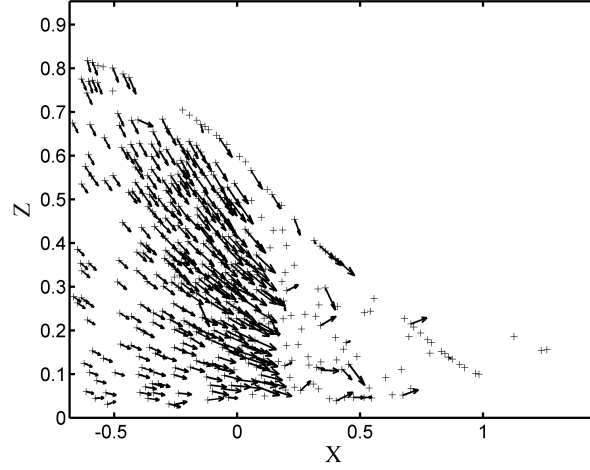


Figure B.1: Non-structured velocity field for $T = 1$ by means of the Voronoï PTV algorithm.

The two structuring methods proposed, binning and bilinear interpolation, are here applied and the results compared. The obtained results are represented in Figures B.2 to B.4. When analyzing the bin size effect (Figure B.2 a), B.3 a) and B.4 a) it is possible to see that with increasing bin size, (less bins) the flow field tend to be smoother, but with lesser spatial resolution.

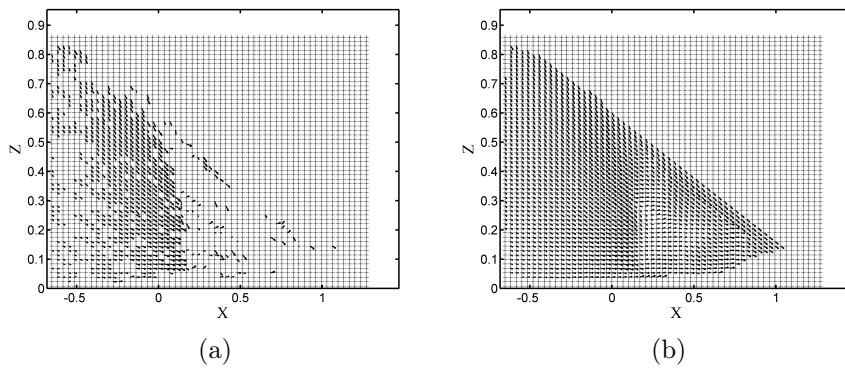


Figure B.2: Structured velocity field obtained for $T = 1$ and $n = 1$, (a) binning and (b) interpolation.

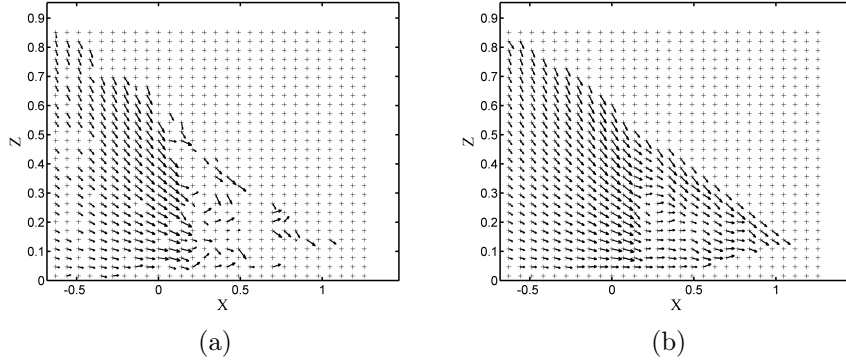


Figure B.3: Structured velocity field obtained for $T = 1$ and $n = 2$, (a) binning and (b) interpolation.

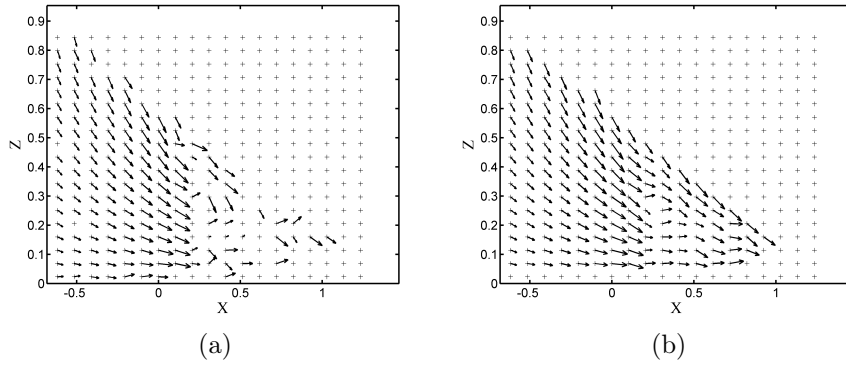


Figure B.4: Structured velocity field obtained for $T = 1$ and $n = 3$, (a) binning and (b) interpolation.

With the binning method there are data where only particles are successfully detected and tracked. When considering the interpolation method one sees that this method determines values for the velocity field in regions of the flow where no particles were tracked. These features are put into evidence in Figures B.5 to B.7 showing the velocity profiles at $X = 0$, i.e. at the gate section for the different bin sizes.

From Figures B.5 to B.6 it can be seen that for $n = 2$ one

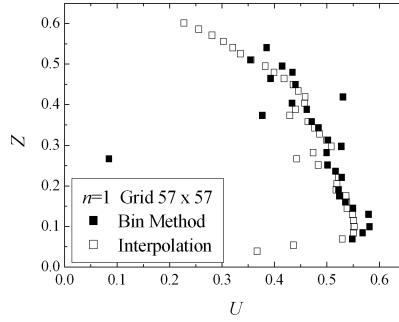


Figure B.5: Horizontal component, U for $X = 0$ and $n = 1$.

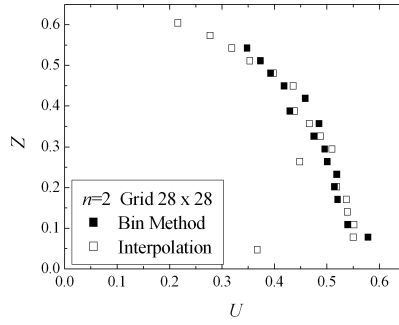


Figure B.6: Horizontal component, U for $X = 0$ and $n = 2$.

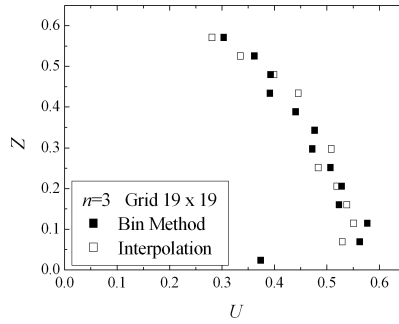


Figure B.7: Horizontal component, U for $X = 0$ and $n = 3$.

obtains a good compromise between spatial resolution and profile smoothing. It is also possible to see that the values obtained from the interpolation and the binning schemes used do agree well.

In Table B.2 some the advantages and disadvantages of each method are presented. Among the advantages of the binning level, one has to point out that it is possible to determine statistics at the bin level, on the other hand, it constitutes a 0^{th} order interpolation scheme. it is possible of course to use an interpolation schemes, with a higher order.

Table B.2: List of some of the advantages and disadvantages of the binning and interpolation method.

Method	Advantages	Disadvantages
Binning	Outlier filtering	0-th order
	No <i>artificial</i> data	Bin size criterion
	Statistics at bin level	Time consuming
Interpolation	Many schemes	No outlier filtering
	Easy to implement	Bin size criterion
	Good for derivatives	No bin statistics

Appendix C

Analysis of the Transversal Uniformity of the Flow

Since the PVC pellets bed was easier to prepare an investigation regarding the transversal uniformity of the dam-break flow over the pellets bed was also performed. This investigation encompasses the experiments h32.5-hsP010.D3.3-BW(A) to h32.5-hsP010.D3.3-BW(C) (see section 2.5) where the velocity field was measured in two different sections parallel to the channel's centreline ($Y = 0$): $Y = -0.25$ and $Y = +0.25$ by means of the classical PIV technique described in section 2.2.2. Two instants were considered a) $T = 1.1$ and b) $T = 2.6$. For $T = 1.1$ the profiles for U and W at the gate section, $X = 0$ are depicted in Figures C.1 and C.2.

It can be seen that for the water layer ($Z > 0.05$) the U component is practically coincident between the three Y -sections, however the vertical component is higher at the centreline than at $Y = -0.25$ and $Y = 0.25$ by approximatively a factor of 2. For those Y -sections ($Y = -0.25$ and $Y = 0.25$) the W profile is found to be coincident. This indicates that in the initial instants the flow can be assumed to be symmetric in relation to the channel's centreline. For the sediment layer ($Z < 0.05$) it was found that for the U component the profiles at $Y = -0.25W$ and $Y = +0.25W$ are coincident for both time instants. For the near field results the gate downward motion is of crucial importance as already

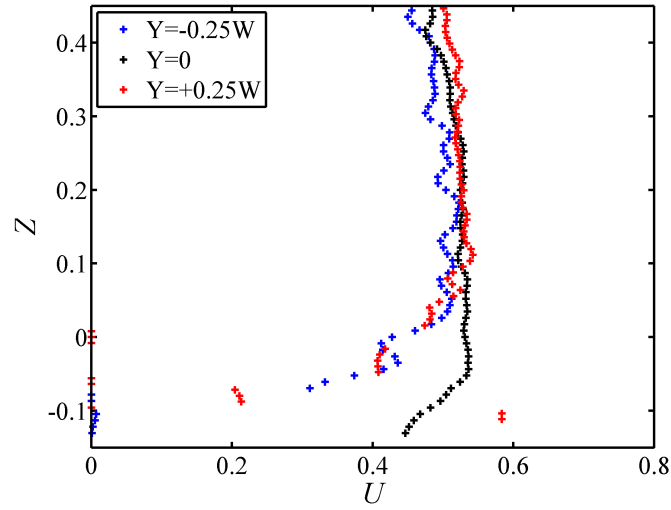


Figure C.1: Profiles of U measured at $X = 0$ in different Y -sections: -0.25, 0 and 0.25. ($T = 1.1$).

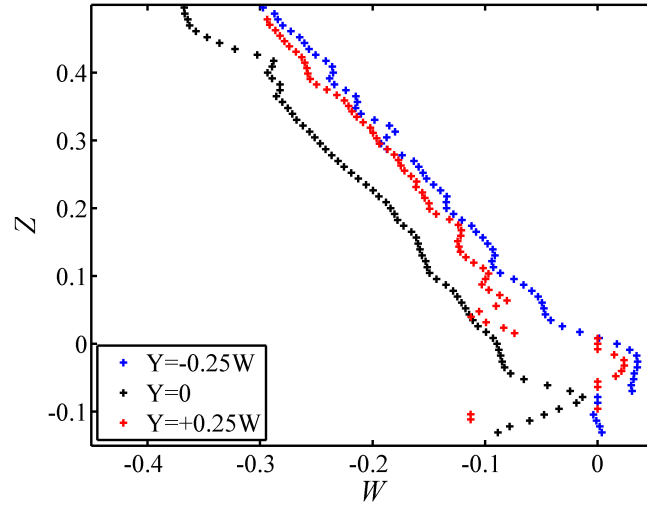


Figure C.2: Profiles of W measured at $X = 0$ in different Y -sections: -0.25, 0 and 0.25. ($T = 1.1$).

seen in section 2.1.1. The differences between the experiments made for $Y = 0$ and the $Y = \pm 0.25W$ can be related to the fact that the gate trigger circuit did not always acted at the same instant as can be seen in Figures C.3 (a) and (b), where the instants $T_r + \Delta T$ are depicted with T_r being the non-dimensional gate removal time and ΔT the time interval between consecutive PIV image pairs. If the gate movement was always the same the images would be the identical. One of the reasons for the differences in the gate motion was the presence of pellets in the gate removal mechanism.

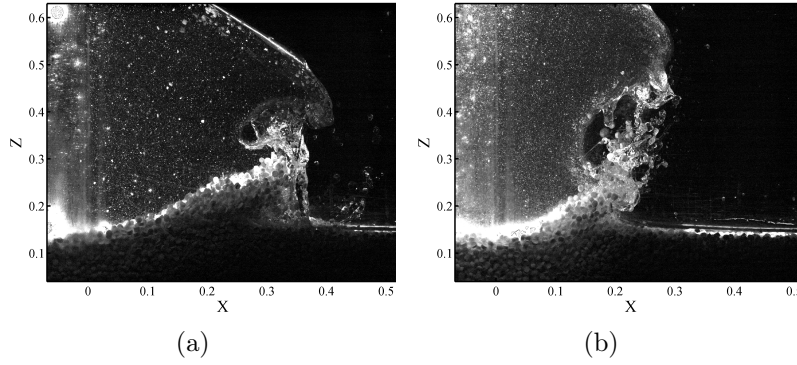


Figure C.3: Comparison between two runs of the dam-break flow over a sediment bed for $T_r + \Delta T$. The two images should be coincident if the gate movement was the same in each run.

For later times, the gate movement has little effect. The velocity profiles at $X = 0$ and $X = 0.5$ at $T = 2.6$ are depicted in Figures C.4 to C.7. For the water layer ($Z > 0.05$) the following can be observed:

1. For the horizontal component U :
 - (a) From Figure C.4, at $X = 0$, U is practically coincident in sections $Y = -0.25$ and $Y = 0.25$.
 - (b) At the centreline ($Y = 0$) the velocity, U , is about 5% higher than in the two other parallel sections (Figure C.4).
 - (c) For $X = 0.5$, U is practically coincident (Figure C.6).

2. For the vertical component W :

- (a) From Figure C.5, at $X = 0$, W is practically coincident in all the measured Y -sections .
- (b) For $X = 0.5$, W has the same behaviour but higher differences are now measured (about 60%).

For the sediment layer ($Z < 0.05$) the following conclusions can be stated:

1. For the horizontal component U :

- (a) From Figure C.4, at $X = 0$, U is practically coincident in the measured Y -sections.
- (b) At $X = 0.5$, U is practically coincident for the measurements made in $Y = -0.25$ and $Y = 0.25$. A higher velocity (about 10%) is measured in $Y = 0$ for $-0.1 < Z < -0.025$.

2. For the vertical component W :

- (a) From Figure C.5, at $X = 0$, W is practically coincident in all the measured Y -sections .
- (b) However, for $X = 0.5$, W has the same behaviour in sections $Y = -0.25$ and $Y = 0.25$ (Figure C.7), but significant differences can be observed for $Y = 0$ as depicted in the same Figure.

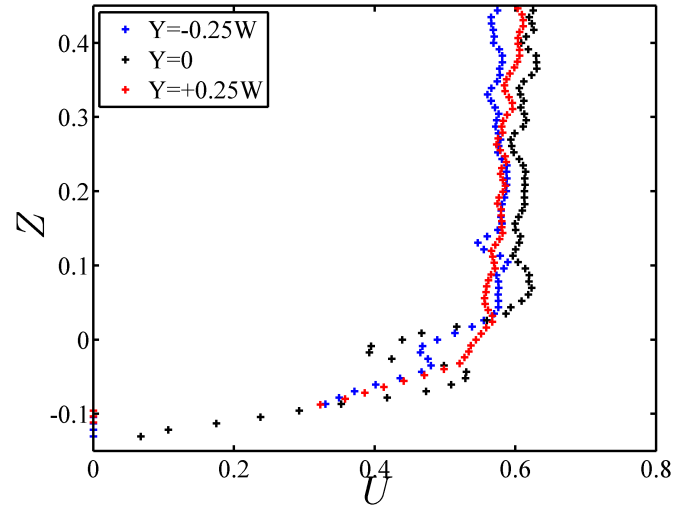


Figure C.4: Profiles of U measured at $X = 0$ in different Y -sections: -0.25, 0 and 0.25. ($T = 2.6$).

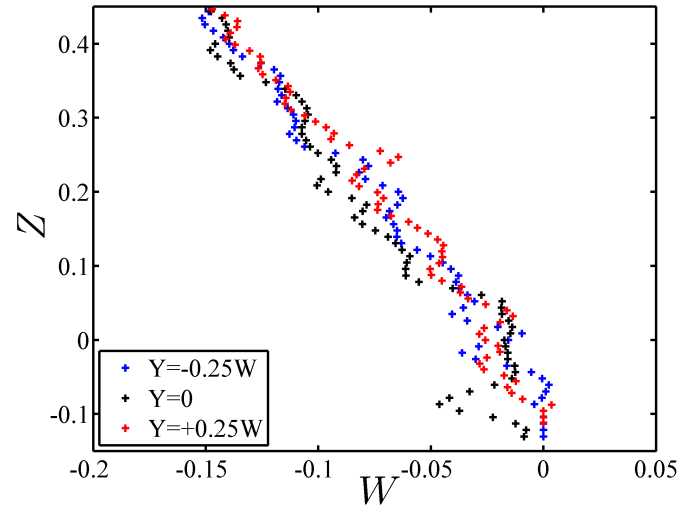


Figure C.5: Profiles of W measured at $X = 0$ in different Y -sections: -0.25, 0 and 0.25. ($T = 2.6$).

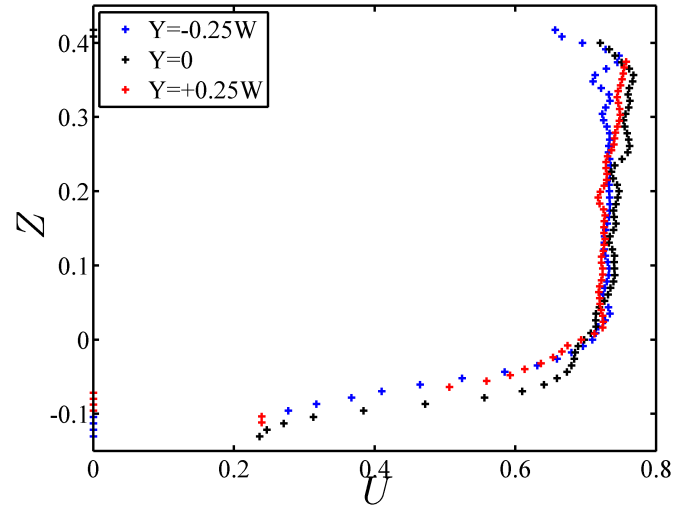


Figure C.6: Profiles of U measured at $X = 0.5$ in different Y -sections: -0.25, 0 and 0.25. ($T = 2.6$).

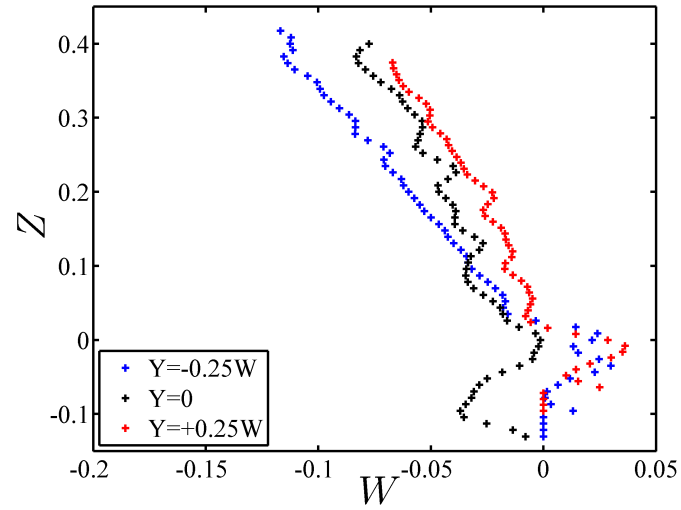


Figure C.7: Profiles of W measured at $X = 0.5$ in different Y -sections: -0.25, 0 and 0.25. ($T = 2.6$).

Appendix D

Tukey 53H Despiking Method

Despiking an electrical signal is an important problem in electrical engineering (Goring and Nikora, 2007). In this annex the method known as Tukey 53H (Tukey, 1977) is presented.

The starting point of this method is considering that the median is a robust estimator of the mean in order to generate a smooth sequence that can be subtracted from the original signal. The Tukey 53H despiking method can be divided in the following steps (Otnes and Enochson, 1978; Goring and Nikora, 2007):

1. From the raw signal, u , determine the 5-value median from u_{i-2} to u_{i+2} :

$$\bar{u}_i^5 = \text{median}(u_{i-2}, \dots, u_{i+2}) \quad (\text{D.1})$$

2. Determine a time series from the median of three data points from sequence $\bar{u}_i^5$, i.e., u_{i-1}^5 to u_{i+1}^5 :

$$\bar{s}_i^3 = \text{median}(u_{i-1}^5, \dots, u_{i+1}^5) \quad (\text{D.2})$$

3. Determine the Hanning smoothing filter given by:

$$u_i^H = \frac{1}{4} \left(s_{i-1}^{(3)} + 2s_i^{(3)} + s_{i+1}^{(3)} \right) \quad (\text{D.3})$$

4. Obtain the time series:

$$\Delta_i = |u_i - u_i^H| \quad (\text{D.4})$$

5. the point is rejected if the following condition is respected:

$$\Delta_i > \kappa\sigma \quad (\text{D.5})$$

where σ is the standard-deviation of u_i and κ a threshold value. for the present case, $\kappa = 1.3$.

6. Replace the spike.

After the spikes' identification a replacement criterion is needed. Several criteria exist according to Goring and Nikora (2007), namely:

1. extrapolating from the preceding point;
2. extrapolating from the two preceding point;
3. the overall mean of the signal;
4. the interpolation between the extremities of the spike;
5. etc.

Here the criterion used was to consider a 4-point average of the points around the identified spike:

$$u_{i,r} = \frac{1}{4} (u_{i-2} + u_{i-1} + u_{i+1} + u_{i+2}) \quad (\text{D.6})$$

For points A, B, C and D of Figure D.1 the *raw* values and the filtered values of the horizontal velocity component are depicted in Figures D.2 to D.5.

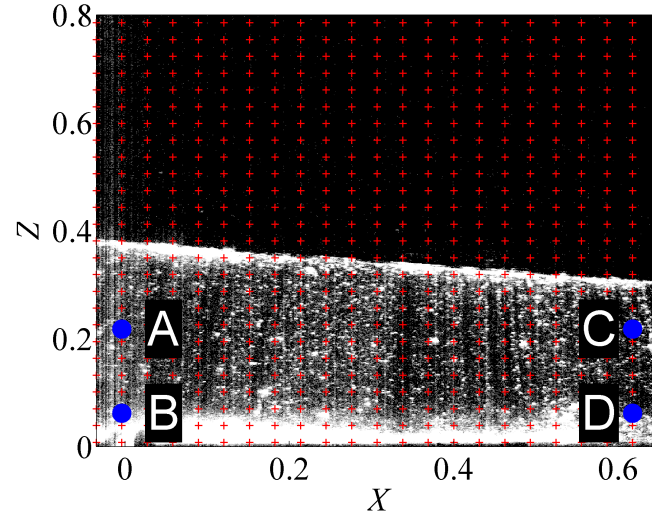


Figure D.1: Localisation of the four chosen points, A, B, C and D. Flow image taken at $T = 5.5$. The gate is located at $X = 0$ and the flow region was highlighted by means of enhancing the image contrast.

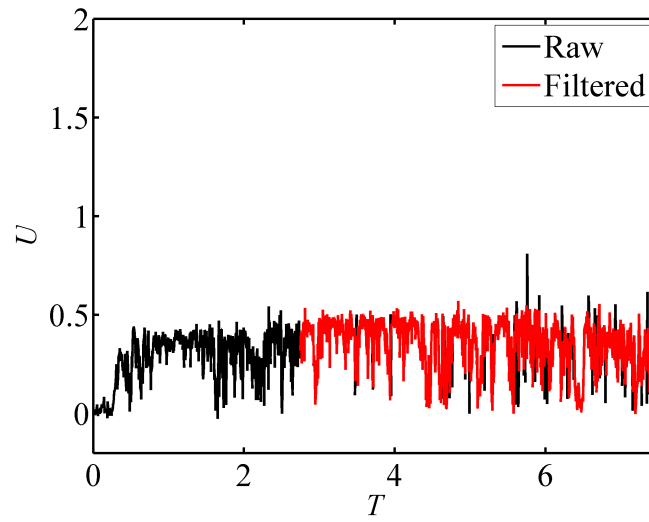


Figure D.2: Application of the Tukey 53H despiking method to the time-series of point A (run 01).

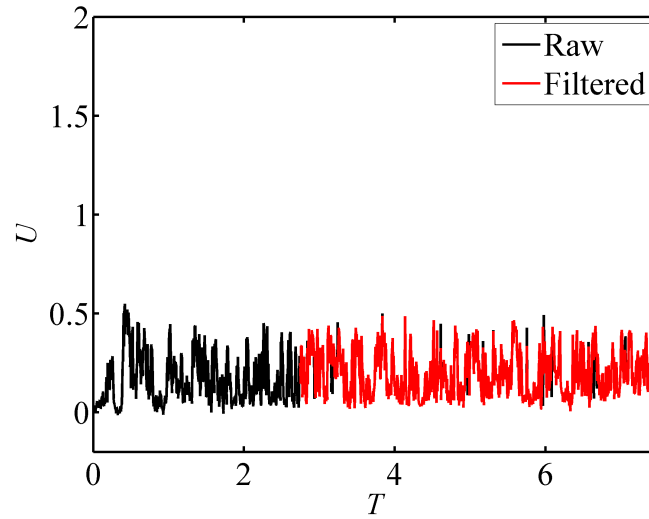


Figure D.3: Application of the Tukey 53H despiking method to the time-series of point B (run 01).

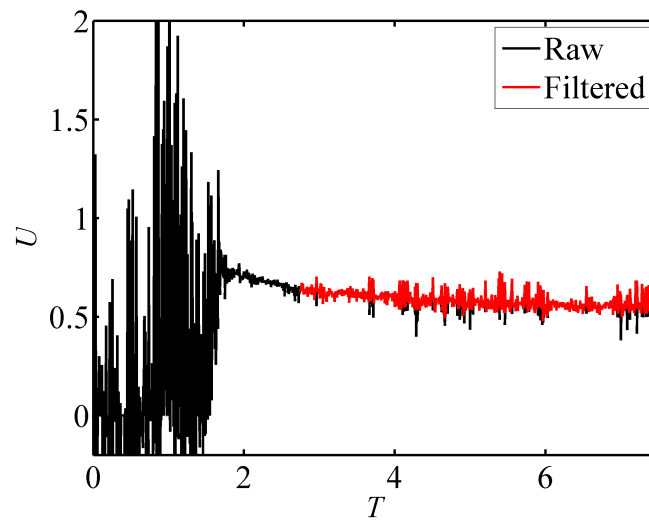


Figure D.4: Application of the Tukey 53H despiking method to the time-series of point C (run 01).

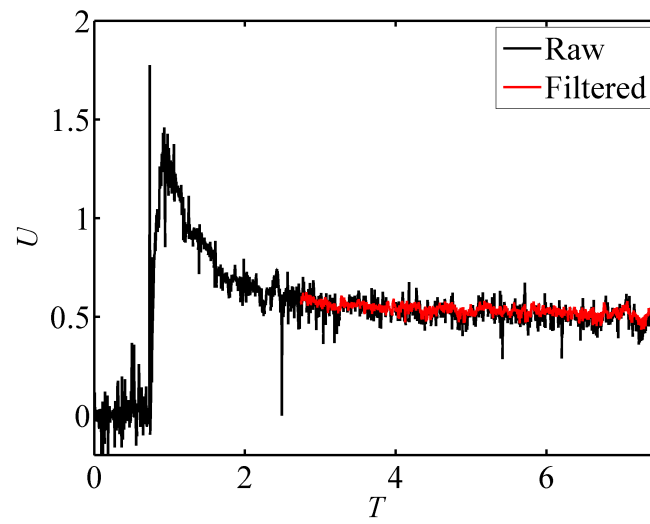


Figure D.5: Application of the Tukey 53H despiking method to the time-series of point D (run 01).

Appendix E

Dam-break Flow Analysis for $X = -2$

For the times $T = 10$, $T = 12$ and $T = 14$ The boundary-layer at $X = -2$ is here characterized in terms of boundary layer height, δ , displacement thickness, δ^* and momentum thickness. The corresponding definitions are briefly presented here. For a more comprehensive analysis of the boundary-layer the reader is invited to read the following references: Schlichting (1966), Okiishi (1998), Novais-Barbosa (1985) for example.

A possible definition for the boundary-layer thickness is the height for which the velocity is 99% of the non-disturbed flow, U_o :

$$u(\delta) = 0.99U_o \quad (\text{E.1})$$

this definition is somewhat arbitrary, since it could have been chose 0.95, 0.90, etc. A physically more meaningful definition is the displacement thickness.

The displacement thickness is defined as the distance by which a surface would have to move in the direction parallel to its normal vector towards the reference plane in an inviscid fluid stream of velocity U_o to give the same flow rate as occurs between the surface and the reference plane in a real fluid (Schlichting, 1966). For an incompressible flow it is

given by:

$$\delta^* = \int_0^\infty \left(1 - \frac{u(z)}{U_0}\right) dz \quad (\text{E.2})$$

Another useful parameter for the characterization of the boundary-layer is the momentum thickness. The momentum thickness is defined by the following relation:

$$\Theta = \int_0^\infty \left(1 - \frac{u(z)}{U_0}\right) \frac{u(z)}{U_0} dz \quad (\text{E.3})$$

Similarly to the displacement thickness, the momentum thickness can be seen as the distance by which a surface would have to move in the direction parallel to its normal vector towards the reference plane in an inviscid fluid stream of velocity U_o to give the same total momentum (Schlichting, 1966).

An example is here given for $T = 14$. The determination of δ , δ^* and Θ is depicted in Figures E.1-E.3.

The shape-factor is defined as:

$$H = \frac{\delta^*}{\Theta} \quad (\text{E.4})$$

For a laminar (Blasius) boundary layer one has $H = 2.59$ Schlichting (1966). For turbulent boundary layers, typically, $H = 1.3 - 1.4$ (White, 1994).

With the thickness δ^* and Θ it is possible to determine the associated local Reynolds numbers:

$$\text{Re}_{\delta^*} = \frac{U_o \delta^*}{\nu} \quad (\text{E.5})$$

and

$$\text{Re}_{\Theta} = \frac{U_o \Theta}{\nu} \quad (\text{E.6})$$

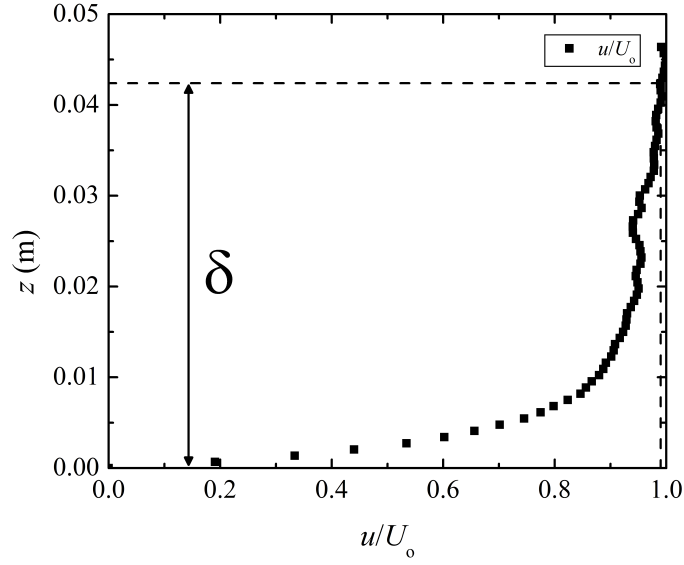


Figure E.1: Boundary-layer height, δ : $u(\delta) = 0.99U_o$, indicated by the dashed lines.

According to Winkelmanns and Bricteux (2008) the laminar boundary-layer with constant U_o becomes unstable at $Re_{\delta^*} \approx 400$. Table E.1 resumes the different boundary-layer parameters.

Table E.1: Obtained parameters for the boundary layer at $T = 10$, $T = 12$ and $T = 14$ at $X = -2$.

T	δ (m)	δ^* (m)	Θ (m)	H	Re_{δ^*}	Re_{Θ}	U_o (m/s)
10	0.063	0.00757	0.00475	1.59	5576	3499	0.735
12	0.058	0.00620	0.00386	1.61	4653	2897	0.758
14	0.0424	0.0024	0.00326	1.61	3917	2437	0.755

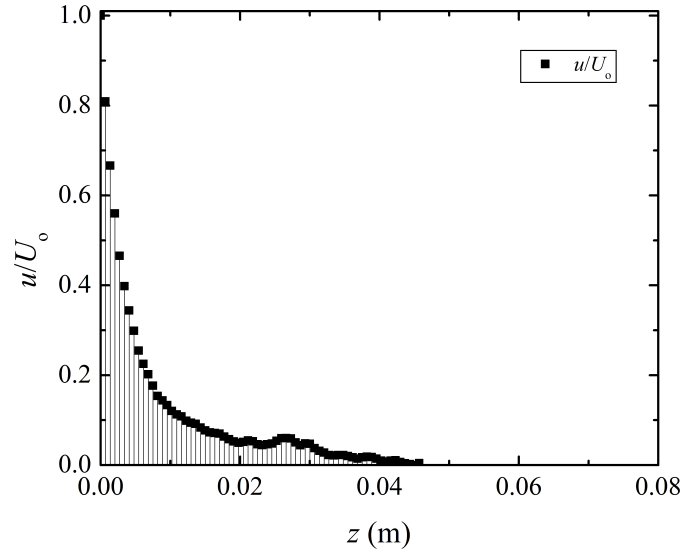


Figure E.2: Determination of the displacement thickness height, δ^* , for $T = 14$.

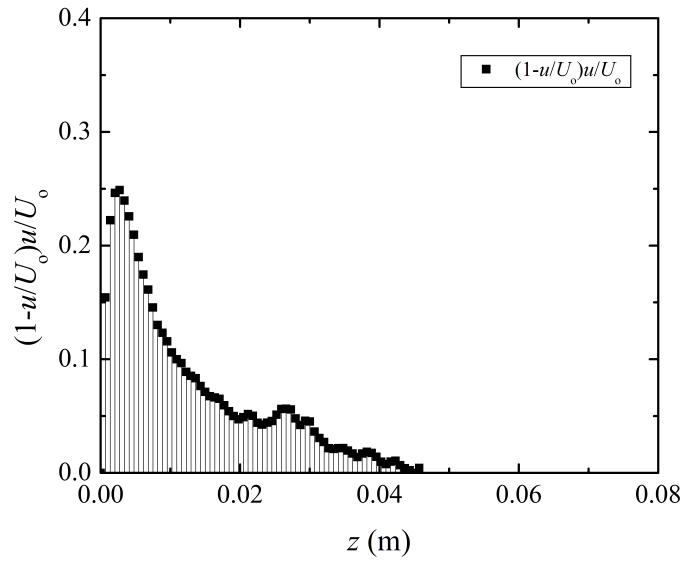


Figure E.3: Determination of the momentum thickness height, Θ , for $T = 14$.

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Bibliography

- Abdolmaleki, K., K. Thiagarajan, and M. Morris-Thomas (2004). Simulation of the dam break problem and impact flows using a navier-stokes solver. In *Proceedings of the 15th Australian Fluid Mechanics Conference*.
- Acharya, M., J. Bornstein, M. Escudier, and V. Vokurka (1985). Development of a floating element for the measurement of surface shear stress. *AIAA* 23, 410–415.
- Adrian, R. J. (2005). Twenty years of particle image velocimetry. *Experiments in Fluids* 39, 159–169.
- Adrian, R. J. and J. Westerweel (2011). *Particle Image Velocimetry*. Cambridge University Press.
- Aguirre-Pe, J., F. Plachco, and S. Quisca (1995). Tests and numerical one-dimensional modelling of a high viscosity fluid dam-break wave. *Journal of Hydraulic Research* 33, 1.
- Ahuja, N. (1982). Dot pattern processing using Voronoï neighborhoods. *IEEE Trans Pattern Anal Mach Intel* 4, 336–343.
- Alcrudo, F. and J. Mulet (2007). Description of the Tous Dam break case study (Spain). *Journal of Hydraulic Research* 45-1 Special issue, 45–57.
- Aleixo, R. (2011). Time resolved PIV analysis of the initial stages of the dam break. In *Proceedings of the 34th IAHR Conference, Brisbane, Australia*.

- Aleixo, R. (2012). A list of dam-break events - not published. Personal note.
- Aleixo, R., S. Soares-Frazão, and Y. Zech (2011). Velocity-field measurements in a dam-break flow using a ptv voronoï imaging technique. *Experiments in Fluids* 50:6, 1633–1649.
- Aleixo, R., S. Soares-Frazão, and Y. Zech (2010). Transient flow analysis by imaging methods - Voronoï particle tracking velocimetry applied to the dam-break flow. In *Proceedings of the 5th European Conference on Computational Fluid Dynamics, ECCOMAS 2010, June 14th-17th, Lisbon Portugal*.
- Aleixo, R., B. Spinewine, S. Soares-Frazão, and Y. Zech (2009). Non-intrusive measurements of water surface and velocity profiles in a dam flow. In *Proceedings of the 33rd IAHR Congress, Vancouver, Canada.*, pp. 5896–5903.
- Aleixo, R., B. Spinewine, S. Soares-Frazão, and Y. Zech (2010). Velocity profiles in dam-break flows: Water and sediment layers. In *Proceedings of the RiverFlow 2010 conference, Braunschweig, Germany*.
- Ancey, C. and S. Crochard (2009). The dam-break problem for herschel-bulkley viscoplastic fluids down steep flumes. *Journal of Non-Newtonian Fluid Mechanics* 158, 18–35.
- Aureli, F., A. Maranzoni, P. Mignosa, and C. Ziveri (2008). Dam-break flows: Acquisition of experimental data through an imaging technique and 2d numerical modeling. *Journal of Hydraulic Engineering* 134:8, 1089–1101.
- Aureli, F., A. Maranzoni, P. Mignosa, and C. Ziveri (2011). An image processing technique for measuring free surface of dam-break flows. *Experiments in Fluids* 50:3, 665–675.
- Bailly, D. and A. V. Reybroeck (2003). Conception et réalisation d un canal pour la modélisation expérimentale de la propagation sur lit

- mobile de l onde de rupture d un barrage. Master's thesis, Université catholique de Louvain. Belgium.
- Barenblatt, G. (1993). Scaling laws for fully developed turbulent shear flows. part 1. basic hypotheses and analysis. *Journal of Fluid Mechanics* 248, 513–520.
- Barenblatt, G. I. and A. Chorin (1996). Small viscosity asymptotics for the inertial range of local structure and for the wall region of wall-bounded turbulent shear flow. *Proc. Natl. Acad. Sci. USA* 93, 6749–6752.
- Bastiaans, R., G. van der Plas, and R. Kieft (2002). The performance of a new PTV algorithm applied in super-resolution PIV. *Experiments in Fluids* 32, 346–356.
- Bates, C. and T. Hughes (1976). Real-time statistical ldv system for the study of a high Reynolds number, low turbulence intensity flow. *J. Physics E Scientific Instruments* 9, 955–958.
- Bates, C. and T. Hughes (1977). The effect of both sample size and sampling rate on the statistical fluid flow parameters in a high Reynolds number, low turbulence intensity flow. In *5th Biennial Symposium on Turbulence, University of Missouri-Rolla*,.
- Becker, A. and D. Kaiser (2008). Ecoulement concecutif a une rupture de barrage sur lit mobile dans un canal presentant un elargissement brusque: mesures de la surface libre et de la topographie finale. Master's thesis, Ecole Polytechnique de Louvain.
- Begnudelli, L. and B. Sanders (2007). Simulation of the st. francis dam-break flood. *Journal of Hydraulic Engineering* 133:11, 1200–1212.
- Bell, S. (2001). A begginer's guide to uncertainty of measurement. Technical report, National Physical Laboratory, Teddington, Middlesex, United Kingdom, TW11 0LW.

- Bell, S., R. Elliot, and M. H. Chaudry (1992). Experimental results of two-dimensional dam-break flows. *Journal of Hydraulic Research* 30 (2), 225–252.
- Bellos, C., J. Soulis, and J. Sakkas (1992). Experimental results of two-dimensional dam-break induced flow. *International Journal of Hydraulic Research* 30:1, 165–187.
- Bendat, J. and A. Piersol (2011). *Random Data: Analysis and Measurement Procedures*. Wiley.
- Bukreev, V. and A. Gusev (2005). Initial stage of the generation of dam-break waves. *Doklady Physics* 50:4, 200–203.
- Canelas, R., R. Aleixo, and R. Ferreira (2012). SPH-based numerical simulation of the velocity field in a dam-break flow. In *Actas da 4 Conferência nacional em Mecânica de Fluidos, Termodinâmica e Energia, Lisboa, 2012*.
- Capart, H. and L. Fracarollo (2011). Transport layer structure in intense bed-load. *Geophysical Research Letters* 38, doi:10.1029/2011GL049408.
- Capart, H. and D. Young (2002). Two-layer shallow water computations of torrential geomorphic flows. In D. Bousmar and Y. Zech (Eds.), *Proceedings of the River Flow Conference*.
- Capart, H., D. Young, and Y. Zech (2002). Voronoï imaging methods for the measurement of granular flows. *Experiments in Fluids* 32, 121–135.
- Carroll, L. (1865). *Alice's Adventures in Wonderland (2007)*. Vintage Books.
- Carvalho, E., R. Maia, and M. Proença (2010). Shear stress measurements over smooth and rough channel beds. In *Proceedings of the RiverFlow 2010 Conference*.

- Chanson, H. (2004). *Environmental hydraulics of open channel flows*. Butterworth-Heinemann Ltd.
- Chanson, H. (2005a). Analytical solution of dam break wave with flow resistance. application to tsunami surges. In I. S. B.H. Jun, S.I. Lee and G. Choi (Eds.), *Proceedings of the 31th Biennial IAHR Congress, Seoul, Korea*, Korea.
- Chanson, H. (2005b). Applications of the saint-venant equations and method of characteristics to the dam break wave problem - report ch55/05. Technical report, University of Queensland.
- Chanson, H. (2006a). Analytical solutions of laminar and turbulent dam break wave. In R. Ferreira, E. Alves, J. Leal, and A. Cardoso (Eds.), *Proceedings of the RiverFlow 2006 Conference, Lisbon, Portugal*.
- Chanson, H. (2006b). A simple solution of the laminar dam break wave. *Journal of Applied Fluid Mechanics* 1:1, 1–10.
- Chanson, H. and S. Aoki (2001). Dam break wave with energy dissipation: Two case studies. In *Proceedings of the 29th IAHR Congress, Beijing, China*.
- Chanson, H., S. Jarny, and P. Coussot (2006). Dam break wave of thixotropic fluid. *Journal of Hydraulic Engineering* 132:3, 280–293.
- Chauvenet, W. (1863). *A Manual of Spherical and Practical Astronomy* V. Dover Publications Inc, 1960.
- Clauser, F. (1956). The turbulent boundary layer. *Advances in Applied Mechanics* 4, 1–51.
- Cochard, S. and C. Ancey (2008). Tracking the free surface of time-dependent flows: Image processing for the dam-break problem. *Experiments in Fluids* 44, 59–71.
- Coles, D. (1956). The law of the wake in the turbulent boundary layer. *Journal of Fluid Mechanics* 1, 191–226.

- Cooley, J. W. and J. Tukey (1965). An algorithm for the machine calculation of fourier series. *Mathematical Computing* 19, 297–301.
- Dambusters (2007, June). Website. Acessed: 8th Februrary 2012.
- Delvaux, C. (2011). Mesure de la concentration en sédiments dans la couche de transport en régime transitoire severe. Master’s thesis, Ecole Polytechnique de Louvain - Universite catholique de Louvain.
- Dhawan, S. (1952). Direct measurements of local skin-friction. Technical report, National Advisory Comm. for Aeronautics.
- Doppler, C. (1843). Ueber das farbige licht der doppelsterne und einiger anderer gestirne des himmels. *Abhandlungen der k. bohm. Gesellschaft der Wissenschaften* 2, 465–482.
- Dressler, R. (1952). Hydraulic resistance effect upon the dam-break functions. *Journal of Research, Ntl. Bureau of Standards* 49-3, 217–225.
- Dressler, R. (1954). Comparison of theories and experiments for the hydraulic dam-break wave. *Publication 38 Association Internationale d’Hydrologie* 38, 319–328.
- Durst, F., H. Kikura, I. Lekakis, J. Jovanovic, and Q. Ye (1996). Wall shear stress determination from near-wall mean velocity data in turbulent pipe and channel flows. *Experiments in Fluids* 20, 417 – 428.
- Durst, F., A. Melling, and J. Whitelaw (1981). *Principles and Practice of Laser-Doppler Anemometry* (2nd ed.). London: Academic Press.
- Durst, F., R. Muller, and J. Jovanovic (1988). Determination of the measuring position in laser doppler anemometry. *Experiments in Fluids* 6, 105–110.
- Eaket, J., F. Hicks, , and A. Peterson (2005). Use of stereoscopy for dam-break flow measurement. *Journal of Hydraulic Engineering* 131:1, 24–29.

- Eguiazaroff, I. (1935). Regulation of the water level in the reaches of canalized rivers and regulation of the flow below the last cock dam according to wheter the water power is or is not used. In *Report No 8, Inland Navigation, 2nd question, sec. 1, 16th International Congress of Navigation, Brussels*.
- Elliot, R. and M. Chaudry (1992). A wave propagation model for two-dimensional dam-break flows. *Journal of Hydraulic Research* 30:4, 467–483.
- Elsinga, G., F. Scarano, B. Wieneke, and B. van Oudheusden (2006). Tomographic particle image velocimetry. *Experiments in Fluids* 41, 933–947.
- Estrade, J. (1967). Contribution à l'étude de la supression d'un barrage. *Direction des etudes er Recherches Chatou Serie A- Volume 1*, 3–128.
- Ettema, R., P. Roberts, R. Arndt, and T. Wahl (2000). *Hydraulic Modeling - Concepts and Practice*. American Society of Civil Engineers.
- Fagherazzi, S., P. Rasetarinera, M. Hussaini, and D. Furbish (2004). Numerical solution of the dam-break problem with discontinuous galerkin method. *Journal of Hydraulic Engineering* 130:6, 532–539.
- Fennema, R. and M. Chaudry (1987). Simulation of one-dimensional dam-break flows. *Journal of Hydraulic Research* 25:1, 41–51.
- Fracarollo, L. and A. Armanini (1999). A semi-analytical solution for the dam-break problem over a movable bed. In *Proceedings of the 28th IAHR Conference, Graz, Austria*.
- Fracarollo, L. and H. Capart (2002). Riemann wave description of erosional dam-break flows. *Journal of Fluid Mechanics* 461, 183–228.
- Fracarollo, L. and E. Toro (1995). Experimental and numerical assessment of the shallow water model for two-dimensional dam-break type problems. *Journal of Hydraulic Research* 33:6, 843–864.

- Frost, R. (1921). *Mountain Interval 2nd Edition*. Henry Holt.
- Galilei, G. (1632). *Dialogo sopra i due massimi sistemi del mondo tolemaico e copernicano*. Gradiva Tradução de Mario Brito; anotacao e prefacio de Jose Trindade Santos, (1979).
- Glisson, T. (1985). *Introduction to system analysis*. McGraw-Hill.
- Gonzalez, R. C. and R. E. Woods (1993). *Digital Image Processing*. Addison-Wesley.
- Goring, D. and V. I. Nikora (2007). Despiking acoustic doppler velocimeter data. *Journal of Hydraulic Engineering* 128, 117–126.
- Graf, W. H. and M. Altinakar (2001). *Fluvial Hydraulics*. New York: John Wiley and Sons.
- Halliday, D., R. Resnick, and J. Walker (2001). *Fundamentals of Physics*. New York: John Wiley & Sons.
- Hanratty, T. and J. Campbell (1983). *Measurement of Wall Shear Stress*. R. J. Goldstein, Hemisphere, Washington DC.
- Hart, D. P. (1998). The elimination of correlation errors in PIV processing. In *Proceedings of the 9th International Symposium on Applications of Laser Techniques to Fluid Mechanics, Lisbon, Portugal*.
- Harten, A., P. Lax, and B. V. Leer (1983). On up-stream differencing and godunov-type schemes for hyperbolic conservation laws. *SIAM Review* 25, 35–61.
- Hassan, Y. and R. Canaan (1991). Full field bubbly flow velocity measurements using a multiframe particle tracking technique. *Experiments in Fluids* 12, 49–60.
- Hecht, E. (2001). *Optics*. Addison-Wesley.
- Henderson, F. (1966). *Open Channel Flow*. Mcmillan.

- Hirt, C. W. and B. D. Nicols (1981). Volume of fluid method (vof) for the dynamics of free boundaries. *Journal of Computational Physics* 39-1, 201–225.
- Hitching, E. and A. Lewis (1999). Sampling techniques for accurate measurement of Reynolds stresses using laser doppler anemometry. *Flow Measurement and Instrumentation* 10, 241–247.
- Hogg, A. and D. Pritchard (2004). The effects of hydraulic resistance on dam-break and other shallow inertial flows. *Journal of Fluid Mechanics* 501, 179–212.
- Holman, J. (2007). *Experimental Methods for Engineers*. Tata McGraw-Hill.
- Hunt, B. (1982). Asymptotic solution for dam-break problem. *Journal of the Hydraulics Division* 108:1, 115–126.
- Hunt, B. (1984). Perturbation solution for dam break floods. *Journal of Hydraulic Engineering* 110:8, 115–126.
- Jähne, B. (1995). *Digital image processing*. Springer. Berlin. Heidelberg.
- Kahler, C. J., S. Scharnowski, and C. Cierpka (2012). On the uncertainty of digital PIV and PTV near walls. *Experiments in Fluids* 52:6, 1641:1656.
- Keane, R. and R. Adrian (1990). Optimization of particle image velocimeters, i: double pulsed systems. *Measurements Science Technology* 1, 1202–1215.
- Keane, R. and R. Adrian (1992). Theory of cross-correlation analysis of PIV images. *Applied Science Research* 49, 191–215.
- Kinsler, L., A. R. Frey, A. B. Coppens, and J. V. Sanders (2000). *Fundamentals of Acoustics*. John Wiley and Sons.
- Korobkin, A. and O. Yilmaz (2009). The initial stage of a dam-break flow. *Journal of Engineering Mathematics* 63, 293–308.

- Kundu, P. K., I. M. Cohen, and D. R. Dowling (2011). *Fluid Mechanics*. San Diego: Academic Press.
- Lamarre, E. and W. Melville (1991). Air entrainment and dissipation in breaking waves. *Nature* 351, 469:472.
- Landau, L. and E. Lifshitz (1987). *Course of Theoretical Physics - Volume 6 Fluid Mechanics 2nd Edition*. Butterworth-Heinemann Ltd.
- Lauber, G. and W. Hager (1998a). Experiments to dambreak wave: horizontal channel. *Journal of Hydraulic Research* 36, 291–307.
- Lauber, G. and W. Hager (1998b). Experiments to dambreak wave: Sloping channel. *Journal of Hydraulic Research* 36:5, 761–773.
- Lavision (2007). Flow master product manual 1105011-4. Technical report, Lavision GmbH.
- Leal, J., R. Ferreira, and A. Cardoso (2001). Dam-break waves over movable bed channels experimental study. In *Proceedings of the 29th IAHR Conference, Beijing, China*.
- Leal, J., R. Ferreira, and A. Cardoso (2002). Dam-break waves on movable bed. In *Proceedings of the Riverflow 2002 Conference, Louvain-la-Neuve, Belgium*.
- Leal, J., R. Ferreira, and A. Cardoso (2006). Dam-break wave-front celerity. *Journal of Hydraulic Engineering* 132:1, 69–76.
- Lemmin, U. and T. Rolland (1997). Acoustic velocity profiler for laboratory and field studies. *Journal of Hydraulic Engineering* 123:12, 1089–1098.
- Lencastre, A. (1996). *Hidráulica Geral*. Author's Edition.
- Lide, D. (1992). *CRC Handbook of Chemistry and Physics*. CRC Press Inc.

- Liem, R. and J. Kongeter (1999). Application of high-speed digital image processing to experiments on dam break waves. In *Proceedings of the CADAM Meeting, Zaragossa, Spain*, pp. 399-411.
- Lin, P. and P.-F. Liu (1998). A numerical study of breaking waves in surf zone. *Journal of Fluid Mechanics* 359, 239–264.
- Lofdahl, L. and M. G. el Hal (1999). MEMS-based pressure and shear-stress sensors for turbulent flow. *Measurements Science* 10, 665–686.
- Lumley, J. (1979). *Stochastic Tools in Turbulence*. Academic Press.
- Mahmood, K. and V. Yevjevich (1975). *Unsteady flow in Open Channels - Volume I*. Water Resources Publications.
- Mangeney, A., P. Heinrich, and R. Roche (2000). Analytical solution for testing debris avalanche numerical models. *Pure and Applied Geophysics* 157, 1081–1096.
- Maron, B. D. I. (1973). *Elements de Calcul Numerique*. Mir.
- Martin, H. (1990). *Plotzlich veranderliche instationare Strömungen in offenen Gerinnen (in German)*. G. Bollrich Verlag für Bauwesen Berlin.
- Matson, G. and A. J. Hogg (2007). Two-dimensional dam break flows of herschel-bulkley fluids: The approach to the arrested state. *Journal of Non-Newtonian Fluid Mechanics* 142, 79–94.
- Melling, A. (1997). Tracer particles and seeding for particle image velocimetry. *Measurements Science and Technology* 8:12, 1406–1416.
- Melville, H. (1851). *Moby Dick*. Harper and Brothers.
- Met-Flow (2002). Uvp monitor model uvp-duo with software version 3. Technical report, Met-Flow SA.
- Meyer-Peter, E. and R. Muller (1948). Formulas for bed-load transport. In *Proceedings of the 2nd meeting of the International Association for Hydraulic Structures Research*.

- Meynart, R. (1980). Equal velocity fringes in a rayleigh-bernard flow by a speckle method. *Applied Optics* 19, 1385–1386.
- Meynart, R. (1982). Digital image processing for speckle flow velocimetry. *Res Sci Instrum* 53, 110–111.
- Meynart, R. (1983). *Mesure des champs de vitesse d'écoulements fluides par analyse de suites d'images obtenus par diffusion d'un feuillet lumineux*. Ph. D. thesis, Université Libre de Bruxelles.
- Millikan, C. M. (1938). A critical discussion of turbulent flows in channels and circular tubes. In *Proc. 5th Intl. Congress of Appl. Mech*, pp. 386.
- Modaress, D. and D. Wilson (2006). Micro-optical sensors for boundary layer flow studies (keynote paper). In *2006 ASME Joint U.S. - European Fluids Engineering Summer Meeting*.
- Nezu, I. and H. Nakagawa (1993). *Turbulence in Open-Channel Flows*. Rotterdam: A.A. Balkema.
- Novais-Barbosa, J. (1985). *Mecânica dos Fluidos e Hidráulica Geral* (1^o ed.), Volume 1. Porto: Porto Editora.
- Nowak, M. (2002). Wall shear stress measurement in a turbulent pipe flow using ultrasound doppler velocimetry. *Experiments in Fluids* 33, 249–255.
- Nsom, B. (2002). Horizontal viscous dam-break flow: Experiments and theory. *Journal of Hydraulic Engineering* 128:5, 543–546.
- Oertel, M. and D. Bung (2012). Initial stage of two-dimensional dam-break waves: laboratory versus vof. *Journal of Hydraulic Research* 50:1, 2012.
- Ohmi, K. and H.-Y. Li (2000). Particle-tracking velocimetry with new algorithms. *Measurements Science Technology* 11, 603–616.
- Okabe, A., B. Boots, and K. Sugihara (1992). *Spatial tessellations: concepts and applications of Voronoï diagrams*. Willey - New York.

- Okiishi, B. R. M. D. F. Y. T. H. (1998). *Fundamentals os Fluid Mechanics*. New York: John Wiley.
- Otnes, R. and L. Enochson (1978). *Applied Time-Series Analysis Vol 1*. Wiley and Sons.
- Oweis, G. F., E. S. Winkel, J. Cutbrith, S. L. Ceccio, M. Perlin, and D. Dowling (2010). The mean velocity profile of a smooth-flat-plate turbulent boundary layer at high Reynolds number. *Journa of Fluid Mechanics* 665, 357–381.
- Papantoniou, D. and T. Dracos (1989). Analyzing 3-d turbulent motions in open channel flow by use of stereoscopy and particle tracking. *Adv Turb* 2, 287–285.
- Peregrine, D. and S. Williams (2001). Swash overtopping a truncated plane beach. *Journal of Fluid Mechanics* 440, 391–399.
- Pilotti, M., A. Maranzoni, M. Tomirotti, and G. Valerio (2011). 1923 gleno dam break: Case study and numerical modeling. *Journal of Hydraulic Engineering* 137:4, 480–492.
- Pohle, F. (1952). Motion of water due to breaking of a dam and related problems, in gravity waves. *Ntl. Bureau of Standards Circular* 521, 47–53.
- Pope, S. (2000). *Turbulent Flows*. Cambridge University Press.
- Prandtl, L. (1935). *The mechanics of viscous fluid. In aerodynamics theory vol. 3*. Julius Springer W.F. Durand (ed).
- Pritchard, D. and A. Hogg (2002). On sediment transport under dam-break flow. *Journal of Fluid Mechanics* 473, 265–274.
- Raffel, M., C. Willert, S. Werely, and J. Kompenhans (2007). *Particle Image Velocimetry - A practical guide*. Springer-Verlag.
- Raudkivi, A. J. (1998). *Loose Boundary Hydraulics*. Rotterdam: A. A. Balkema.

- Reynolds, O. (1895). On the dynamical theory of incompressible viscous fluids and the determination of the criterion. *Philos. Trans. R. Soc. London, Ser. A* 186, 123.
- Ritter, A. (1892). Die fortpflanzung der wasserwellen. *Zeitschrift Verein Deutscher Ingenieure* 36, 947–954.
- Rodgers, R. and O. Hammerstein (1959, November). The sound of music. Broadway Musical.
- Roe, P. (1981). Approximate riemann solvers, parameter vectors and difference schemes. *Journal of Computational Physics* 43:2, 357–372.
- Rogers, J. D. (2007). Website. Accessed: 8th February 2012.
- Russ, J. C. (2011). *The Image Processing Handbook 6th edition*. CRC Press Inc.
- Safety, D. (2012). Dam-safety. website. Accessed 19th November 2012.
- Sanborn, V. (Fort Collins, 1972). *Resistance Temperature Transducers*. Metrology.
- Savary, C. (2007). *Transcritical transient flow over mobile bed. Two-layer shallow-water model*. Ph. D. thesis, Université Catholique de Louvain, Belgium.
- Savary, C. and Y. Zech (2007). Boundary conditions in a two-layer geomorphological model. application to a hydraulic jump over a mobile bed. *Journal of Hydraulic Research* 45:3, 316–332.
- Scarano, F., L. David, M. Bsibsi, and D. Calluaud (2005). S-PIV comparative assessment: Image dewarping + misalignement correction and pinhole + geometric back projection. *Experiments in Fluids* 39, 257–266.
- Scarano, F. and C. Poelma (2009). Three-dimensional vorticity patterns of cylinder wakes. *Experiments in Fluids* 47, 69–83.

- Scarano, F. and M. Riethmuller (2000). Advances in iterative multigrid PIV image processing. *Experiments in Fluids* 29, S051–S060.
- Scheimpflug, T. (1904). Improved method and apparatus for the systematic alteration or distortion of plane pictures and images by means of lenses and mirrors for photography and for other purposes.
- Schlichting, H. (1966). *Boundary-Layer Theory*. New York: McGraw-Hill.
- Schoklitsch, A. (1917). *Über Dambruchwellen*, pp. 1489–1514. Akademie der Wissenschaften.
- Sédov, L. (1977). *Similitude et dimensions en mécanique*. Moscou: Éditions Mir.
- Shakibaeinia, A. and Y.-C. Jin (2011). A mesh-free particle model for simulation of mobile bed dam-break. *Advances in Water Resources* 34, 794–807.
- Shigematsu, T., P. L.-F. Liu, and K. Oda (2004). Numerical modeling of the initial stages of dam-break waves. *Journal of Hydraulic Research* 42:2, 183–195.
- Soares-Frazão, S. (2007). Experiments of dam-break wave over a triangular bottom sill. *Journal of Hydraulic Research* 45 Extra Issue, 19–26.
- Soares-Frazão, S., N. L. Grelle, B. Spinewine, and Y. Zech (2007). Dam-break induced morphological changes in a channel with uniform sediments: measurements by a laser-sheet imaging technique. *Journal of Hydraulic Research* 45 Extra Issue, 87–95.
- Soares-Frazão, S. and Y. Zech (2008). Dam-break flow through an idealised city. *Journal of Hydraulic Research* 46 (5), 648–658.
- Song, X., F. Yamamoto, M. Iguchi, and Y. Murai (1999). A new tracking algorithm of PIV and removal of spurious vectors using delaunay tessellation. *Experiments in Fluids* 26, 371–380.

- Spedding, G. and E. Rignot (1993). Performance analysis and application of grid interpolation techniques for fluid flows. *Experiments in Fluids* 15, 417–430.
- Spinewine, B. (2005). *Two-layer flow behaviour and the effects of granular dilatancy in dam-break induced sheet flows*. Ph. D. thesis, Université catholique de Louvain, Belgium.
- Spinewine, B., R. Aleixo, and H. Capart (2011a). Intense bedload associated with unsteady dam-break surges. In *Proceedings of the RCEM Conference, Beijing, China, September 6-8*.
- Spinewine, B., R. Aleixo, and H. Capart (2011b). Velocity and concentration profiles within dam-break-induced intense bedload layers. In *Proceedings of the Thesis 2011 conference (in CD), Chatoux, France*.
- Spinewine, B., H. Capart, L. Fraccarollo, and M. Larcher (2011). Laser stripe measurements of near-wall solid fraction in channel flows of liquid-granular mixtures. *Experiments in Fluids* 50:6, 1507–1525.
- Spinewine, B., V. Guinot, S. Soares-Fraza, and Y. Zech (2011). Solution properties and approximate Riemann solvers for two-layer shallow flow models. *Computers and Fluids* 44, 202:220.
- Spinewine, B. and Y. Zech (2007). Small-scale laboratory dam-break waves on movable beds. *Journal of Hydraulic Research* 45 Special Issue, 73–86.
- Stansby, P., A. Chengini, and T. Barnes (1998). The initial stages of the dam break flow. *Journal of Fluid Mechanics* 374, 407–424.
- Stitou, A. and M. Riethmuller (2001). Extension of PIV to super-resolution using PTV. *Measurements Science Technology* 13, 1398–1403.
- Stocksley, M. (2001). *Abdominal Ultrasound*. Cambridge University Press.

- Stoker, J. (1957). *Water Waves: The mathematical theory with applications*. New York: Wiley-Interscience.
- Sveen, J. K. and E. A. Cowen (2004, January). Quantitative imaging techniques and their application to wavy flows. url.
- Swartenbroekx, C., V. Guinot, S. Soares-Fraza, B. Spinewine, and Y. Zech (2010). Modified hll solver for two layer shallow flow simulations. In *Proceedings of the River Flow Conference, Braunschweig, Germany*, Volume 2, pp. 1379–1387.
- Swartenbroekx, C., S. Soares-Fraza, R. Aleixo, and Y. Zech (2011). Dam-break flows over mobile bed: comparison between PIV measurements and a 2-layer shallow water model. In *Proceedings of the River, Coastal and Estuarine Morphodynamics: RCEM 2011, Beijing, China, Tsinghua University Press, Beijing*, pp. 767-783.
- Taishi, T., H. Kikura, and M. Aritomi (2002). Effect of the measurement volume in turbulent pipe flow measurement by the ultrasonic velocity profile method (mean velocity profile and Reynolds stress measurement). *Experiments in Fluids* 32, 188–196.
- Takeda, Y. (1987). Measurement of velocity profile of mercury flow by ultrasound doppler shift method. *Nuclear Technology* 79, 120–124.
- Takeda, Y. (1995). Velocity profile measurement by ultrasonic doppler method. *Experiments in Thermal Fluid Sciences* 10, 444–453.
- Takehara, K., R. Adrian, G. Etoh, and K. Christensen (2000). A kalman tracker for super-resolution PIV. *Experiments in Fluids (suppl) Supplement*, S34–s41.
- Tan, L. W. and V. H. Chu (2009). Lauber and hager’s dam-break wave data for numerical model validation. *Journal of Hydraulic Research* 47:4, 524–528.

- Tanner, L. and L. Blows (1976). A study of the motion of oil films on surfaces in air flow, with application to the measurement of skin friction. *Journal of Physics E*. 9, 194–202.
- Tavoularis, S. (2009). *Measurement in Fluid Mechanics*. Cambridge University Press.
- Tennekes, H. and J. Lumley (1972). *A First Course in Turbulence*. Cambridge: The MIT Press.
- Townsend, A. (1976). *The Structure of Turbulent Flow 2nd Edition*. Cambridge University Press.
- Tritton, J. (1988). *Physical Fluid Dynamics 2nd Edition*. Oxford Science Publications.
- Tropea, C., A. Yarin, and J. Foss (2007). *Handbook of experimental fluid mechanics*. Springer-Verlag.
- Tukey, J. (1977). *Exploratory Data Analysis*. Addison-Wesley.
- USACE (1960). Floods resulting from suddenly breached dams. Technical report, US Army Eng. Wat Exp. Station. Vicksburg. Mississippi.
- USACE (1961). Floods resulting from suddenly breached dams - conditions of high resistance report 2. Technical report, US Army Eng. Wat. Exp. Station. Vicksburg. Mississippi.
- Vacondio, R., B. Rogers, P. Stansby, and P. Mignosa (2012). SPH modeling of shallow flow with open boundaries for practical flood simulation. *Journal of Hydraulic Engineering* 138(6), 530–541.
- Valliani, A., V. Caleffi, and A. Zanni (2002). Case study: Malpasset dam-break simulation using a two-dimensional finite volume method. *Journal of Hydraulic Engineering* 128:5, 460–472.
- Vischer, D. and W. Hager (1998). *Dam Hydraulics*. Wiley.

- Vorfi, E. (2011). Caractérisation par mesure non-intrusive du champ de vitesses d'un écoulement consécutif à une rupture de barrage. Master's thesis, Université Libre de Bruxelles.
- Waters, R. (1973, March). Brain damage. Pink Floyd - The Dark Side of the Moon LP.
- Watson, D. and G. Phillip (1984). Triangle based interpolation. *Mathematical Geology* 16(8), 779–795.
- Wernet, M. and A. Pline (1993). Particle displacement tracking technique and cramer-rao lower bound error in centroid estimates from ccd imagery. *Experiments in Fluids* 15, 295–307.
- Westerweel, J. (2000). Effect of sensor geometry on the performance of PIV interrogation. In R. Adrian, D. Durao, F. Durst, M. Heitor, and J. Whitelaw (Eds.), . *Laser techniques applied to fluid mechanics*.
- White, F. M. (1991). *Viscous Fluid Flow* (2nd ed.). Singapore: McGraw-Hill.
- White, F. M. (1994). *Fluid Mechanics* (3rd ed.). New Jersey, USA: McGraw-Hill.
- Whitham, G. (1955). The effects of hydraulic resistance in the dam-break problem. *Proceedings of the Royal Society* 227, 399–407.
- Winckelmans, G. and L. Bricteux (2008). Modified law of the wall leading to turbulent channel flow universal velocity profiles valid down to $re_\tau = 395$. In *61st Annual Meeting of the APS Division of Fluid Dynamics Volume 53, Number 15*.
- Winter, K. (1977). An outline of the techniques available for the measurement of skin friction in turbulent boundary-layers. *Prog. Aerospace Sci* 18, 1–57.
- Yalin, M. S. (1971). *Theory of Hydraulic Models*. London: Macmillan.

- Yanta, W. and R. Smith (1973). Measurement of turbulence transport properties with a laser doppler velocimeter. In *AIAA, 11th Aer. Sc. Meeting, Washington DC*, pp. 73–169.
- Yeh, Y. and H. Z. Cummings (1964). Localized fluid flow measurements with an he-ne laser spectrometer. *Appl. Phys. Letters* 4, 176–178.
- Zagarola, M. V. and A. J. Smits (1998). Mean-flow scaling of turbulent pipe flow. *Journal of Fluid Mechanics* 373, 33–79.
- Zanoun, E.-S., F. Durst, and H. Nagib (2003). Evaluating the law of the wall in two-dimensional fully developed turbulent channel flows. *Physics of Fluids* 15:10, 3079–3089.
- Zech, Y., S. Soares-Frazão, B. Spinewine, and N. le Grelle (2008). Dam-break induced sediment movement: Experimental approaches and numerical modeling. *Journal of Hydraulic Research* 46(2), 176–190.
- Zech, Y., S. Soares-Frazao, B. Spinewine, C. Savary, and L. Goutiere (2009). Inertia effects in bed-load transport models. *Canadian Journal of Civil Engineering* 36, 1587–1597.
- Zidouh, H., L. Labraga, and M. William-Louis (2009). Unsteady wall shear stress in transient flow using electrochemical method. *Journal of Fluids Engineering* 131, 051403–1:051403–8.

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