

École polytechnique de Louvain

Dam break experiment over a mobile bed

Determination of the bed shear stress based on velocity profile measurements

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Abstract

Dams have been part of our world for decades and lately, their number notably increased especially in the hydro-power field. A larger number of structure involves a larger risk of failure. Dam-break events rarely occur but are still happening to this day. The consequences of this catastrophes are not well defined yet, in particular the water and the bed sediment behaviour. The erosion of the soil and the topographical impact resulting from a dam failure have not been precisely defined.

This master thesis aims to explore these imprecise phenomenons by characterizing in a better way the shear stress in the water-sediment boundary, based on the water velocity profiles.

To achieve this, we had at our disposal an hydraulic laboratory. It contained a channel able to simulate a dam-break event. This simulation was the base of our analysis and helped us to understand the interactions between the water and the sediments.

We proceeded to record the water velocity profiles necessary to determine the bed shear stress. Thus, we used a high-speed camera to capture the simulations by the mean of a non-intrusive tracer. The images gathered were then investigated throughout the software DaVis8 in order to obtain the horizontal velocity vectors of the water.

From this point, two distinct theoretical methods were investigated to determine their effectiveness in a dam failure case. The Clauser method and the slope method are primarily intended to calculate the bed shear stress in common flows thanks to the velocity profiles but they could still be useful in our investigation.

The simulations were made for several different initial conditions. We varied the sediments properties and the water height upstream the dam. We then compared the results of both methods and how they reacted to the different cases.

Finally, we defined the limits for those methods and proposed changes to their theoretical formulas to adapt them to a dam-break case.

Remerciements

C'est une longue aventure que la réalisation d'un mémoire ; elle ne va pas sans péripéties, sans investissement personnel, sans aide et participation de plusieurs personnes ; et dans le cas présent, avec nous, deux mémorants, de nombreux intervenants ont donné un soutien indéfectible par leur présence, leur expérience, le temps donné sans compter.

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Les auteurs des mémoires et doctorats qui ont précédé le présent mémoire, Adlane Lac Henri Gilmar, Donifan Camacho-Fernandes Pauline Radelet, ainsi que Illaria Fent nous ont fourni des documents clairs et nous ont offert des pistes à explorer pour notre propre travail. Nous souhaitons également les remercier pour leur contribution indirecte dans la réalisation de cet ouvrage.

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Aujourd'hui, forts de leur exemple, de leur soutien à tous, nous nous tournons vers l'avenir, tant de projets encore à réaliser, et nous l'espérons, en étant dignes de leur confiance. . .

List of symbols

A	Wet cross section area of a channel	$[\text{m}^2]$
B	Integral constant for the logarithmic profile of the velocity	$[-]$
B_r	Integral constant for rough boundaries	$[-]$
B_s	Integral constant for smooth boundaries	$[-]$
d	Sediment diameter	$[\text{m}]$
D	Tube diameter of the flow	$[\text{m}]$
d_{50}	Median sediment diameter	$[\text{m}]$
d_x	Sediment diameter for which x is the percentage of finer grains	$[\text{m}]$
d_*	Sedimentological diameter	$[-]$
F_x	x-component of the force vector used in Newton's second law	$[\text{N}]$
g	Gravity acceleration in z-direction	$[\text{m}/\text{s}^2]$
h	Flow depth	$[\text{m}]$
H	Piezometric head	$[\text{m}]$
H	Total head $[\text{m}]$	
h_0	Initial water depth upstream the gate	$[\text{m}]$
k_s	Equivalent bed roughness	$[\text{m}]$
m	Mass of a particle	$[\text{kg}]$
n	Manning coefficient	$[\text{s}/\text{m}^{1/3}]$
P	Wet perimeter	$[\text{m}]$
q	Discharge per unit width	$[\text{m}^3/\text{s}/\text{m}]$
R	Hydraulic radius	$[\text{m}]$
R_b	Bed hydraulic radius	$[\text{m}]$
Re	Reynolds number	$[-]$
Re_*	Sedimentological Reynolds number	$[-]$
s	Relative density	$[-]$
S_0	Bed slope	$[-]$
S_f	Friction slope	$[-]$
$S_{f,MOM}$	Friction slope calculated with the momentum equation	$[-]$
$S_{f,SG}$	Friction slope calculated with the Song and Graf equation	$[-]$

$S_{f,ML}$	Friction slope calculated with the ML model	[-]
$S_{f,SS}$	Friction slope calculated with the steep slope model	[-]
$S_{f,CH}$	Friction slope calculated with the Boussinesq's equation	[-]
$S_{f,Exp}$	slope calculated with the experimental values	[-]
S_w	the water surface	[-]
t	Time	[s]
t^{ND}	Nondimensional time	[-]
u	x-component of the velocity vector	[m/s]
U	Depth-averaged velocity in x-direction	[m/s]
u_*	Shear velocity	[m/s]
u'	Longitudinal velocity fluctuation in turbulent flow	[m/s]
$\bar{u}$	Time-average velocity in turbulent flow	[m/s]
u^{ND}	Nondimensional horizontal velocity	[-]
v	y-component of the velocity vector	[m/s]
V	Velocity vector	[m/s]
V_x	x-component of the velocity vector used in Newton's second law	[m/s]
w	z-component of the velocity vector	[m/s]
W	Wet density	[N/m ³]
w'	Vertical velocity fluctuation in turbulent flow	[m/s]
w_s	the falling velocity	[m/s]
x	Longitudinal coordinate	[m]
x^{ND}	Nondimensional longitudinal coordinate	[-]
z	Vertical coordinate	[m]
z_0	Reference level	[m]
z_b	Bed elevation (generally corresponding to $z_{b,up}$)	[m]
$z_{b,low}$	Bed elevation of the non-moving sediments	[m]
$z_{b,up}$	Bed elevation of the upper moving sediments	[m]
z^{ND}	Nondimensional vertical coordinate	[-]
z_w	Water level	[m]
α	Coriolis coefficient	[-]
α_k	Empirical coefficient in the calculation of the bed roughness	[-]
Δz	Theoretical level from the fixed bed towards which the velocity logarithmic law goes asymptotically	[m]
δ	Thickness of the sheet flow layer	[m]
γ	Specific weight	[N/m ³]

γ_s	Sediment specific weight	$[\text{N}/\text{m}^3]$
ϵ_0	Bed porosity	$[-]$
ϵ_1, ϵ_2	Parameters in the Boussinesq model	$[-]$
θ	Shield's parameter	$[-]$
κ	Von Karman coefficient	$[-]$
μ	Water dynamic viscosity	$[\text{kg}/\text{m}/\text{s}]$
ν	Kinematic viscosity	$[\text{m}^2/\text{s}]$
ρ	Water density	$[\text{kg}/\text{m}^3]$
ρ_m	Depth averaged density of the water-sediment flow mixture	$[\text{kg}/\text{m}^3]$
ρ_s	Sediment density	$[\text{kg}/\text{m}^3]$
ρ_0	Bed density	$[\text{kg}/\text{m}^3]$
τ	Shear stress	$[\text{N}/\text{m}^2]$
τ_b	Bed shear stress	$[\text{N}/\text{m}^2]$
τ_*	Non-dimensional shear stress	$[\text{N}/\text{m}^2]$
ϕ	Internal angle of friction of the bed material	$[\circ]$

Contents

Introduction	15
Motivation and interest of dam break events	15
Context of the thesis	16
Overview	17
1 Theoretical background and assumptions	19
1.1 Steady and unsteady flow	19
1.2 Laminar and turbulent flows	20
1.2.1 Hydraulically smooth and hydraulically rough flows . . .	23
1.3 Depth-averaged equations for a steady free surface flow	23
1.3.1 Uniform flow	25
1.3.2 Gradually varied flow	26
1.4 Influence of the heterogeneity of a section	27
1.5 Determination of the hydraulic radius for the channel	28
1.6 Sediment transport	30
1.6.1 Theory	30
1.6.2 Our case	33
1.7 Different methods to compute the bed shear stress τ_b	33
1.7.1 Clauser method	33
1.7.2 Slope method	34
1.7.3 Reynolds stress method	35
1.8 Investigation of the Sumer et al.'s equation by Adlane Lac and Henri Gilmard	36
2 State of the art	40
2.1 Velocity distribution in steady flows over fixed bed	41
2.2 Velocity distribution in unsteady flow over fixed bed	42
2.3 Velocity distribution in steady flow over mobile bed	44
2.4 Velocity distribution in unsteady flows over mobile bed	47

2.5	Dam break over mobile bed	48
2.5.1	Conclusion of Ilaria Fent's PhD (2018)	49
2.5.2	Progress and conclusions of previous master theses	52
2.5.3	Research leads	55
3	Dam break experiment	56
3.1	General description	56
3.2	Material	57
3.3	General procedure	61
3.4	Gate opening	64
3.5	Parameters	65
3.5.1	Height	65
3.5.2	Distance	65
3.5.3	Time	66
3.5.4	Sand	66
4	Experimental data processing	68
4.1	Determination of the velocity profile	68
4.1.1	Theoretical procedure	69
4.1.2	Calibration	70
4.1.3	Non linear filter and scale length	71
4.1.4	Vector calculation - Time-series of single frames (PIV)	73
4.1.5	Repeatability	76
4.2	Determination of the water and bed surfaces	78
5	Results and analysis	81
5.1	Clauser method analysis	81
5.1.1	Velocity profile found by Sumer et al.	82
5.1.2	Relation $\Delta z - u_*$: Comparison between Sumer et al.(1996) and Lac and Gilmard (2020)	83
5.1.3	Analysis procedure for the Clauser method	85
5.1.4	Results and conclusion of the Clauser method	87
5.2	Slope method	99
5.2.1	Analysis procedure for slope method equation	100
5.2.2	Results and conclusions for the Slope method	100
5.3	Comparison of the two methods	106
5.4	Experimental conclusions	109
5.4.1	Effectiveness of the methods for dam-break analysis	109
5.4.2	Analysis of the Manning coefficient n	110

5.4.3	Questioning the $\Delta z - u_*$ relation	111
6	Conclusion	114
	Annex 1	120
	Annex 2	127
	Annex 3	134

Introduction

Motivation and interest of dam break events

As everyone knows, dams are structures increasingly more present in our society. Their functions can be divers: controlling the flow of rivers, supplying water or even producing hydro-power energy. It is a fact: there are more dams. There are more than 45 000 in the world, two dams are built every day and they take an essential place [7]. Therefore, it becomes interesting to look at the break events of those monuments. This is pretty rare but these events can happen and may be catastrophic [21]. It can cause destruction of lands and communities, it leads inevitably to huge economic losses and moreover can lead to human loses. This still happens and in all countries (recently in 2019 in Brazil where more than 200 persons were missing after the catastrophe [20]).

In the future, humans will maintain and build dams. Even if our capacity to build is more and more sophisticate, it is still possible for one of them to collapse (due to earthquake, climate change, human mistake, etc.). So it seems important to have a better knowledge about the behaviour of those collapses and all the consequences. One of those consequences is the huge flow of water, flow that have impacts on the bed sediment downstream the dam, among others: the behaviour of the flow close to the bed rock, the shear stress in the bed sediment and its erosion. Actually, after a dam break, the huge amount of water released leads to a huge transportation of sediment. There are some theories [1] [22] that predict these phenomenons but there are still a lot of uncertainties. This is why we studied incident throughout numerous experiments.

Context of the thesis

Before presenting the overview of the thesis, it is important to specify that the subject has been actively studied at "*Université Catholique de Louvain*" during the last past years. The sediment movements following a dam failure has been simulated in ideal conditions in a laboratory in order to study it. The determination of the bed shear stress necessary to explain the sediment behaviour could not be performed directly without using intrusive measurement tools that would distort the results. Therefore, the water velocity profiles were measured instead by the mean of a high-speed camera. The profiles were thus determined by digital imaging which did not interfere with the flow itself. Then, several theoretical methods could be used to link the velocity profiles and the bed shear stresses, but their effectiveness is not proved yet in a case of a dam-break event. Those methods are to be investigated later in this work.

This master thesis is in the continuation of the PhD thesis of Ilaria Fent: "*Dam break over mobile bed: experimental and numerical study*" (2018) [9] and two other master theses: "*Mesure du profil de vitesses dans un écoulement consécutif à une rupture de barrage sur lit de sédiments*" by Donifan Camacho-Fernandes and Pauline Radelet (2019) [4] and finally: "*Dam-break over mobile bed: Determination of bed shear stress based on velocity profile measurements*" by Henri Gilmard and Adlane Lac (2020) [13].

Last year, Henri Gilmard and Adlane Lac explored new ideas to improve the determination of the shear stress, they did data analysis on previous experiments to refine those analysis. They also wanted to enlarge the database of the experiments but the lockdown due to the COVID-19 prevented them from going further ahead.

This year, with the partial end of the lockdown, we managed to do a lot of new experiments, and we dealt with these experiments in accordance with the research and conclusions of Gilmard and Lac. We indeed tried to start where they ended and based our analysis on their conclusions. Moreover, we used most of the state of art approaches and assumptions gathered by Ilaria Fent. Also, we explored other tracks in order to complete the experiments with other parameters.

Overview

The first chapter exposes the theoretical background necessary to a good understanding of the master thesis. We wrote here some basic theories of hydraulics like the depth-average equations for steady free surface flow and the differences between a laminar and a turbulent flow that are obviously useful for the subject, but also theories about the sediment transport and the very important point about the different ways to compute the shear stress, with a special attention to the two that we studied in this master thesis: the Clauser method and the slope method.

The second one addresses the state of the art. It puts forward a summary of what has already been studied and what remains to be done. We expose and explain the knowledge in different situations studied prior to the dam break case. More precisely, we explain the steady and unsteady cases over fixed or mobile bed and the many equations used in those cases with a very important one: the Sumer et al. equation. The chapter ends with the dam break case and the conclusions of the previous theses.

The chapter three shows the dam-break experiment itself, i.e. the material, the set-up and the different parameters that we varied during the experiments. Moreover, we elaborate a complete operating procedure to perform the experiments in the best conditions. You will find it in the Annex 1.

In chapter four, we explain how we use the software DaVis8 (LaVision), used to deal with the numerous videos of the experiments taken on the channel. We explain the main technique that is the Particle Image Velocimetry (PIV). We detail the choices made in the program, why using this set-up, or this size of interrogation window for example. It is a crucial step for the rest of our analysis and we are really rigorous about it. It is one of our important contributions to the subject.

The fifth chapter contains the most interesting part of our work: our analysis, our results and the interpretations of them. We expose our results for the two methods and compare them in order to extract some conclusions in the sixth chapter.

Finally, we conclude with the general tendencies that we extracted from our analysis and interpretations. We explain what are the forces and weaknesses

of the two methods, what are the leads that we have excluded and the leads that led us to interesting ways. Finally, we encourage the next thesis writers to investigate some of our tracks that we truly think that could lead to a great advancement.

Chapter 1

Theoretical background and assumptions

In this chapter, we will introduce a series of useful theoretical concepts for our thesis. Some of them are just reminders aiming to help the reader understand some points while others are more central and essentials to the achievement itself of this work.

1.1 Steady and unsteady flow

The steady or unsteady nature of the flow is a very important point of the subject. A flow is said to be steady when its parameters such as velocity, pressure, and flow density are locally independent of time.

$$V(x, y, z) = \begin{pmatrix} u \\ v \\ w \end{pmatrix} \neq f(t) \quad (1.1)$$

On the contrary, the unsteady flow disrespects this condition and the properties are no longer constant in time.

In the case of a dam break event, the flow is highly unsteady. For now, we know a lot more about the behaviour of steady flows. So, we will start from this point in order to study the dam break case.

1.2 Laminar and turbulent flows

It is also interesting to distinguish a laminar flow from a turbulent one [11]. In a laminar flow, particles stay adjacent with time. Their trajectory is predictable. It means that the stream lines are parallel; the flow streamlines are parallel to the limits.

As we see in the course of Fluid mechanics [17], the velocity profile of a laminar flow follows a parabolic distribution (see figure 1.1) and the shear stress follows a linear law whereas a turbulent flow sees velocity fluctuations appear (see figure 1.2).

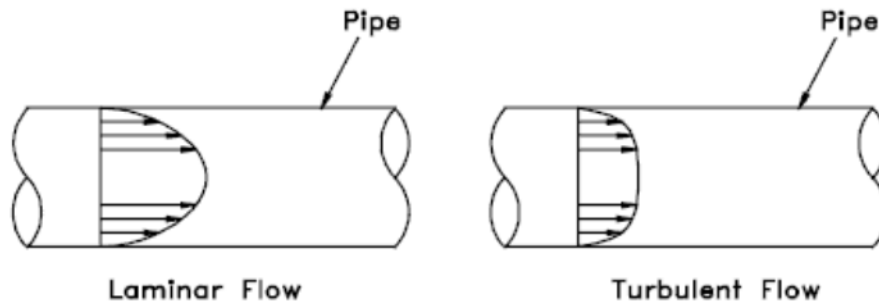


Figure 1.1: Velocity profiles for laminar and turbulent flows.

Reference: www.gunt.de [14]

The velocity in turbulent flow writes (see figure 1.2):

$$u = \bar{u} + u' \quad (1.2)$$

With:

- $\bar{u}$ the time-average velocity [m/s]
- u' longitudinal velocity fluctuation [m/s].

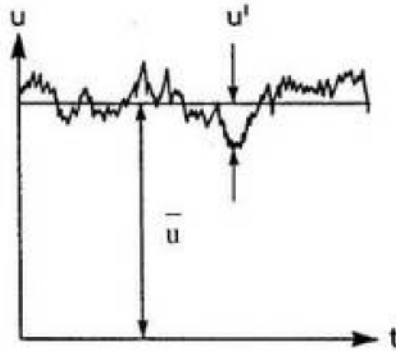


Figure 1.2: Fluctuation term for turbulent flows.
Reference: www.gunt.de [14].

The streamlines stay parallel to the limits but vortices are created, the later disturb the velocity profile and make it less predictable (see figure 1.3). Between those phases, there is a transitory phase.

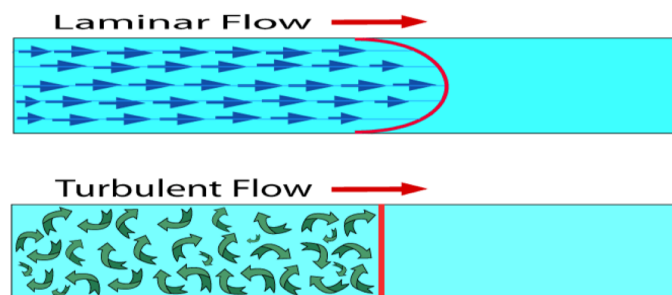


Figure 1.3: Comparison Laminar and turbulent flow.
Reference: www.astarmathsandphysics.com.

The non-dimensional Reynolds number can characterise the laminar or turbulent behaviour of a flow:

$$Re = \frac{UD}{\nu} = \frac{\rho UD}{\mu} \quad (1.3)$$

With:

- U the depth-averaged velocity in the x direction [m/s].
- D the tube diameter of the flow [m] (for a channel, we have to use a $D_{equivalent}$ that corresponds to the water height).
- ν the kinematic viscosity of the fluid [m^2/s].
- ρ the density of the fluid [kg/m^3].
- μ the dynamic viscosity of the fluid [$kg/m/s$].

We know that a flow is laminar for $Re < 2000$ and is turbulent for $Re > 3000$. The flow is in the transitory phase when $2000 < Re < 3000$ (see figure 1.4).

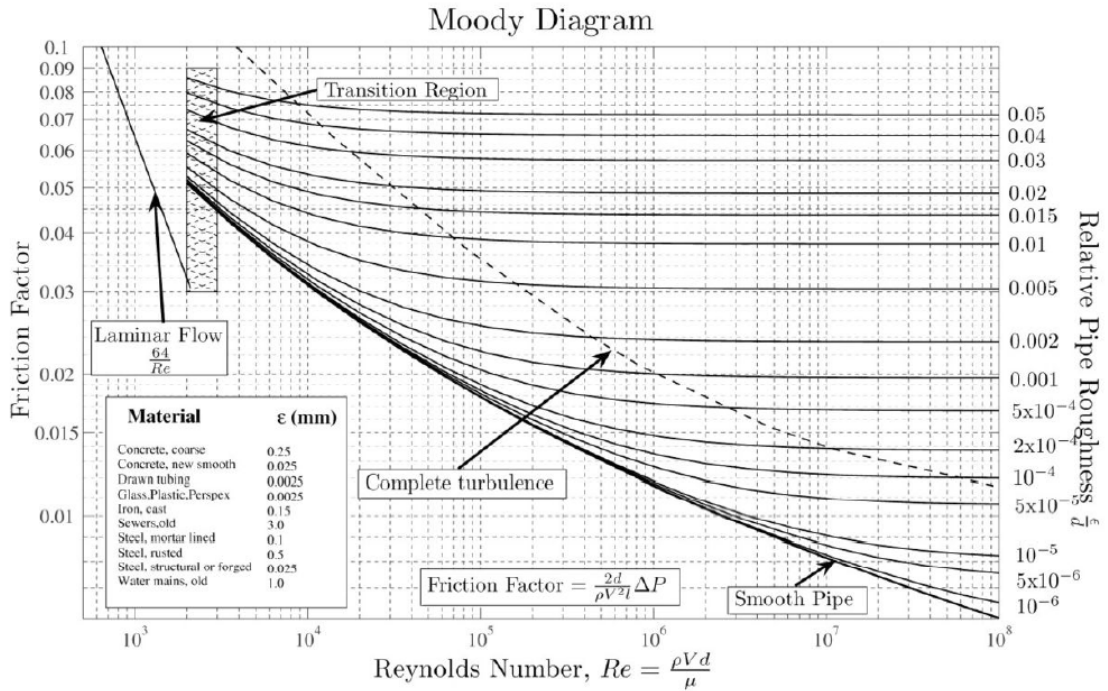


Figure 1.4: Moody diagram.
Reference: S. Beck R. Collins (2008) [2].

Ilaria Fent (2018) [9] and Henri Gilmard and Adlane Lac (2020) [13] have already computed the value of Re corresponding to our experiment.

They calculated it at the section $x = 0$ and after a relatively long time: $t = 5s$. The average velocity at that time was $0.4650m/s$ and the water level was $0.126m$ ($D_{equivalent}$) so they found a turbulent flow with $Re = 58590$.

Knowing that the height and the velocity are much higher in time between 0 and 2 seconds, our experiment is also in a turbulent phase.

1.2.1 Hydraulically smooth and hydraulically rough flows

Now that we know our experiment to be turbulent, we have to determine whether it consists of a smooth or a rough flow. To do so, Gilmard and Lac used the following non dimensional number: the sedimentological Reynolds number:

$$Re_* = \frac{u_* k_s}{\nu} \quad (1.4)$$

Where:

- u_* is the shear velocity [m/s].
- k_s is the equivalent bed roughness [m].
- ν is the kinematic viscosity [m^2/s].

The equivalent bed roughness is related to a specific sediment diameter d_* (often d_{50}) and an empirical coefficient α_k (that they fixed to 1).

$$k_s = \alpha_k d_x \quad (1.5)$$

If $Re_* < 11.6$, the flow is hydraulically smooth. Otherwise, it is rough.

In the case of our experiment, they found $Re_* = 154.8$. This means that our analysis will focus on the case of a rough turbulent flow.

1.3 Depth-averaged equations for a steady free surface flow

For the depth-averaged equations for a steady free surface flow to be valid, our case has to follow a few basic assumptions [24]:

- The flow only varies in one dimension (in the x-direction).
- The discharge does not vary with the distance (continuity).
- The flow streamlines are parallel.

The slopes, S_0 , S_w and S_f are respectively the channel bed, the water surface and the energy line as shown in figure 1.5.

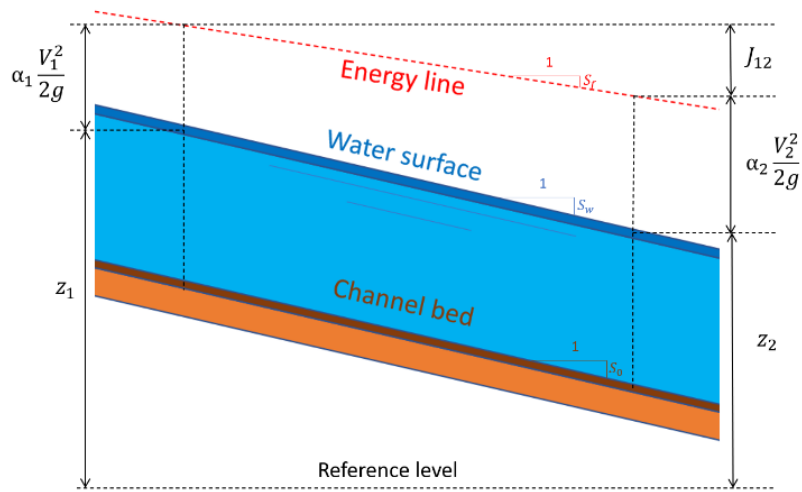


Figure 1.5: Slopes in a free surface channel.

Reference: Adapted from LGCIV2051 - Applied hydraulics: open-channel flows [24].

If we now imagine the section perpendicular to the direction of the flow, the general cross-section of a channel can be represented as follows (see figure 1.6):

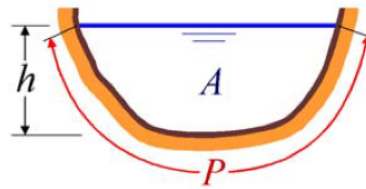


Figure 1.6: General representation of an irregular cross-section.

Reference: Adapted from LGCIV2051 - Applied hydraulics: open-channel flows [24].

where A and P are respectively the cross-section area and the wet perimeter. The hydraulic radius is defined by the relation:

$$R = \frac{A}{P} \quad (1.6)$$

Based on this, we have two possibilities. Either the flow is uniform and the slopes remain parallel or the flow is gradually varied and we then have different slopes.

1.3.1 Uniform flow

In a case of a uniform flow:

- The cross-section area of the channel A is constant.
- The water depth h is constant.
- The velocity U is constant.
- $S_0 = S_w = S_f$.

The depth-averaged velocity U can be determined from equilibrium of friction shear forces and gravity along the direction of flow (see figure 1.7).

$$\tau_b P dx = \gamma A S_0 dx \quad (1.7)$$

where:

- $\gamma = 9810 N/m^3$ is the water density.
- $\tau_b = kU^2$ is the bed shear stress, proportional to the square of average velocity $[N/m^2]$.

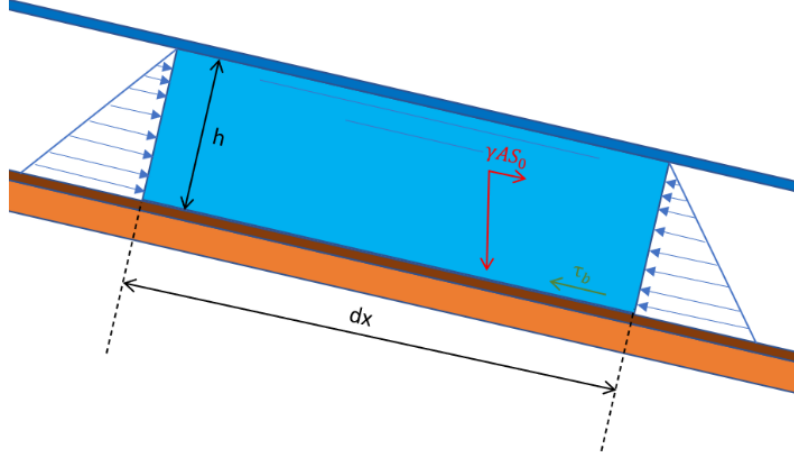


Figure 1.7: Flow with the different forces applied on the control volume.
Reference: Adapted from LGCIV2051 - Applied hydraulics: open-channel flows [24].

Using the equation (1.7) and $\tau_b = kU^2$ leads to the Chézy equation:

$$U = \sqrt{\frac{\gamma}{k}} \sqrt{\frac{A}{P}} \sqrt{S_0} = C \sqrt{RS_0} \quad (1.8)$$

In this equation, $C [m^{1/2}/s]$ refers to the Chézy coefficient. It takes into account the effects of friction.

Manning suggested another expression for C :

$$C = \frac{1}{n} R^{1/6} \quad (1.9)$$

$$U = \frac{1}{n} R^{2/3} S_0^{1/2} \quad (1.10)$$

The equation (1.10) using the bed channel slope S_0 is referred to as the Manning equation for a uniform flow, where $n [s/m^{1/3}]$ is the coefficient of Manning.

1.3.2 Gradually varied flow

In other cases, flows are said gradually varied. The water depths and the velocities show a progressive evolution in space, but the velocity vectors of this type of flows can still be considered parallel.

Therefore, the slopes are not parallel anymore:

$$S_0 \neq S_w \neq S_f$$

The fundamental assumption of the gradually varied flow is that its friction loss in a cross-section is the same as the friction loss from a uniform flow with same discharge and depth.

The equation (1.10) can be adapted to this case by replacing the bottom slope S_0 by the friction slope S_f :

$$U = \frac{1}{n} R^{2/3} S_f^{1/2} \quad (1.11)$$

The equation (1.11) will be referred to as the Manning equation for non-uniform flows.

As we can see, this equation requires to know the energy slope S_f . Adlane Lac and Henri Gilmard [13] explored this question and compared the different ways to calculate this S_f .

1.4 Influence of the heterogeneity of a section

A section may not have all the same surfaces. It is the case in our experiments where the bed is made of mobile sand and the sides walls are made of glass.

Here it is a small demonstration that enables us to compute the equivalent Manning coefficient of our channel:

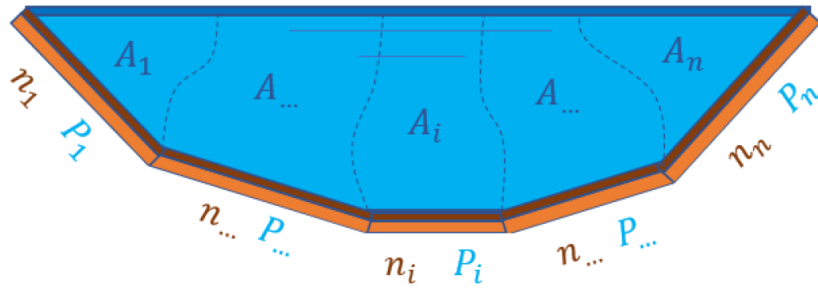


Figure 1.8: Cross section for a bed heterogeneity.

Reference: Adapted from LGCIV2051 - Applied hydraulics: open-channel flows. [24].

We divide the cross-section in different local fictive portions "i". By assuming that the velocity in each area is the same as the average velocity:

$$U_{av} = U_1 = \dots = U_i = \dots = U_n \quad (1.12)$$

So that:

$$\frac{1}{n} R^{2/3} S_0^{1/2} = \frac{1}{n_i} R_i^{2/3} S_0^{1/2} \quad (1.13)$$

By isolating R_i , we have:

$$R_i = R \left(\frac{n_i}{n} \right)^{3/2} \quad (1.14)$$

By definition of R, we know that $R = A/P$:

$$\frac{A}{n^{3/2} P} = \frac{A_i}{n_i^{3/2} P_i} = \frac{\sum A_i}{\sum n_i^{3/2} P_i} \quad (1.15)$$

We find finally:

$$n = \left(\frac{\sum n_i^{3/2} P_i}{P} \right)^{2/3} \quad (1.16)$$

Which represents the equivalent Manning coefficient of the cross-section. For the experiment we do, Ilaria Fent [9] has already computed this equivalent Manning coefficient:

$$n = 0.0165 s/m^{1/3} \quad (1.17)$$

We will use this value in our thesis.

1.5 Determination of the hydraulic radius for the channel

Some equations that we will use later need the hydraulic radius R. We explain here how we computed it for our channel.

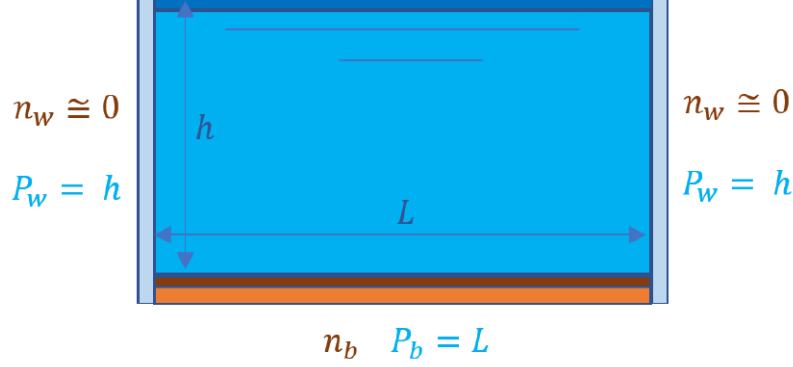


Figure 1.9: Rectangular cross-section of a flume.
Reference: Adlane Lac and Henri Gilmard's thesis. [13]

We see the flume illustrated in figure 1.9. In our experiments, the vertical walls are made of glass. We assumed that the friction on these walls can be neglected, so the Manning coefficient related to the walls is equal to 0. The Manning coefficient n_b is hence the only one to take into account.

Moreover, with the equation (1.14) and (1.16) adapted for our case, we have:

$$n = \left(\frac{n_b^{3/2} P_b}{P} \right)^{2/3} = n_b \left(\frac{P_b}{P} \right)^{2/3} \quad (1.18)$$

$$\frac{n_b}{n} = \left(\frac{P}{P_b} \right)^{2/3} \quad (1.19)$$

And:

$$R_b = R \left(\frac{n_b}{n} \right)^{3/2} \quad (1.20)$$

With those two equations, we have:

$$R_b = R \frac{P}{P_b} = \frac{A}{P} \frac{P}{P_b} = \frac{A}{P_b} = \frac{Lh}{L} = h \quad (1.21)$$

So for all formulas with an hydraulic radius, we will simply use the height h of the flow.

1.6 Sediment transport

The evaluation of solid transport remains uncertain and complex because it depends on numerous parameters, one of them being the way of transportation: by suspension or by bed load. Our dam break experiment is in a case by bed load. For this, we have to determine the minimum and maximum diameters of the grain in function of other parameters.

1.6.1 Theory

In general, in a case of sediment transport, the flow structure can be divided in three zones (see figure 1.10): A zone of sliding and rolling, a zone of saltation and a zone of suspension. The first two zones are called the bed load.

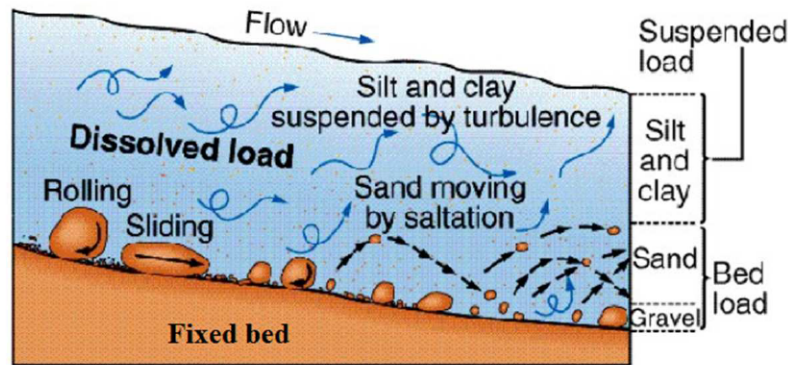


Figure 1.10: The process by which sediment particles are propelled by water down the river channel.

Reference: Physical Geology, (1996) [6].

The movement of a grain is essentially driven by the flow because the velocity gradient in the flow generates a shear stress at the bed level that, if sufficiently high, induces the movement of grains.

We will consider only non-cohesive sediment that are transported as bed load. In this work, we use a deterministic approach for the transport. This approach supposes that the transition between no motion and initiation of grain motion is discontinuous. It happens when the forces applied on the grain become larger than the Coulomb force, which is a combination of the gravity and lift forces (Shields, 1936) [26].

In order to test other sands than the original one, we tried to determine the sediment motion before doing the experiment. We wanted a large enough bed of sediment in order to have a larger part to analyse. But we wanted to avoid a suspension of silt or clay in the water that would distort our results. So we had to determine a grain size range.

The way to know if this kind of transport occurs or not is based on the Shields diagram, which was then readjusted by Van-Rijn in 1984 (fig. 1.11).

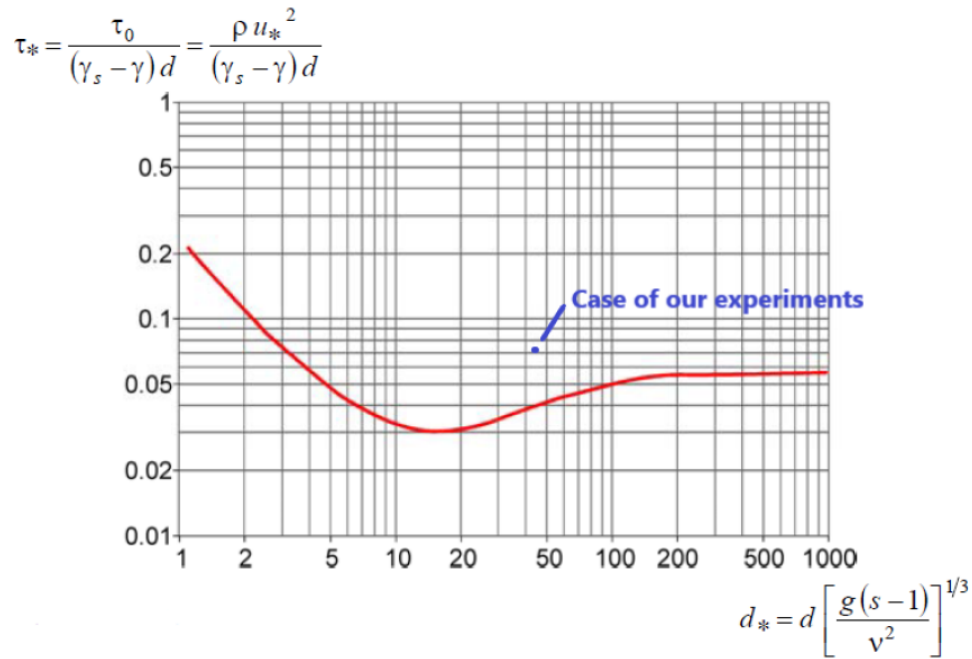


Figure 1.11: Shields - Van-Rijn diagram.

Reference: Adapted from LGCIV2053 - Hydraulique Fluviale. [25]

This diagram (see figure 1.11) makes a relation between the non-dimensional shear stress τ_* and the sedimentological diameter d_* . The area above the red curve corresponds to a flow with sediment transport, while the area below that curve corresponds to a flow without sediment transport.

The blue point is the case of the experiment with the sand used by the previous theses. In our thesis, we will work with other sands as well. We used this dot to compare it with those sands.

In 2008, Garcia [19] suggested a variation of the Shields diagram (see figure 1.12) where we can determine the kind of sediment transport. The bottom curve being the limit between the moving and non-moving sediments. The upper one is the limit between the suspension and non-suspension sediments.

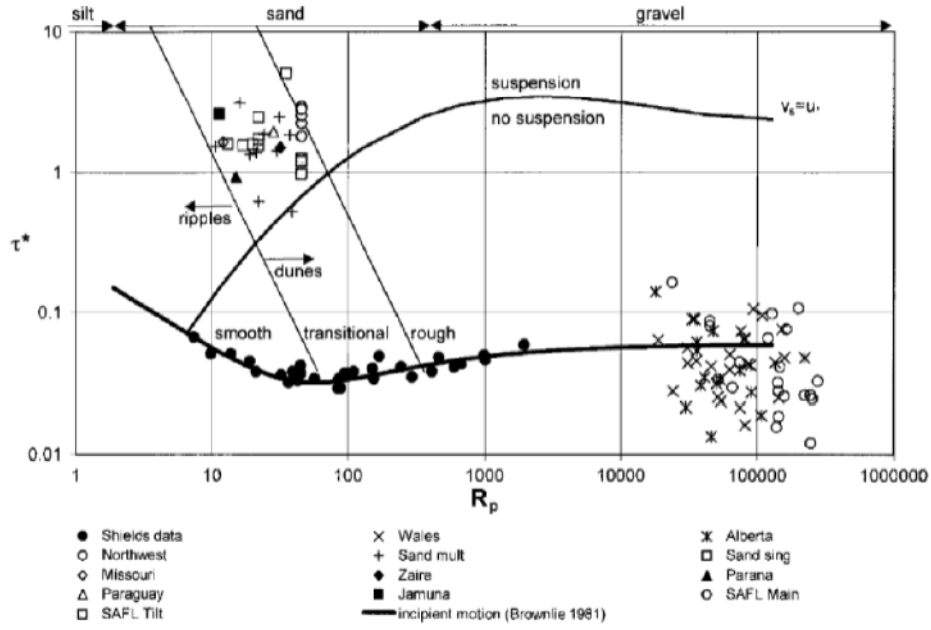


Figure 1.12: Shields diagram from Garcia's letter. [19]

With R_p and τ_* :

$$R_p = \frac{\sqrt{g(s-1)}d^{3/2}}{\nu} \quad (1.22)$$

$$\tau_* = \frac{\tau_b}{(\gamma_s - \gamma)d_{50}} \quad (1.23)$$

Other formulas were determined in order to know if a flow will lead to suspension or bed-load. For example Sumer et al. (1996) [8] proposed that if $\frac{w_s}{u_*} \leq 0.8$ to 1.0 then a bed-load transport occurs. Where u_* is the shear velocity and w_s is the falling velocity of the sediment for which Van Rijn gave a value of $w_s = 1.1\sqrt{sgd}$.

1.6.2 Our case

We needed to find another sand diameter for new analysis. This diameter needed to be small enough to differ from the sand previously used, but it could not lead to suspension behaviour.

We decided to use the Garcia diagram. The main reason is because it puts a limit for suspension/non suspension nature of the grain but also because it shows the smooth, transitional and rough behaviours.

By using the same τ_* as Adlane Lac and Henri Gilmard and thanks to the equations (1.22) and (1.23), we varied the sand's diameter in order to delimit its scope.

We finally found another sand diameter equal to $1.2mm$, which respected the conditions in the Garcia diagram.

1.7 Different methods to compute the bed shear stress τ_b

To compute the bed shear stress is overriding. There are several ways to estimate it in the literature. We exposed those ways in this section. We will investigate and use the two first methods whereas we will just explain the third one.

1.7.1 Clauser method

This method has as starting point the logarithmic law established by Sumer et al. (1996) for the velocity distribution of a steady flow over mobile bed (that will be introduced in the chapter *State of the art*):

$$\frac{u}{u_*} = \frac{1}{\kappa} \ln\left(\frac{z - \Delta z}{k_s}\right) + B_r \quad (1.24)$$

The goal of the Clauser method is to fit the logarithmic line from the logarithmic law (equation (1.24)) with the experimental one.

By varying those values:

- Δz the theoretical level from the fixed bed towards which the velocity logarithmic law goes asymptotically [m].

- u_* the shear velocity [m/s].
- B_r an integral constant for the rough boundaries which varies along the flow [-].

Knowing those values:

- $\kappa = 0.4$ is the Von Kerman constant. [-]
- k_s is a fixed parameter corresponding to the equivalent bed roughness [m].

When the two curves are superimposed, we can extract u_* and use it in the equation (1.25) to compute τ_b .

$$\tau_b = \rho u_*^2 \quad (1.25)$$

Equally, Ferreira et al. (2012) [10] used the differential form of the logarithmic profile:

$$u_* \left(\frac{du}{dz} \right)^{-1} = \kappa(z - \Delta z) \quad (1.26)$$

This form (1.26) can be more convenient for the optimization of u_* and Δz as it does not require the estimation of k_s and B_r and it can be easier to optimize than a logarithm. Yet, it requires the numerical estimation of $\frac{du}{dz}$ which is more complicated with an experimental process.

1.7.2 Slope method

From the section 1.3, we obtain the bed shear stress τ_b which leads to:

$$\tau_b = \gamma R S_0 \quad (1.27)$$

$$\tau_b = \gamma R S_f \quad (1.28)$$

So, knowing γ , R and S_f , it is possible to find τ_b .

As mentioned in section 1.3, there are a lot of way to compute S_f .

- The $S_{f,Manning}$ which is the one that we will use in this thesis.

$$S_{f,Manning} = U^2 n^2 R^{-4/3} \quad (1.29)$$

- The $S_{f,steady}$ and $S_{f,unsteady}$ use two methods based on the definition of the total head.
- Ilaria Fent listed several other methods:
 - The clear layer model $S_{f,MOM}$.
 - The mixture layer model $S_{f,ML}$.
 - The Clear-water layer model over steep slope $S_{f,SS}$.
 - The Boussinesq Model $S_{f,CH}$.

We will not describe all those methods because they are not used in our thesis. You'll find more information about them in the master thesis of Adlane Lac and Henri Gilmard [13].

1.7.3 Reynolds stress method

As explained at the beginning of the section, we will only describe this method but not use it in the thesis. This enables us to not exclude it from the subject if someone want to study it later.

The shear stress can be separated into a laminar component and a turbulent component. The laminar component τ_{lam} can be calculated using the vertical variation of the horizontal component of the velocity and the turbulent component τ_{turb} is determined after the application of the time-average for the turbulent components of velocity [9]:

$$\tau = \tau_{lam} + \tau_{turb} = \mu \frac{\delta u}{\delta z} - \rho \overline{u'w'} \quad (1.30)$$

Where:

- μ is the fluid viscosity [kg/m/s]
- $\overline{u'w'}$ is the time-average [m^2/s^2] for u' and w' that are respectively the longitudinal and vertical velocity fluctuations .

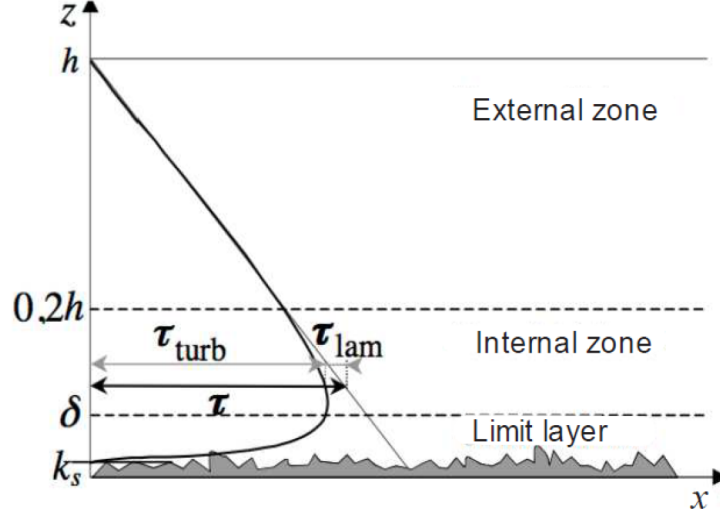


Figure 1.13: Turbulent and laminar components of shear stresses for a steady turbulent flow (Yalin 1977).

This method is typically used for uniform flows. Because of the high unsteadiness of our flow, it can be difficult to make the distinction between fluctuations related to turbulent effects and those related to experimental uncertainties.

Relation between τ_b and the velocity profile

If we pay attention to each of these methods, we can see that $\tau_b = f(U)$ is a function of the velocity. It is the reason why it is essential to analyse the velocity profile as we will see in the next chapter.

1.8 Investigation of the Sumer et al.'s equation by Adlane Lac and Henri Gilmard

In this thesis, we will manipulate the Sumer et al. equation:

$$u = \frac{u_*}{\kappa} \ln(z - z_{b,low} - \Delta z) + u_* \left(B_r - \frac{1}{\kappa} \ln(k_s) \right) \quad (1.31)$$

In order to understand our analysis, let us firstly see the influence of each parameters of this equation on the next graph (see figure 1.14):

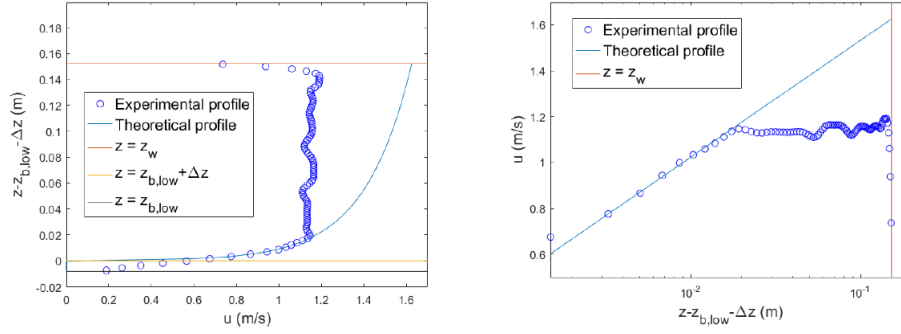


Figure 1.14: Theoretical line fitted on the experimental graph (with regular axes on the left and with logarithmic x-axis on the right).

Reference: Henri Gilmard and Adlane Lac [13].

- The Von Karman constant $[-] \kappa$ which we fixed at 0.407 in our experiments.
- The bed roughness $k_s = d_{50} = 0.0018m$. The similitude between the diameter and the equivalent bed roughness is possible only if the sediments are considered uniform. This is the assumption we made.
- The lower level of the solid transport layer $z_{b,low}$ [m] which varies with time and space as a function of the sediment transport, can be determined locally from the images taken by the camera.
- The shear velocity u_* [m/s] which varies with time and space. If we work with a logarithmic scale, the theoretical log-law profile becomes linear on the graph, and, as a consequence, the effect of the variation of u_* is to affect the slope of the theoretical profile (see figure 1.15).

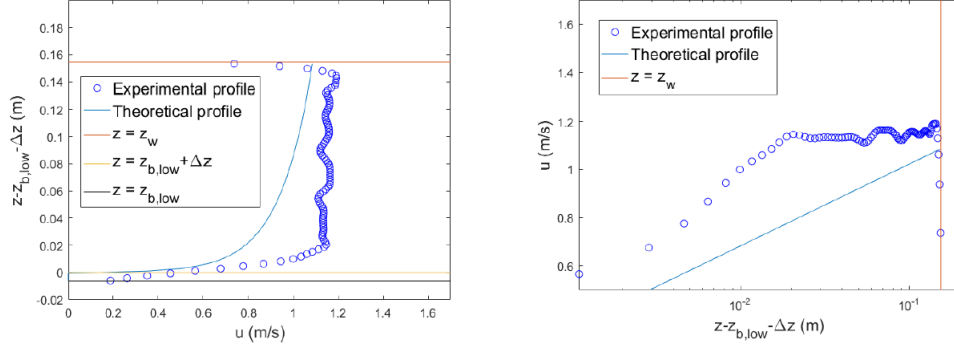


Figure 1.15: Qualitative example of the effect of the diminution of u_* on the velocity profile.

Reference: Adlane Lac And Henri Gilmar d [13].

- Δz [m] which also varies with time and space. Δz corresponds to the theoretical level from $z_{b,low}$ towards which the velocity logarithmic law tends asymptotically. It's important to know that, in the analyses, the reference level of the graph has to be set at $z_{b,low} + \Delta z$. If we work with regular axes (see figure 1.16 on the left), the effect of the variation of Δz is to shift vertically the theoretical profile compared to the experimental one.

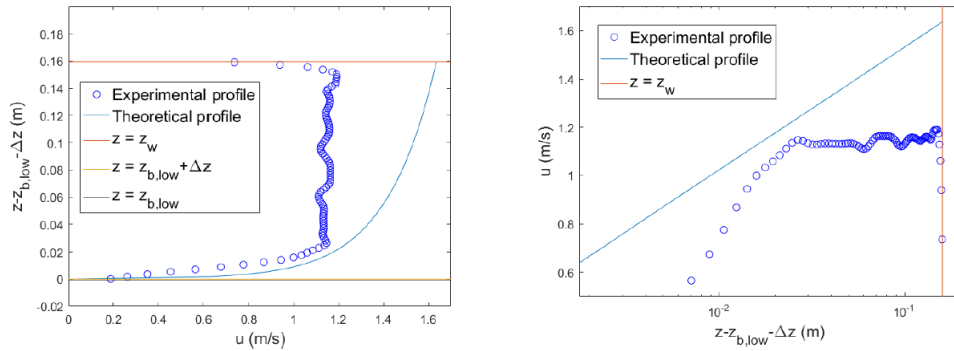


Figure 1.16: Qualitative example of the effect of the diminution of Δz on the velocity profile.

Reference: Adlane Lac and Henri Gilmar d [13].

In this way, Gilmar d and Lac noticed that if they wanted to match the

slope of the theoretical and the experimental profiles in semi-logarithmic scale, they had to take account of both u_* and Δz simultaneously.

- B_r which varies with time and space. The effect of the variation of B_r is to shift vertically the theoretical profile compared to the experimental one (see figure 1.17). In fact B_r is compensating the k_s factor because we are still not sure whether it is the right factor or not (knowing the numerous values possible for k_s).

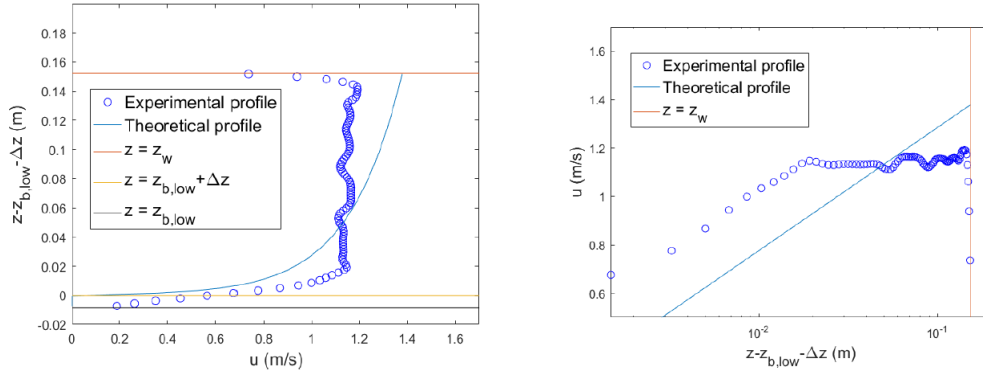


Figure 1.17: Qualitative example of the effect of the diminution of B_r on the velocity profile.

Reference: Henri Gilmard and Adlane Lac [13].

Chapter 2

State of the art

A lot of researches were done in this field, for different kinds of flows : steady or unsteady and different kinds of beds : fixed or mobile. These are the 4 cases that will be described in this chapter. We can find those different situations in the everyday life and some of those are well known while other are still studied. All those cases lead to different profiles of velocity in the flow that we will present in this chapter, with a particular attention about the horizontal velocity profile u .

Actually, this part has already been done by Ilaria Fent in her thesis. Donifan Camacho and Pauline Radelet, Adlane Lac and Henri Gilmard used her work in their thesis too, so it will be very similar. But it is necessary to present this to understand a fifth case that is "the velocity distribution after a dam break" which is the main subject of our master thesis.

Moreover, we have to do a summary of what has already been done and what remains to be done. There are some mistakes in the previous theses. Adlane Lac and Henri Gilmard (2020) gave us some leads that have not been considered before. That is why we finish this section by exposing their results and leads because we will base our analysis from this very end.

2.1 Velocity distribution in steady flows over fixed bed

In 1933, Nikuradse [16] made several experiments in steady flow conditions in circular pipes inside which the walls were covered with sand of a known grain size glued on. The diameter of these grains being defined as the Nikuradse's equivalent sand roughness k_s . Combining his experiments with the experiments of Keulegan (1938)[12] in a flume with steady uniform flow, a logarithmic velocity distribution was observed.

Indeed, for hydraulically smooth boundaries, the flow profile is linear near the wall (equation (2.1)), while it is logarithmic above a certain level (equation 2.2).

$$\frac{u}{u_*} = \frac{u_* z}{\nu} \quad (2.1)$$

$$\frac{u}{u_*} = \frac{1}{\kappa} \ln\left(\frac{u_* z}{\nu}\right) + B_s \quad (2.2)$$

Where :

- u_* is the friction velocity [m/s].
- z is the distance from the fixed bottom [m].
- ν is the kinematic viscosity of water [m^2/s].
- $\kappa = 0.4$ is the Von Kerman constant [-].
- B_s is the integral constant for smooth boundaries, equal to 5.50 from Nikuradse experiments [-].

For hydraulically rough boundaries, the velocity profile is directly logarithmic because of the wall irregularities that break down the laminar layer :

$$\frac{u}{u_*} = \frac{1}{\kappa} \ln\left(\frac{z + z_0}{k_s}\right) + B_r \quad (2.3)$$

Where :

- z_0 is a reference level [m].

- B_r is the integral constant for rough boundaries [-]. If the wall is completely rough, then the constant $B_r = 8.5$ for $z_0 = 0$, according to Nikuradse results.
- k_s is the equivalent sand roughness [m].

Several authors determined the value of k_s : $k_s = 3.5d_{85}$ by Leopold et al. (1964), $k_s = 5d_{50}$ by Griffiths (1981), $k_s = d_{65}$ by Wilberg and Smith (1987) and Patel and Rangea Raju (1999), ... where d_{84} , d_{65} , d_{50} are particle sizes at which 84%, 65% and 50% by weight of the sample is finer, respectively.

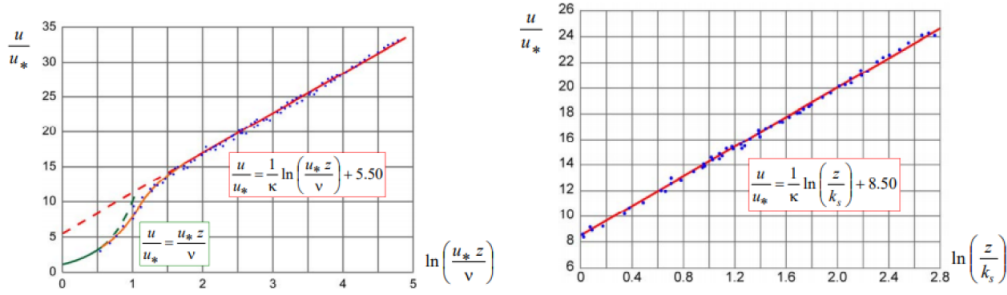


Figure 2.1: Velocity profile (Nikuradse, 1933) for smooth pipes (on the left) and for rough pipes (on the right).

Reference : LGCIV2053 - Hydraulique Fluviale.[25]

It's interesting to know that for free surface flows, a wake effect (described by Coles, 1956 [5]) affects the region close to the free surface, we called it the outer region. It is not shown in figure 2.1 which is based on experiments in pipe flows. However, it is visible in the case of dam break.

The inner region, where the log-law is valid, is often limited to the relative value $z/h < 0.2$, determined by Graf and Altinakar (1988).

2.2 Velocity distribution in unsteady flow over fixed bed

Thanks to imagery techniques such as PIV and PTV (non intrusive velocity measurements), it is possible to investigate the velocity distribution also in unsteady flows.

In 1991, Tu [15] carried out 13 tests in unsteady flows in a inclined fixed-gravel flume with 3 different kinds of gravel. In his analysis, the velocity distribution was slightly different in the rising part of the hydrograph than in the falling part, but the log-law velocity profile (equation (2.3)) was still valid in the inner region (see figure 2.2).

In unsteady flow experiments over non-moving sediments, Song and Graf (1996) took the roughness height directly as $k_s = d_{50}$. They defined the origin of z_0 as the top surface of the gravel bed but with a reference adjustment $z_0 = 0.25d_{50}$ and the constant $B_r = 8.5 \pm 15\%$ in the inner region, this constant being almost independent of the flow unsteadiness.

As can be seen in figure 2.3, the log-law was still valid but with those previous parameters adaptations.

In this case, the logarithmic law is still valid in the inner region $z/h \leq 0.2$.

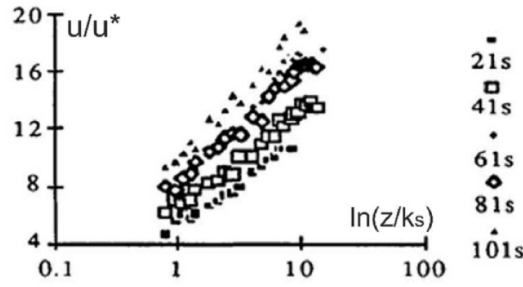


Figure 2.2: Logarithmic velocity profile in unsteady flow over fixed bed.
Reference : PhD thesis of Tu, 1991 [15].

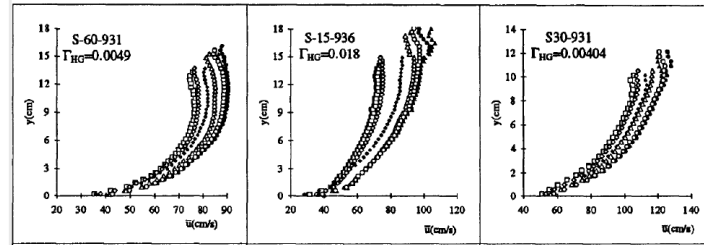


Figure 2.3: Logarithmic velocity profile in unsteady flow over fixed bed.
Reference : Article of Song and Graph, 1996.

Finally, Nezu et al. (1997) determined that the Von Karman constant κ is not considerably affected from the unsteadiness so it can be taken $\kappa = 0.41$.

2.3 Velocity distribution in steady flow over mobile bed

For this kind of flows, Sumer et al. (1996) carried out experiments with water and a sediments recirculation mechanism. The flume was 10 meters long, 0.3 meter large and 0.3 meter depth. They carried out the experiments 158 times with 4 different sizes of sediments and the water surface was smoothed by a rigid lid.

They aimed to study the velocity profile in the layer where the sediment transport takes place, called the "sheet-flow layer".

Based on the experimental velocity profile, Sumer et al. observed that a log-law clearly appears in the upper part of the sheet flow layer and continues in the lower part of the clear water (see figure 2.4). This log-law was similar to equation (2.3) but with a modified definition of z :

$$\frac{u}{u_*} = \frac{1}{\kappa} \ln\left(\frac{z - \Delta z}{k_s}\right) + B_r \quad (2.4)$$

Where :

- z is now the distance to the bottom of the sheet-flow layer [m] (the limit between immobile and mobile sediments).
- Δz is the theoretical level from the fixed bed towards which the velocity logarithmic law goes asymptotically [m] (see figure 2.4).

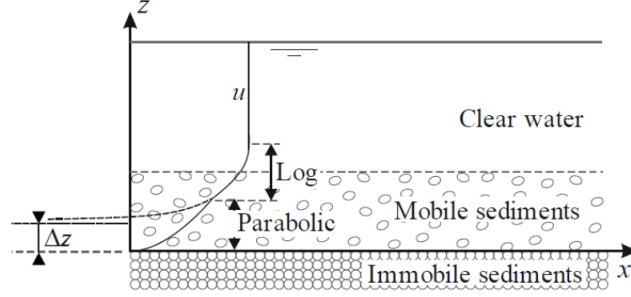


Figure 2.4: Definition of variables by Sumer et al. (1996), for a velocity distribution with logarithmic law near bed in presence of a sheet-flow layer.

Reference : Thesis of Ilaria Fent, 2018 [9].

Sumer et al. defined $B_r = 8.5$ and $\kappa = 0.407$. In order to determine the value of the parameters u_* , Δz and k_s in equation 2.4, they proceeded as follows :

- They combined the equation 1.25 (Clauser method) and 1.28 (Slope method) to obtain :

$$\tau_b = \gamma R S_f = \rho u_*^2 \longrightarrow u_* = \sqrt{g R S_f} \quad (2.5)$$

where they used $S_{f,manning}$ (equation (1.29)) in the Slope method. This value of u_* divided by κ , $\frac{u_*}{\kappa}$, corresponds in fact to the slope of the log-law velocity (equation (2.4)) in the semi-logarithmic scale as we can see in the following development :

$$u = \frac{u_*}{\kappa} \ln\left(\frac{z - \Delta z}{k_s}\right) + u_* B_r \quad (2.6)$$

Putting k_s out of the main logarithmic part :

$$\underbrace{u}_y = \underbrace{\frac{u_*}{\kappa}}_m \underbrace{\ln(z - \Delta z)}_x + \underbrace{u_* \left(B_r - \frac{1}{\kappa} \ln(k_s) \right)}_p \quad (2.7)$$

Giving, in the semi-logarithmic scale :

$$y = mx + p \quad (2.8)$$

With $m = \frac{u_*}{\kappa}$, the slope.

- Once u_* is defined, they determined the value of k_s , as part of the independent term p in equation (2.8) that allows the vertical translation.
- Finally, they determined the value of Δz in order to fit the theoretical log-law profile (equation (2.4)) with the experimental profile on the right portion of the profile as can be seen in figure 2.5 on the left. Sumer et al. also showed, in the same figure on the right, the dependence of the Δz values (adimensionalized dividing by d) with the Shield's parameter θ :

$$\theta = \frac{u_*^2}{g(\rho_s - \rho)d} \quad (2.9)$$

Where :

- u_* is the shear velocity [m/s].
- ρ_s is the density of sand [kg/m^3].
- ρ is the density of water [kg/m^3].
- d the grain's diameter [mm].

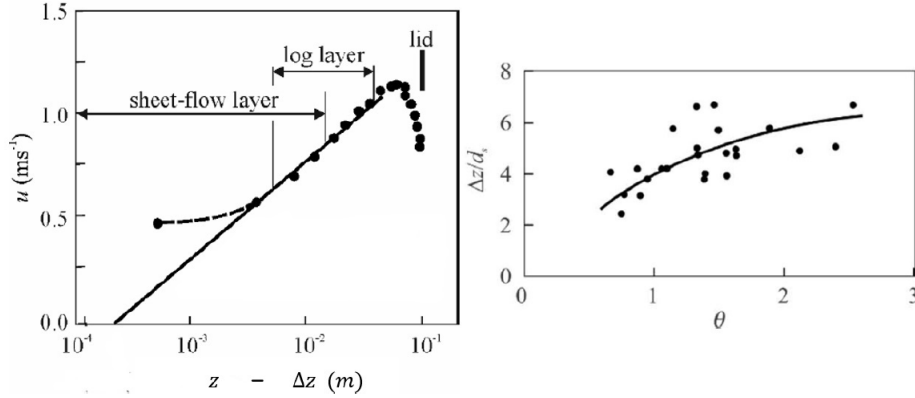


Figure 2.5: On the left : Experimental velocities fitted with the theoretical one.

On the right : level of the theoretical bed Δz in function of θ .

Reference : Sumer et al., 1996 [8].

Sumer et al. represented the thickness of the sheet-flow layer δ (adimensionalized dividing by d) in function of the Shield's parameter θ (see figure 2.6) where we can see that δ is proportional to θ .

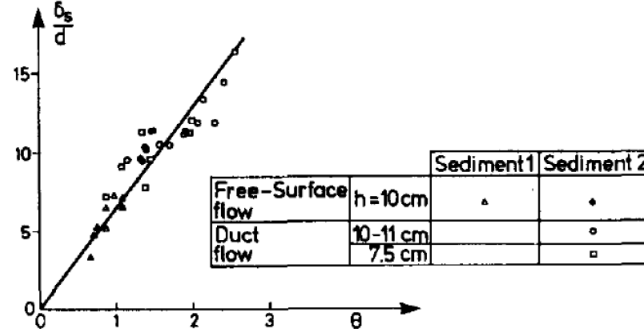


Figure 2.6: Thickness of the sheet-flow layer in function of θ .

Reference : Sumer et al., 1996 [8].

In the lowest part of the sheet-flow layer, Sumer et al. observed a parabolic shape of the velocity, then they proposed a power law for the velocity distribution :

$$\frac{u}{u_*} = \frac{2.5}{\theta^{3/4}} \left(\frac{z}{d}\right)^{3/4} \quad (2.10)$$

2.4 Velocity distribution in unsteady flows over mobile bed

For this kind of flows, Qu (2003) carried out 12 experiments with various hydrographs in a tilting flume. The flow starts over a raised fixed bed with an appropriated roughness in order to condition the flow. A little further, the flow goes over a mobile fine gravel bed with the same roughness as the fixed bed. After that part of the flow, there is a trap that collects the transported sediments and behind that trap, we recover the same kind of bed as at the beginning of the flume. From his experiments, Qu deduced that the inner region follows a logarithmic profile and that the outer region deviates from the logarithmic trend and follows the Cole's wake function (see figure 2.7). No significant differences were observed in the inner region between experiments over fixed bed or over mobile bed.

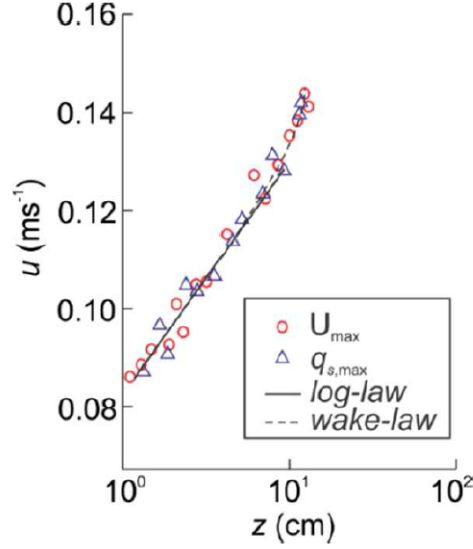


Figure 2.7: Exemple of velocity profile for an unsteady flow over mobile bed.
Reference : Sumer et al., 1996 [8].

2.5 Dam break over mobile bed

A dam break over a mobile bed is similar to an unsteady flow over mobile bed but it has an important transient character. We present in this section the different results obtained on the domain and especially those found by Ilaria Fent in her PhD thesis (2018) and the two master theses that followed.

In the horizontal velocity profile of a dam break over mobile bed, first of all we distinguish three main zones (see figure 2.8):

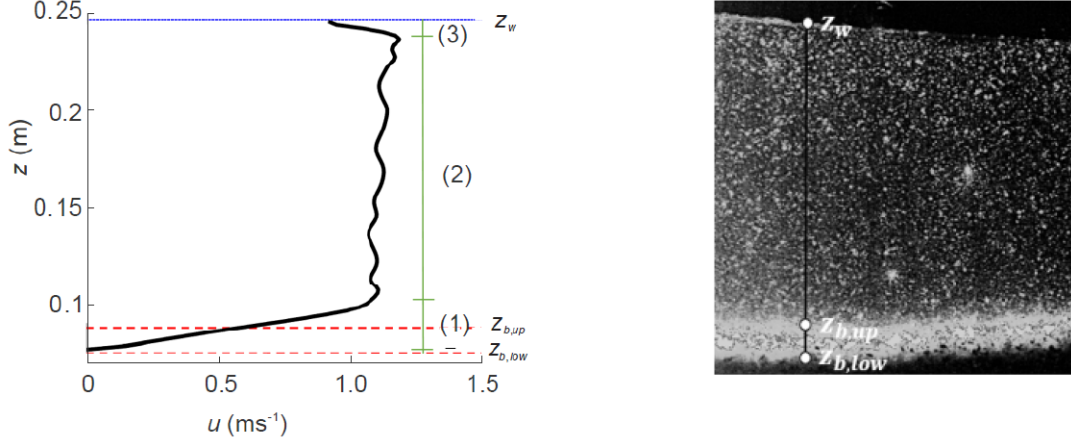


Figure 2.8: On the left : the three main zones (1),(2) and (3) of velocity. On the right a picture of the flow with the level z_w , $z_{b,up}$ and $z_{b,low}$ at the gate section.

Reference : Ilaria Fent's thesis, 2018 [9].

(1) the velocity evolves from zero at the non-mobile bed to a value that remains the same in the zone (2), where the velocity varies around a constant value (this is in line with previous observations, among others by Spinewine and Capart, 2013). In the zone (3) the water surface in contact with the air is affected by a small friction shear stress.

Moreover, on the left figure 2.8, we observe three levels : $z_{b,low}$ is the upper limit of the non moving sand, $z_{b,up}$ the upper limit of the bed load zone and z_w is the free surface of the water.

Be careful, the 3 zones and the 3 "z" are not corresponding. Indeed, The increasing part of the velocity starts from $z_{b,low}$ and stops after z_{bup} ; z_{bup} is approximately the middle of this increasing line.

2.5.1 Conclusion of Ilaria Fent's PhD (2018)

By reversing the axes and making the abscissa logarithmic, Ilaria Fent detected a log-law in the upper part of the sheet-flow layer and in the lower part of the clear water (see figure 2.9), similarly to Sumer et al.'s observation in steady flows over mobile bed (see figure 2.5).

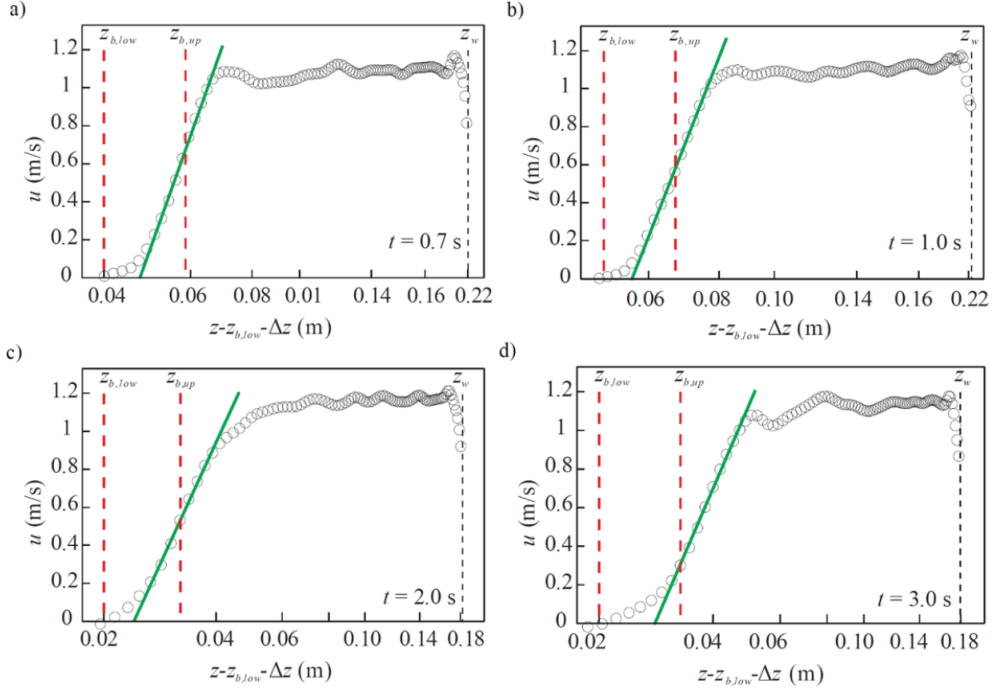


Figure 2.9: Semi-logarithmic plot of horizontal velocity for different times at $x=0\text{m}$.

Reference : Thesis of Ilaria Fent, 2018 [9].

Figure 2.9 shows, at the position of the gate $x = 0$ for times $t = 0.7\text{s}$, 1s , 2s , 3s the comparison between the experimental velocity and the logarithmic regression line computed from :

$$u = \frac{u_*}{\kappa} \ln(z - z_{b,low} - \Delta z) + u_* \left(B_r - \frac{1}{\kappa} \ln(k_s) \right) \quad (2.11)$$

Where :

- $\frac{u_*}{\kappa}$ is the slope of the line on the semi-logarithmic scale where she extracted the value of the shear velocity u_*
- Δz is the theoretical level from the fixed bed towards which the velocity logarithmic law tends asymptotically. The value of Δz is as the one giving the best linear regression for the measured values of the velocity closest to the upper limit of the mobile bed layer $z_{b,up}$, considering at least 8 measured points.

She observed that the clear layer water is practically uniform from 0.7s (see figure 2.10). That is in line with value of 0.46s (corresponding to the time when horizontal velocity components clearly predominate).

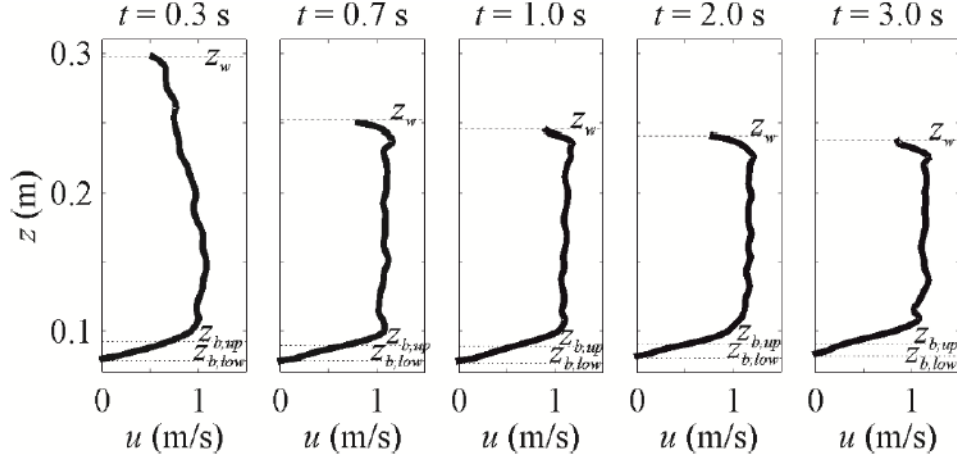


Figure 2.10: Horizontal-velocity profiles at different times at $x = 0\text{m}$. The limits surfaces are indicated with dashed lines.

Reference : Ilaria Fent's thesis, 2018 [9].

She noticed that the velocity distribution in the low part of the sediment layer deviates from the log-law. This was observed by Sumer et al. (1996). But it differs from observations by Spinewine and Capart (2013) who detected linear velocity profiles in the sediment layer.

Ilaria Fent calculated the value of parameters for the four considered times that are summarized in table 2.11.

time (s)	$z_{b,up}$ (m)	$z_{b,low}$ (m)	Δz (m)	u_* (m/s)	θ (-)
0.0	0.10	0.10			
0.7	0.090	0.071	-0.04	1.26	57.26
1.0	0.089	0.072	-0.05	1.32	62.95
2.0	0.091	0.079	-0.02	0.74	19.91
3.0	0.091	0.078	-0.02	0.79	22.61

Figure 2.11: Evolution of the variables needed in the velocity distribution study and the consequent calculation of shear velocity and Shield's parameter

Reference : Ilaria Fent's thesis, 2018 [9]

Once the shear velocity u_* was found and thanks to the Clauser method (section 1.7.1), it is simple to find the bed shear stress $\tau_b = \rho u_*^2$. For all u_* found at all times and all positions.

Ilaria Fent also tried the Reynolds stress method requiring to extract the velocity fluctuations. But she concluded that this method was not easy to apply to her data because of the difficulty to differentiate the right velocity fluctuations caused by the turbulence with the bad ones that are caused by the experimental inaccuracies.

2.5.2 Progress and conclusions of previous master theses

Donifan Camacho and Pauline Radelet (2019)

Donifan Camacho and Pauline Radelet concluded in their thesis that the slope method (subsection 1.7.2) underrated the shear stress τ_b . In fact they obtained a τ_b equal to $9Pa$ with the slope method against a τ_b equal to almost $90Pa$ for the experimental data.

But Adlane Lac and Henri Gilmard found that Donifan Camacho Pauline Radelet was wrong in their data processing. Adlane and Henri explained that Donifan and Pauline shifted horizontally a logarithm function in a standard axis system which is not equivalent to the horizontal shift of a linear function on a semi-logarithmic axis-system. (You will find more details in the Gilmard and Lac's master thesis [13]).

Adlane Lac and Henri Gilmard (2020)

Firstly, they investigate all the terms of the equation of Sumer et al., equation that are used to fit the theoretical profile with the experimental one :

$$u = \frac{u_*}{\kappa} \ln(z - z_{b,low} - \Delta z) + u_* \left(B_r - \frac{1}{\kappa} \ln(k_s) \right) \quad (2.12)$$

They explained the way that the modification of every term can modify the profile in a standard axis system and a semi-logarithmic system. This avoid making mistakes with the data processing, as it was already done before. That was helpful to begin our analysis.

Secondly, they computed the shear velocity and the shear stress by the Clauser method.

1. On the one hand, by using the $\Delta z - u_*$ relation proposed by Sumer et al. . They used the next relation for the Δz :

$$\frac{\Delta z}{d_{50}} = 0.9937 \ln(\theta) + 6.0996 \quad (2.13)$$

With this relation (2.13), every time they changed u_* , Δz was automatically adapted. With this method, they found values of τ_b between 2.025 Pa and 32.4 Pa.

2. On the other hand , by using parametric optimization:

Instead of the equation of Sumer et al., they chose to optimize the relation $(u_* - \Delta z)$ to best fit the equation of the profile of Sumer et al. (equation (2.12)) to the experimental data. But they concluded that it was difficult to find by optimization as it depended on the choice of the points used for the optimization, which is difficult to make perfectly for experimental cases.

Finally, they widely worked on the slope method. As we know, this method requires the calculation of S_f so they reunited all the S_f available in the literature (section 1.7.2) in order to compare them to each other and determine which is correct or not, which is interesting to use or not in the case of a dam break experiment.

They found that the $S_{f,Manning}$ was the simpler S_f to compute because it was the only one to not include any derivative, while all the other methods did. They also excluded the $S_{f,unsteady}$ because it had a totally different behaviour than the other S_f (see figure 2.12).

Moreover, they tried analytically and numerically to compare the different S_f , and they concluded that, except $S_{f,unsteady}$, the irregularities of the different S_f around $S_{f,Manning}$ are actually due to experimental errors and that, in reality, the different experimental ways to find S_f give similar values to $S_{f,Manning}$.

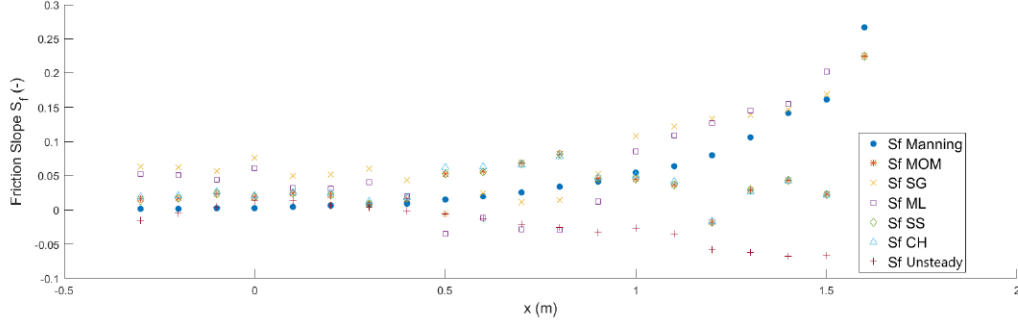


Figure 2.12: Experimental S_f as a function of x for a time $t = 0.9s$.
Reference : Adlane Lac and Henri Gilmard's Thesis, 2018 [13].

So, they encouraged to use this $S_{f,Manning}$, they themselves used it and obtained a τ_b between 4.89 and 31.144 Pa.

They finally plotted their results and made a regression curve on the graph $(\frac{\Delta z}{d} - \theta)$ proposed by Sumer et al. in order to verify the later. They found, by regression, a totally other curve:

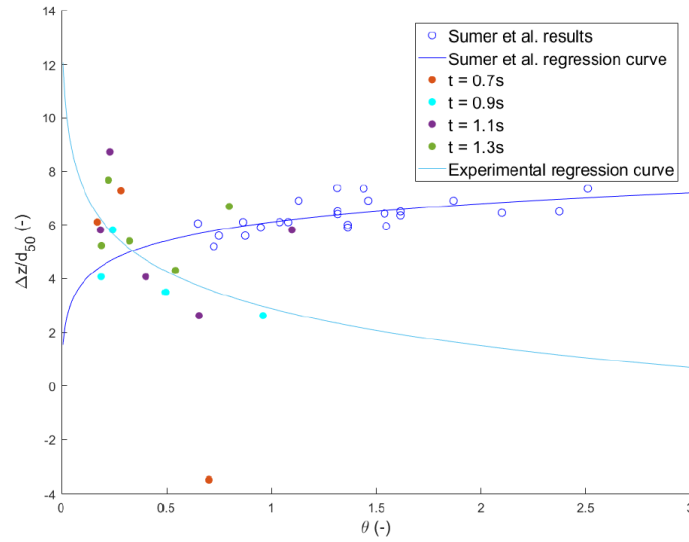


Figure 2.13: Experimental level of the theoretical bed Δz as a function of θ + Results of Sumer et al. (1996).
Reference : Adlane Lac and Henri Gilmard's Thesis, 2018 [13].

They came out with the following logarithmic regression equation:

$$\frac{\Delta z}{d_{50}} = -1.998 \ln(\theta) + 2.8804 \quad (2.14)$$

2.5.3 Research leads

At the end of their master thesis, Adlane Lac and Henri Gilmard let a series of tracks that could complete their work on the subject, we judged those very interesting and decided to stay in their line. In fact they could not do all what they wanted to because of the lockdown, and we wanted to finish their qualitative work. The subject is vast and those leads have partially helped to orientate our choice of research.

Here are exposed some of them (enumerate one by one as they did) that we investigated in the following of this thesis:

- To make the parametric optimization work better, they recommend changing the eroded material. The choice of eroded material can be made from the Shields diagram. The aim is to take a material whose grains are lighter (smaller γ_s) in order to have a bigger sheet flow layer and to better determine the logarithmic portion of the profile because there will be more points to analyze. This could lead to a reduction of the sensitivity of u_* and Δz and make the calculation of the derivative $\frac{du}{dz}$ more accurate in a parametric optimization. Besides, it could expand the database in order to better characterize the terms k_s and B_r that we put a bit aside, because they more than probably depend on the median sediment diameter d_{50} .
- In order to improve the parametric optimization, a lower bound for $z - z_{b,low} - \Delta z$ could be imposed so that the inside of the logarithm in the profile equation is never negative. This way, the differential formulation would not be necessary. Nevertheless, the values of k_s and B_r would have to be fixed.
- There should exist a way to have more points of the velocity profile. If this is indeed possible, it would help for all the methods they used, especially for parametric optimization.

Chapter 3

Dam break experiment

This section explains how we extracted the data samples throughout the dam break experiment. We will go step by step into the way the dam break is simulated and will detail the material used for this profit.

The main observations we can extract is the behaviour of the water and more specifically its velocity profile along the vertical. This profile is influenced by the water height before the break, by the sand properties and by the time.

In order to determine those influences, we took several samples with the same initial conditions and then, we took another set of data with one modification. This helped us to isolate the change of behaviour in the water velocity profile for our analysis.

3.1 General description

These dam break experiments were realised in the LEMSC (*Laboratoire d'Essais Mécaniques, Structures et Génie Civil*) laboratory. It consists of a 6m long channel with a rectangular cross section 0.25m wide and 0.5m high used to simulate a canal with a dam. The channel and its component are more accurately described in the next section.

A movable door placed at its center would act as the dam itself. By moving down very quickly it simulates a dam failure. It is made of aluminum, 12mm thick, 24.4cm wide and 80cm high.

The figure 3.1 shows a picture of the channel:

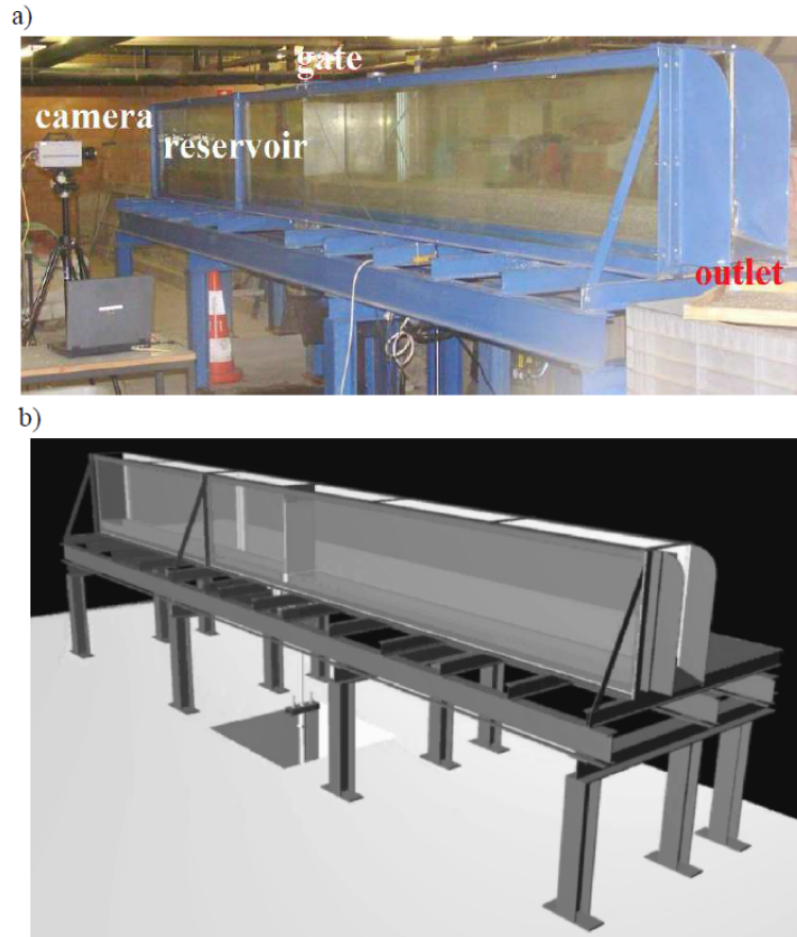


Figure 3.1: Dam break channel in the Hydraulic Laboratory of UCLouvain.

a) picture at the laboratory; b) overall view from Spinewine (2005).

Reference: Ilaria Fent's thesis, 2018 [9].

3.2 Material

As material, we had at our disposal:

1. For the experiment set up:
 - The channel previously described with the following dimension in the schema. We fill up the upstream part with 30cm of water and all the channel with 5cm of sand (see figure 3.2).

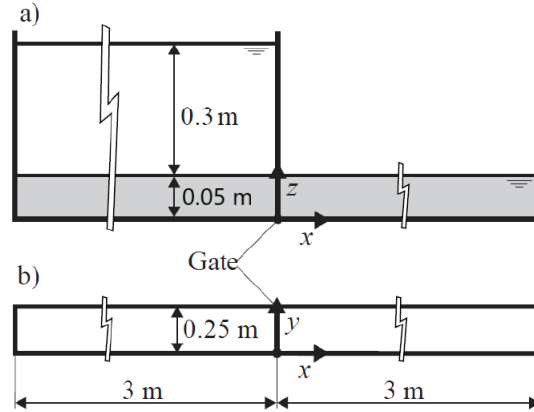


Figure 3.2: Geometry of the dam break channel, with the initial condition of water and sediment depths.

Reference: adapted from Ilaria Fent's thesis, 2018 [9].

- A water pump to fill up the channel and to transfer the used water into the stunt.
- The door that is activated by a pneumatic system. The system served to go down and go up the door (the dam) fast enough to simulate a dam break (see figure 3.3).



Figure 3.3: On the left: The door that simulates the dam. On the right : The compressor.

Reference: Ilaria Fent's thesis, 2018 [9].

- A system of mass (10kg) to compact the bed sand before the experiments (see figure 3.4).



Figure 3.4: The handmade system that permit to compact the sand.
Reference: Ilaria Fent's thesis, 2018 [9].

- A punctured tube used to prevent the water stream of the pump from digging into the sand layer (see figure 3.5).



Figure 3.5: Punctured tube.

2. For the video recording:

- A high-speed camera (fastcam SA3 Photron): The images are recorded

by the high-speed camera at a frequency of 500 fps and saved in the software DaVis8.

- A tracer that we put into the water to detect its displacement in every place of the flow. This tracer is in fact a powder (the pliolite) that we sift in order to obtain a certain seeding in the water. This particular material is used because its density is close to the water density ($\frac{\rho_p}{\rho_w} = 1.03$) making it ideal to follow the water profile. Illaria Fent noticed that using a sieve at $475\mu m$ to filter the pliolite gave satisfying results (see figure 3.6). The particles of pliolite are very small and they tend to float due to the surface tension of the water. To avoid this, the pliolite is firstly mixed with water and a drop of soap inside a bottle to allow better dispersion of seeding in the fluid. The mixture is then incorporated within the entire volume of water hoarded in the reservoir.

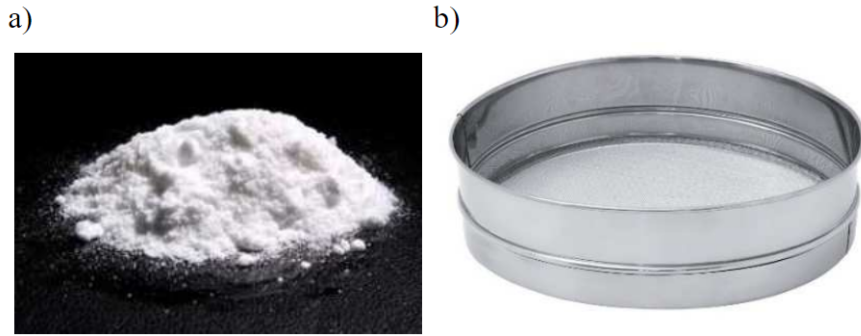


Figure 3.6: Seeding: a) example of pliolite and b) sieve at $475\mu m$ used to filter the pliolite.

Reference: Ilaria Fent's thesis, 2018 [9].

- A laser with a spectral wave length $\lambda = 660\mu m$. This laser allows the software DaVis8 (LaVision) to detect the pliolite particles by enlightening them. The PIV process is explained in more details in the next section. A general set up of the laser-camera combination is shown on the figure 3.7. We can observe the pliolite particles being enlightened by the laser so that the high-speed camera can record its movements.

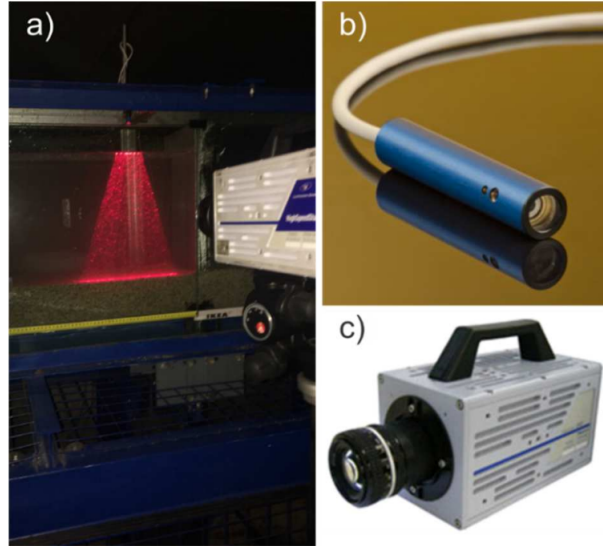


Figure 3.7: General set up of the high speed camera and the laser.
Reference: Ilaria Fent's thesis, 2018 [9].

3.3 General procedure

The main goal of this section is to describe a procedure that ensures repeatability. Each experiment should display results as similar as possible in order to give coherent data.

The laser sheet should be placed at the studied area, 3cm from the glass window of the channel and perfectly parallel to it. It should be strongly attached to make sure it does not shift from one experiment to another.

The channel should first be filled with the wanted width of sand (5cm in our case, more if needed for other cases). This sand layer should now be compressed by the mass system described at the material section. This will prevent unwanted singularities in the sand and give similar conditions for all the experiments.

Then, the upper part of the channel can be filled with water. We used a device (see figure 3.5) to prevent the water stream from digging in the previously compacted sand. The downstream part should be saturated with water to avoid air bubbles. This will also prevent the first test, when the sand is still dry, to differ

from the next tests when all the sand is obviously saturated.

The next step is to fill the upstream water with the tracer described in the previous section. We made sure that the particles were well dispersed in the water by gently mixing the whole. As shown on figures 6.4 and 6.5, the density of the particles of the water should not be less than the first picture and not be more than the second one. The goal is to have enough particles so that the interrogation windows can be reduced as wanted.

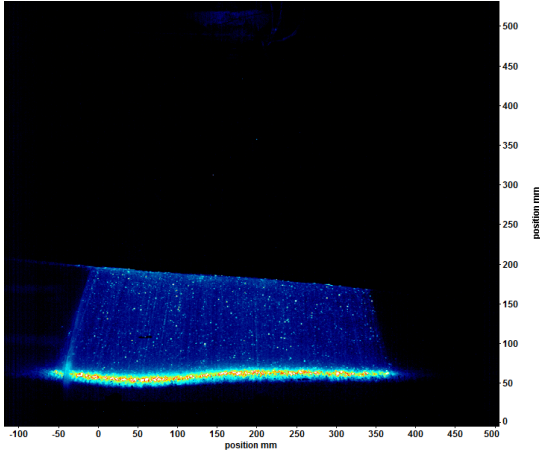


Figure 3.8: Poor seeding.

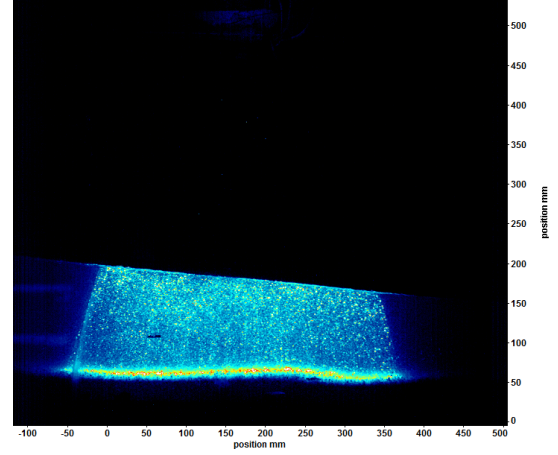


Figure 3.9: Maximum seeding.

Now that the channel is ready, we can proceed to the camera setting. The camera has to be linked to the computer with an ethernet cable and to the PLC (presented after) with a regular cable. We put the tripod and the camera just in front of the section we want to capture, we adjust the height, the inclination and the level of the camera thanks to a little spirit level on the tripod and we do not displace it anymore. When it is done, we can turn the camera on and make the adjustment in parallel to the PFV fastcam software on the computer. We adjust the sharpness of the image under the laser and put the brightness to the highest. Before the opening of the gate, we make sure that the right cable is linked to the PLC and that the save button is activated.

One calibration can be used for all the experiments but the high-speed camera should always be placed on the same spot. We thus marked the ground to ensure that the images were taken with the exact same angle. If the camera

changes place, a new calibration will be needed to ensure that the images are not distorted.

Then, we can start the gate opening and the recording. As we will explain in the following section 3.4, the pressure in compressor (see figure 3.3) should be at 8 bars to correctly simulate a dam-break event. When the compressor is switched on, the air is compressed up to 8 bars and, by opening the valve, the pressurized air is transferred to the compressed air receiver. The purpose of this reservoir is to minimize the distance to the solenoid valve that activates the cylinder, so as to minimize pressure drops. In order to ensure good repeatability, it is important to recharge the pressure in the pneumatic system to 8 bars after each test.

The entire circuit is controlled by a Programmable Logic Controller (PLC) on which we find 3 switches. The first and the third, when held for 5 seconds, controls the descent of the door. The second one, on the other hand, raises the door. It is also possible to connect the camera to the PLC so that holding the first and the third switches triggers the camera (see figure 3.10). This feature is handy in our case to record the exact moment the gate opens.



Figure 3.10: Programmable Logic Controller.
Reference: Ilaria Fent's thesis, 2018.

It is important to remain assiduous with this procedure for each steps when doing the experiment. The same conditions of set-up and execution will lead to coherent results and good repeatability.

3.4 Gate opening

We explained that the pressure managing the gate opening should remain constant at 8bars, The reason behind this is to simulate an instantaneous opening: The gate should open quickly enough so that it does not impact the water velocity profile.

As proposed by Lauber and Hager (1998) and as studied by Aleixo et al. (2011) [23], we can use the following criterion to check that the removal time of the gate t_r is sufficiently small for the removal of the gate to be considered instantaneous:

$$t_r < \sqrt{2 \frac{h_0}{g}} \quad (3.1)$$

The smallest water depth upstream the gate in our experiment is 0.25m, which leads to:

$$t_r < 0.225 \quad (3.2)$$

As explained by Ilaria Fent in her thesis [9], Aleixo (2013) understood that the acceleration of the door depends on the deflection due to the hydrostatic pressure and on the friction between the gate and the water column. He proposed a second criterion to calculate the removal time with a stricter definition:

$$t_r < \sqrt{2\omega \frac{h_0}{g}} \quad (3.3)$$

with $\omega = z_p/h_0$, where z_p is the height of non-perturbed water at the first moment the gate is totally removed.

Ilaria Fent calculated the time interval between the instant corresponding to the top of the gate at the free water surface level, and the instant when it's at the border between water and sediments. The time interval between these two images is $t_r = \Delta t = 9.2 \cdot 10^{-3} s$. She compared this value with $t_r < \sqrt{2\omega \frac{h_0}{g}}$ where she estimated $\omega \approx 0.3$ based on figure 3.11. This leads to $t_r < 0.14s$.

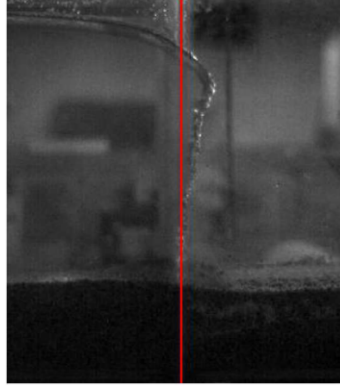


Figure 3.11: Water column perturbation due to removal gate at the instant when the top of the gate is at the border between water and sediments.

Reference: Ilaria Fent's thesis, 2018 [9].

The actual removal time $t_r = 9.2 \cdot 10^{-3}s$ meets both criteria. We can therefore consider the opening of the door to be instantaneous.

3.5 Parameters

Our thesis has the goal to insist and develop the differences created by the variation of one single parameter at a time. From the equation developed by Sumer et al., we choose to study the effect of the next parameters: the initial height of water, the distance from the gate, the time after the dam break event and the last but not least: the size and granulometry of the sand used as bed.

3.5.1 Height

It seems to have a large influence on the shear stress, we wanted to verify if the height influences linearly, parabolically or logarithmically the shear velocity and the shear stress. For this research, we investigate three different height: 25cm, 30cm which will be the reference and 35cm.

3.5.2 Distance

We wanted to see what will be the influence of the distance. In theory, it will not influence the velocity profile but we wanted to know if a distance very close or very far from the dam was interesting and if it distorted the result or not.

3.5.3 Time

Obviously, by the same way as the distance, the goal is to see if a very small time or a too long time gives good results. And especially if there is a "constant" factor on the shear velocity.

3.5.4 Sand

Finally, one of the most important parameters that we study in the thesis: the size of the sand and the variance of its granulometry. Because of the presence of the k_s terms, which itself depends to d_* , in the theoretical equation, the size of the grain is supposed to influence u_* according to Summer et al. .

We also established the variance of the first sand used by Ilaria Fent and the previous thesis in order to reduce it. Then we could do the experiments with the two different variances and compare them to see the influence.

Sand properties

The first sand used in our experiments was the same used by Illaria Fent. However, due to years of manipulation, its median diameter slightly changed. We recalculated its d_{50} and found 1.83mm (instead of 1.72mm). Here are the main properties of our sand (see table 3.1):

Characteristics of the sand	Symbol	Value
Density	ρ_s	2682 k/m^3
Median diameter	d_{50}	1.83 mm
Diameter at 10%	d_{10}	1.15mm
Porosity (void volume fraction)	ϵ_0	0.42
Bed concentration	c_0	0.58
Friction angle	ϕ	38°
Permeability	k	2.27 10-2 m/s
Dry volumetric weight	W	14.54 kN/m^3

Table 3.1: Sand properties.

We found this d_{50} value of 1.83mm by sieving the sand trough several layers and then finding the weighted average. (see figure 3.12).



Figure 3.12: Sieving method.

We then used this data to calculate the standard deviation of this sand diameter and found $\sigma = 0.138mm$. To verify that this standard deviation was small enough, we sieved a new sand using the same method shown in figure 3.12 in order to obtain a standard deviation $\sigma \approx 0$ to compare the results (see subsection 5.1.4). We made sure its d_{50} remained equal to 1.83mm.

Then we sieved a third sand to reduce its d_{50} . The goal was to evaluate if the sand diameter had an effect on the water behaviours. We obtained a new sand with $d_{50} = 1.35mm$. Which is lightly superior to the value of 1.2mm that we wanted but we could not obtain a smaller diameter with the material available.

Chapter 4

Experimental data processing

In this chapter, we explain the different techniques to extract the data from the many videos of the experiments. Mainly, we need two types of data. The first one is the velocity profile of the flow that we obtain with the PIV technique (section 4.1). The second one is the level of the non-moving sand bed, the level of the moving sand bed and the level of the free water surface (section 4.2) that we extract manually (Adlane Lac and Henri Gilmard [13] explaining that it was too inaccurate with the numerical methods we had at our disposal).

4.1 Determination of the velocity profile

There are two important techniques to track the flow. There is the Particle Tracking Velocimetry (PTV) that tracks one particle in particular, and there is the Particle Image Velocimetry (PIV) that tracks a group of particles.

We use PIV in this thesis. It works like this : thanks to the presence of particles illuminated in a flow, it is possible to find the displacement of the groups of particles (the interrogation window) between two consecutive images. Once the displacement of the particles is found, and knowing the time interval between two images, it's possible to compute the velocity of those particles (via cross-correlation).

You can see an illustration of the PIV technique in figure 4.1.

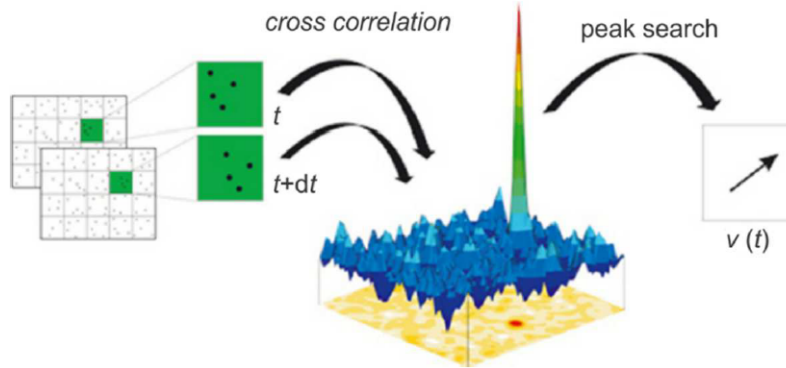


Figure 4.1: PIV data evaluation via cross correlation.
Reference: FlowMaster manual of LaVision [3].

In summary, the position of the correlation peak is the position of the most probable displacement dx of the interrogation window and the particles in it. This displacement dx and the time interval dt (from camera settings) can then be used to compute the velocity v of this interrogation window.

$$\frac{dx}{dt} = v \quad (4.1)$$

The advantage of PIV is that it is a non intrusive technique and so we avoid the modification of the flow behaviour. Nevertheless, it is necessary to have a tracer with a density similar to the flow. It is why we use "pliolite" that has a density $\rho = 1.03$.

It certainly was a brief explanation of how it works but in reality there are some parameters that are important to choose right as the scale length for the *non-linear filter*, the overlap and the interrogation windows for the *vector calculation - time-series of single frames*.

We will develop our choices later in this section.

4.1.1 Theoretical procedure

Here is the procedure to obtain our velocity profiles from the images of the flow taken by the camera (You will find some screen shots of the step in the Annex 2):

1. As a reminder, at this stage, it is supposed that we have the video of the flow with a good seeding of the pliolite in the water, a good quality of image and that we have exported them in the *.bmp* format.
2. From there, we can import the frames on the software DaVis 8 (LaVision).
3. The first thing to do on the software is to calibrate and re-scale all the frames. Indeed, those frames are calibrated in order to correct possible distortion. And re-scaling is essential to transform the pixel system into the metric system (in order to be able to use the data later).
The subsection 4.1.2 explains the way to calibrate and what is at stakes during this step.
4. Now that the frames are calibrated and usable, it is essential to filter the frames to highlight the pliolite particles, we do this with the non linear filter process and with the scale length parameter. Without this step, the software will not be able to detect those one correctly and will compute a wrong velocity profile. In the subsection 4.1.3, we explain our choice for this filter.
5. Finally, by using two images, the software can compute precisely the velocity profile by cross-correlation. We have studied the impact of all the parameters that the software put to use in order to obtain the best velocity profile for our following analysis. All the tests about those parameters which are the interrogation window and the overlap are in the subsection 4.1.4.

4.1.2 Calibration

Before doing the calibration, we have to put a calibration plate in the water in the channel (see figure 4.2 on the left) and put the camera at the place where we will take all our videos (in our case it is right in front of the pneumatic door. Warning! The calibration is valid only for the frames taken from this emplacement of camera). We take a picture of this situation and we put it on the software.

DaVis8 uses this picture and, knowing the distance between the point of the calibration plate, calibrates the frame (see figure 4.2 on the right) and applies this calibration to all the other frames that are in the file in the software.

(For more information about how to do the calibration, please refer to the "*FlowMaster Getting Started*" manual. [18])

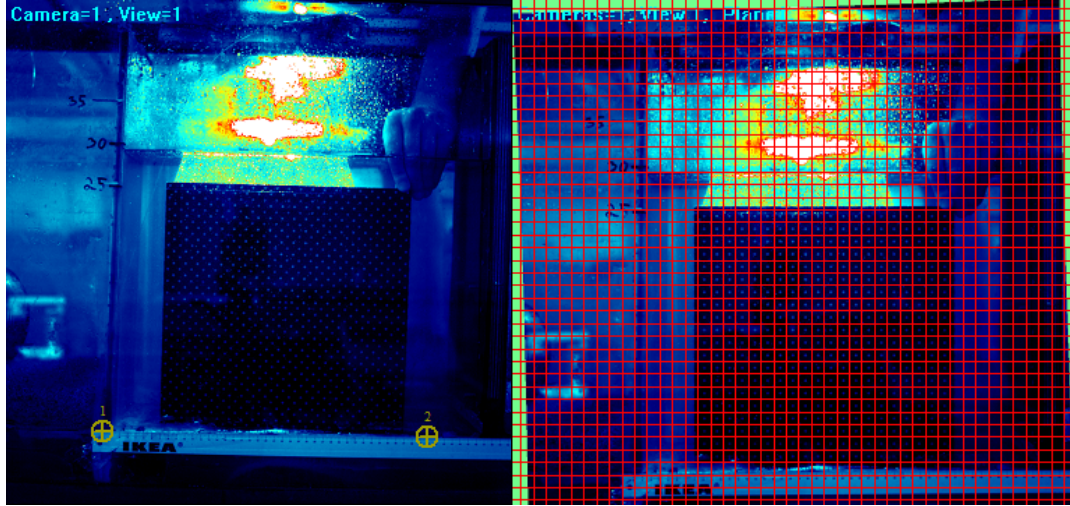


Figure 4.2: The frame used for the calibration - before and after the calibration.

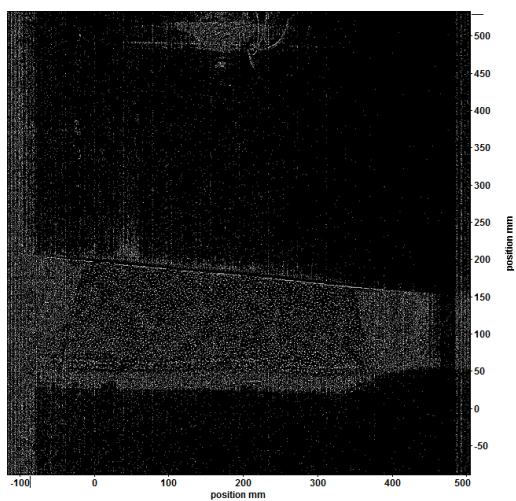
4.1.3 Non linear filter and scale length

Once the calibration was done, we had to filter the frame because the gross one was not directly usable. We needed to put in evidence the seeder (the pliolite in our case). If we did not, the software would end up finding some wrong velocity vector, because there was too much information in the picture.

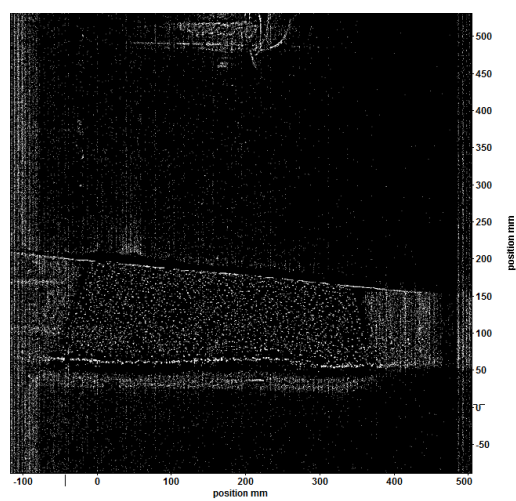
One parameter that we had to choose for the non-linear filter was the scale length and we leaned over it to be sure that, again, we would make the right choice for the rest of our analyses. It is why we made some treatment of several frames with different scale lengths and compared them to each other.

On the figure 4.3, you can see the non linear process for different scale length.

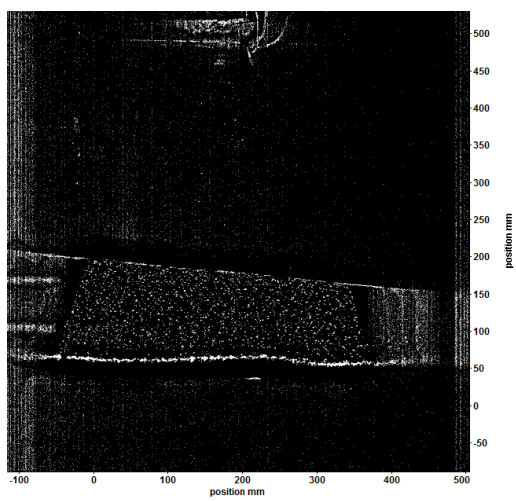
From those frames (see figure 4.3), we can see that higher the scale length is, the less we have particles of pliolite detected by the software. Increasing the scale length may be useful in a case of bad seeding but, in our case, the operation makes the good basic seeding disappear, which is problematic for the next operation. Indeed, after the PIV treatment, with an increasing scale length parameter, we can see that the curve loses in precision. It moves away from the one we expected to have as far as the literature is concerned (see figure 2.4): The steep logarithmic part should start inside the mobile sediment layer



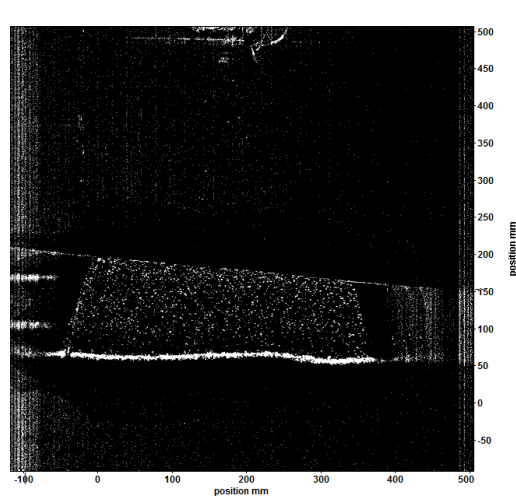
(a) Scale length = 3



(b) Scale length = 9



(c) Scale length = 21



(d) Scale length = 49

Figure 4.3: Different choices of scale length.

(between $z_{b,up}$ and $z_{b,low}$). So, our choice was the scale length 3 (see figure 4.4).

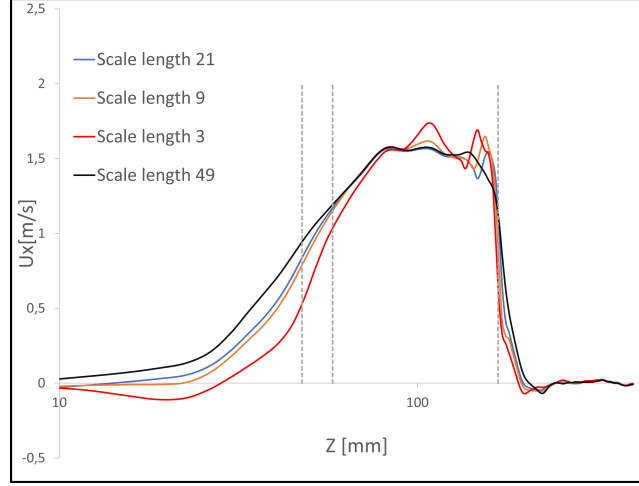


Figure 4.4: Vector calculation of two consecutive frames with four different scale lengths in the non-linear filter step.

4.1.4 Vector calculation - Time-series of single frames (PIV)

From the frames obtained by the non-linear filter function, the software is now able to compute the velocity profiles through the PIV process. But DaVis8 needs to introduce some parameters, and it is up to us to determine them:

Overlap

First of all, the overlap, which is a critical parameter.

The detection of the group of particles in a window from the image at time t and in a window from the image at time $t + dt$ must be done with a certain step, which is linked to an option proposed by DaVis8 called overlap. This overlap is between 0 and 100%

As we can not verify every single parameter, we applied the same values as the precedent thesis of Ilaria Fent (2018) [9] who decided to use a multi-passed option with decreasing size. She suggested to use a first pass with an overlap of 50 % and a second pass with an overlap of 87%. We estimated the results to be sufficiently satisfying.

Interrogation window

In this part, we are interested by the effect on the size of interrogation window on our results. We wanted to determine the smallest interrogation window in order to have the best precision and a maximum number of points in our graphs. Indeed, the more points we had, the closest the curve would be to the reality and the more we would be able to determine a theoretical profile.

On the contrary, we can not take a too small window because in this case, there will be a loss of precision during the treatment and our results will not be correct. The reason why we can not take a too small window is that the resolution of the camera is limited and that the seeding can not be perfect in the experimental case despite an excellent rigor.

To determine the smallest window usable, we chose different sizes of interrogation window for two consecutive frames for the exact same experiment conditions (see figure 4.5):

- The initial height $h_0 = 35cm$
- The time after the opening of the gate $t = 0.9s$
- The distance from the gate $d = 200mm$

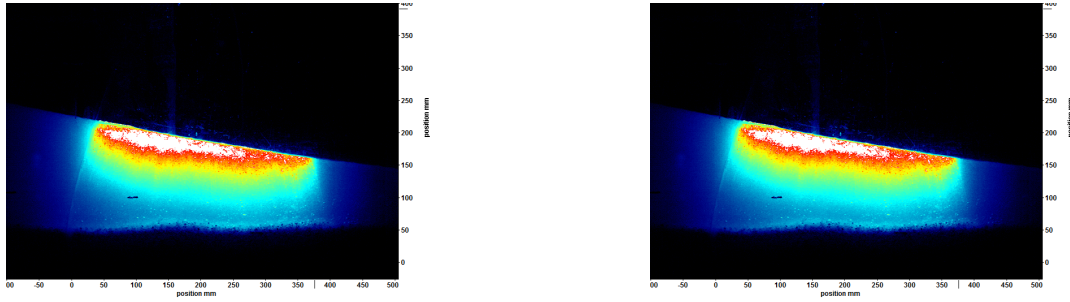
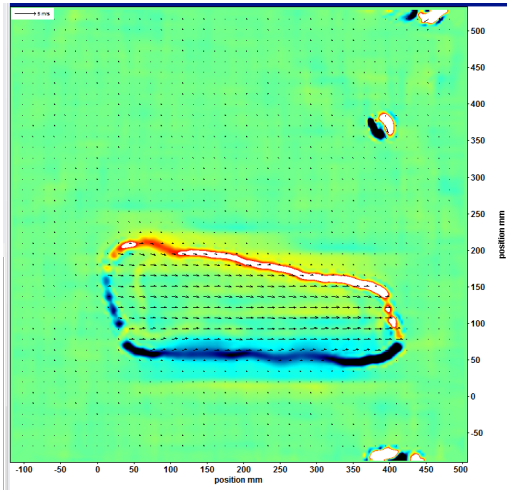
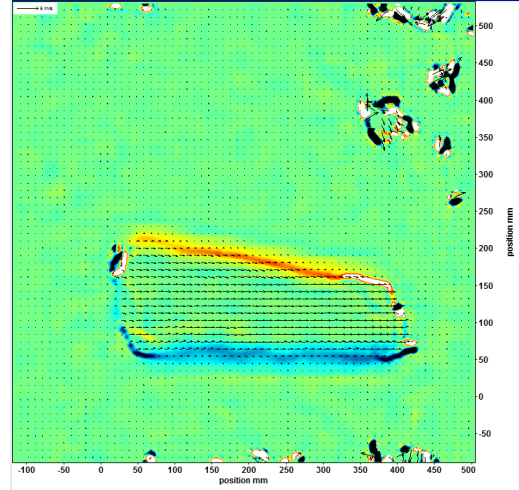


Figure 4.5: 2 consecutive frames used for the PIV treatment.

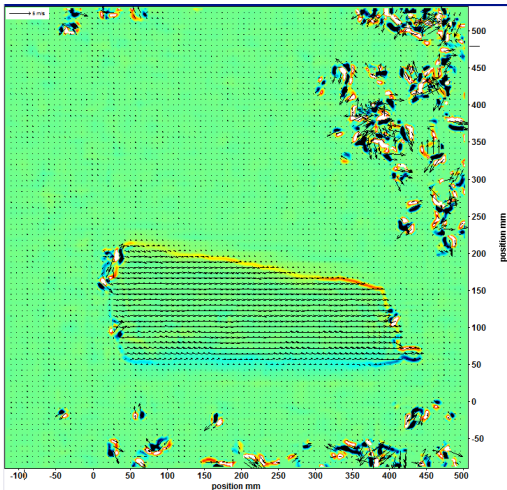
We processed these frames (see figure 4.5) with 4 sizes of interrogation window: 48x48, 36x36, 24x24 and 16x16 pixels. As we can see on figure 4.6 the smaller the interrogation window is, the more we have parasite vectors. The velocity profile however seems correct for all interrogation windows so we decided to analyse the data from all cases in order to choose which is or is not correct and which is the best for our future analysis.



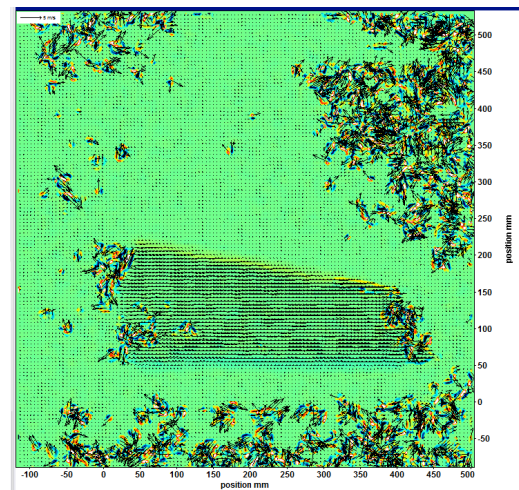
(a) Interrogation window 48x48



(b) Interrogation window 32x32



(c) Interrogation window 24x24



(d) Interrogation window 16x16

Figure 4.6: Different sizes of interrogation windows.

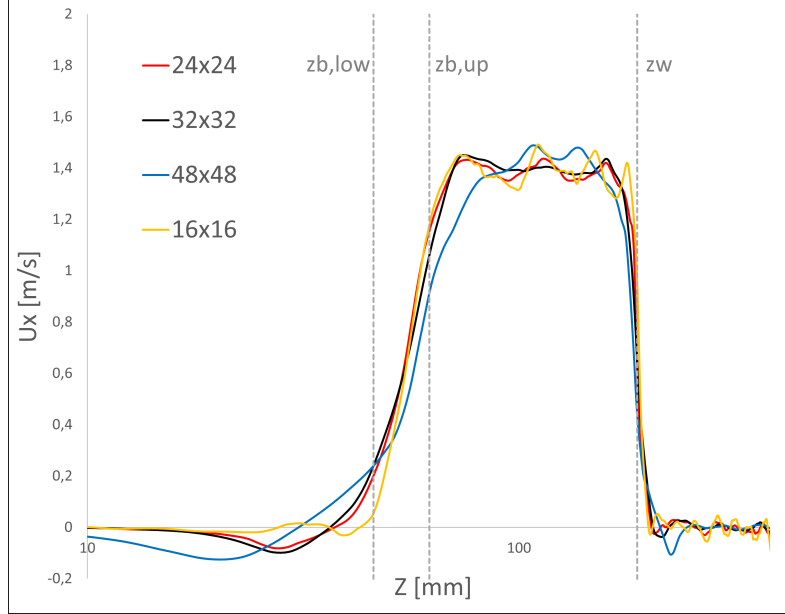


Figure 4.7: Velocity profile with $H = 35\text{cm}$, $t = 0.9\text{s}$ and $d = 200\text{mm}$.

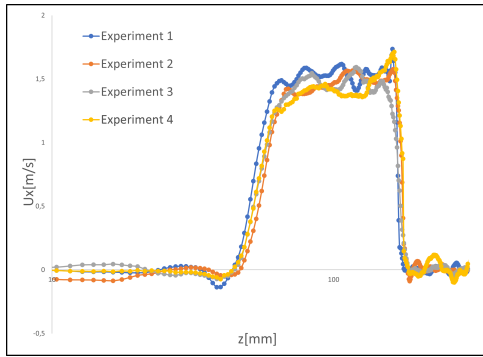
We can see on figure 4.7 that the profile 48x48 is too far from what we expected (Which is the one used in the previous thesis). Also, the velocity curve begins too soon. Actually, the curve fits as much as the interrogation window is small. Indeed, for the 16x16 case, we see that the velocity profile is equal to 0 almost at the same abscissa than the $z_{b,low}$ border. Also the $z_{b,up}$ line is closer to the middle of the log-curve. We were expecting those two properties so we chose the 16x16 pixels interrogation window and processed our analyses with this parameter. The parasite vectors were not a problem since they are not in the flow.

4.1.5 Repeatability

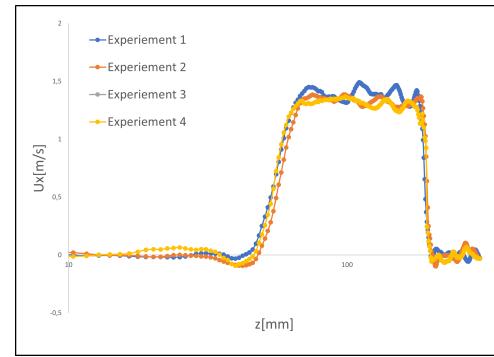
Now that we have determined all the parameters for the PIV analysis. We can verify if we still have a good repeatability after the processing. Ilaria Fent had already assessed the repeatable property of the experiments especially with the water and bed surfaces.

In the figure 4.8, we can see the repeatability of four experiments with a same sand and a height of 35cm. We did the same with other sands and other heights and the repeatability stayed in line with those graphs. We concluded about it that :

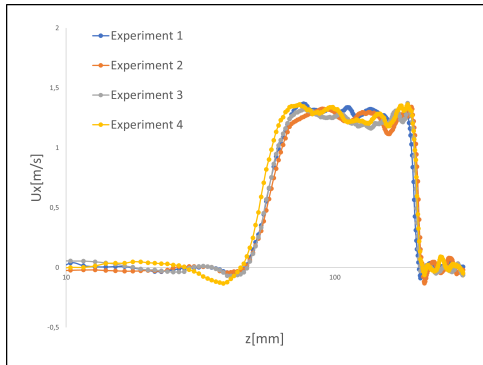
- The repeatability is not enough accurate for time $t=0.6s$ which is in line with the conclusion of Ilaria Fent.
- The repeatability is valid for all our granulometries of sand.
- The repeatability is valid for all our height but seems more accurate with the highest.
- The curve seems closer and closer with time.



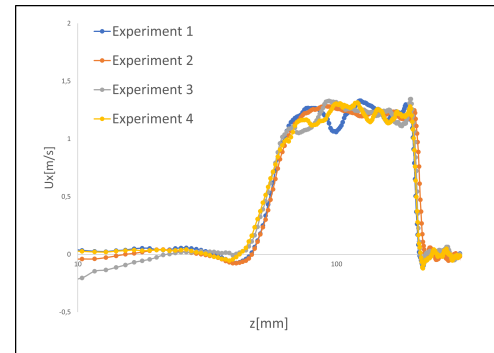
(a) At time $t=0.6s$



(b) At time $t=0.9s$



(c) At time $t=1.2s$



(d) At time $t=1.5s$

Figure 4.8: Repeatability of our process for four different instants for four experiments for the same initial conditions.

On the contrary, to illustrate the usefulness to wet the sand before an experiment, the repeatability between experiments on wet sand and dry sand is not respected (see figure 4.9).

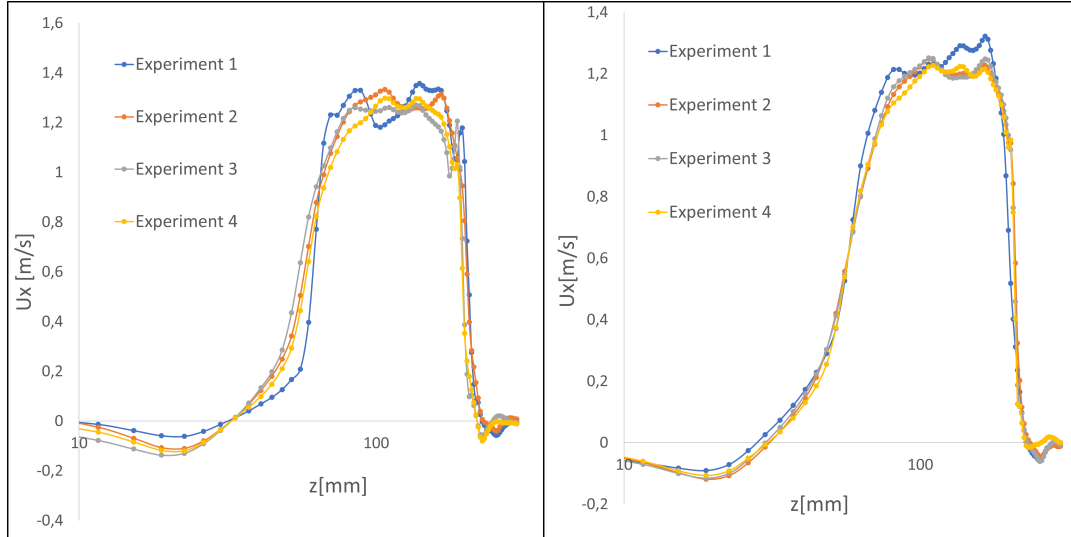


Figure 4.9: Graph of the repeatability with one experiment on dry sand
The blue curve is the experiment with dry sand.

4.2 Determination of the water and bed surfaces

In order to establish conclusions about our analysis and graphs that we plot on the base of the experiments, we have to determine exactly the mobile-immobile bed interface ($z_{b,low}$), the mobile bed-clear water interface ($z_{b,up}$) and the free water surface (z_w). It is pretty simple to determine z_w manually directly on the frames but it becomes difficult for $z_{b,low}$ and $z_{b,up}$.

Adlane Lac and Henri Gilmard spent a lot of time to determine the position of the different interfaces. They firstly concluded that the Matlab code they had did not give them satisfactory results. So they tried a manual option of this Matlab code and they concluded that it gave more precise results and took less time than the first method but they judged it too inaccurate as well (see figure 4.10).

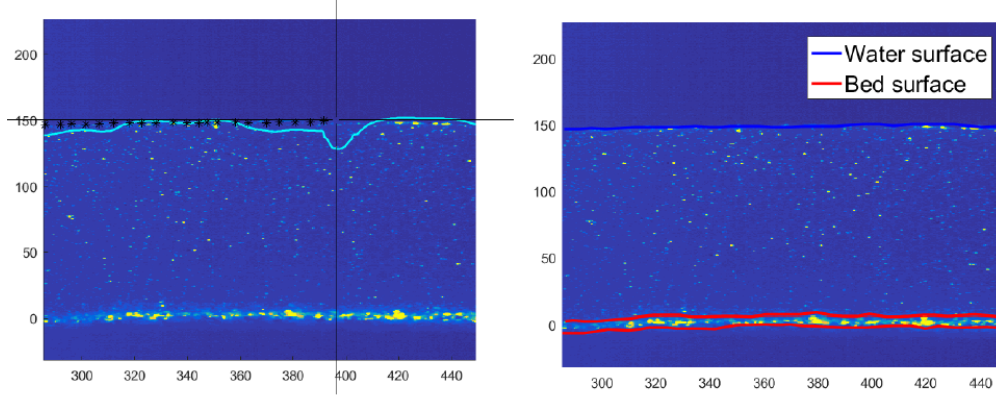


Figure 4.10: On the left : Cursor which allowed to click on points belonging to the level interfaces - On the right : Levels found by manual detection.
Reference : Adlane Lac and Henri Gilmard's thesis [13].

They finally concluded that this Matlab program had to be improved before being used. Unfortunately, we did not put our priorities in this part of the subject. So, in order to determine the interfaces, we took the level directly from the frames (as it is already possible for z_w). Actually, this method is not such inaccurate but takes lot of time. Uncertainties could obviously stay but it is also the case with the Matlab code.

To determinate those levels we looked at some consecutive frames and, on this base, we determined the different levels. In general, the determination of z_w and $z_{b,low}$ was relatively easy while $z_{b,up}$ could vary by a millimeter at most (depending on the quality of the frame).

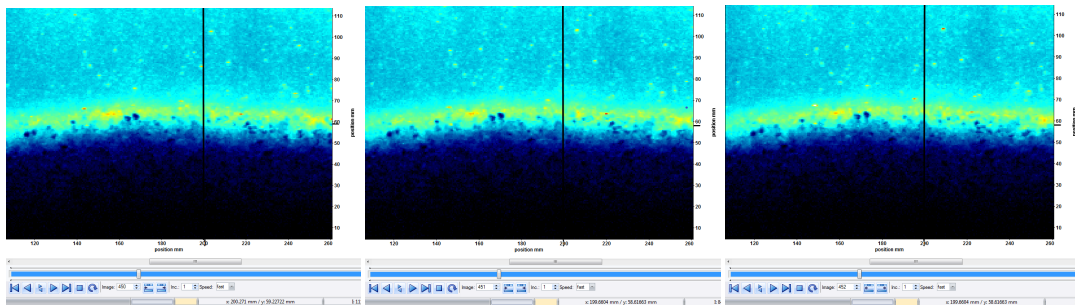


Figure 4.11: 3 consecutive frames from which we determined $z_{b,low}$ and $z_{b,up}$.

In figure 4.11, you can see the sand moving along the bed. Using a vertical line as a referential and using many frames, we were able to detect the moving from the non moving sand and the limit between the moving sand and the clear water layer. Obviously, figure 4.11 is just a sample. In reality, we used much more frames around the two that we analysed to be sure of the position of those levels. Moreover, there were sometimes sand grains in suspension that were detected by the Matlab code and that we did not take into account.

Chapter 5

Results and analysis

The goal of all data gathered with the experiment and the data processing is to have a better understanding of the sediment transport phenomenon and the water behaviour. We tried to link theoretical equations and methods exposed in the section 1.7 with the experimental results obtain by the data processing.

This chapter is the most important one and summarises all the data that we collected and all our interpretations about it.

Firstly, we will present the results from the Clauser method by fitting the theory (Sumer et al. equation) and the reality (experimental curve) together. Moreover, we will discuss more in details the Δz (and its relation with u_*) which seems to be the most sensitive parameter.

Secondly, we computed the bed shear stress and its corresponding shear velocity with the slope method using the data of our experiments.

Finally, we compared the two methods and tried to estimate the difference between them in order to apply a corrector coefficient to the Manning coefficient. We try to determine this corrector coefficient taking into account the following given condition: time, the size of sediment diameter and the water height.

5.1 Clauser method analysis

The first method we used is the Clauser method. As explained before, it is based on Sumer et al. velocity profile. We will briefly recall the later, then expose our analysis procedure and finally share our interpretations and conclusions as far as the method is concerned.

5.1.1 Velocity profile found by Sumer et al.

$$u = \frac{u_*}{\kappa} \ln(z - z_{b,low} - \Delta z) + u_* (B_r - \frac{1}{\kappa} \ln(k_s)) \quad (5.1)$$

The Sumer et al. velocity profile aimed to determine the shear velocity from different parameters. Some of them are fixed as the von Karman constant $\kappa = 0.407$ and the bed roughness $k_s = d_{50}$. Others vary with time and space like z and u . The $z_{b,low}$ must be determined on the experimental frames (section 4.2) and finally B_r and u_* are chosen by ourselves in order to fit with the experimental curve as you can see in figure 5.1 (Δz adapting automatically with the modification of u_*).

You can find the detail of each parameter and their variation in section 1.8.

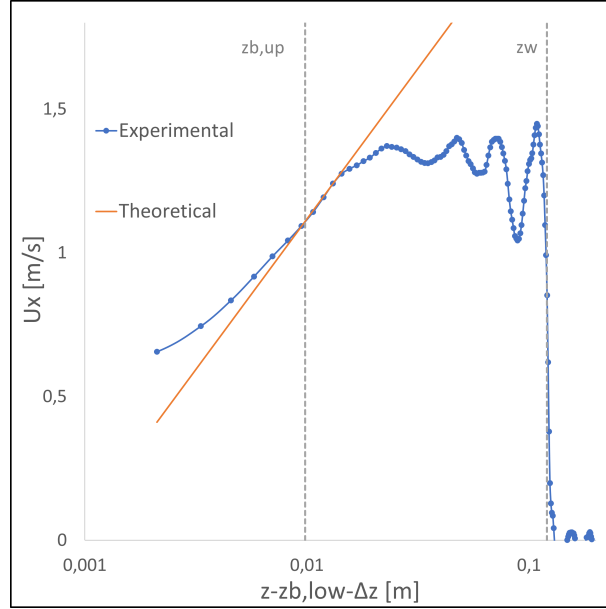


Figure 5.1: Fitted theoretical profile in orange on the logarithmic part of the experimental profile in blue dots.

You can find all of our work on fitting the experimental and the theoretical profiles for each initial conditions in Annex 2.

5.1.2 Relation $\Delta z - u_*$: Comparison between Sumer et al.(1996) and Lac and Gilmard (2020)

In order to fit the theoretical profile found by Sumer et al.(5.1) with the experimental profile, some parameters need to be defined. It was interesting to determine beforehand Δz in order to restrain the analysis to one degree of freedom to find the correct slope and thus the corresponding u_* value.

Therefore, Sumer et al. put in relation Δz and u_* as shown in figure 5.2:

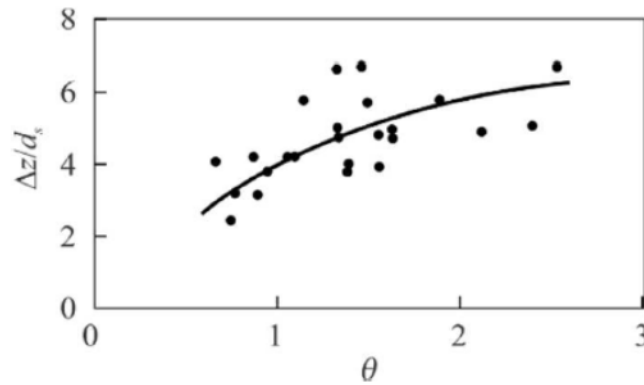


Figure 5.2: Level of the theoretical bed Δz as a function of θ .

Reference: Ilaria Fent's thesis.

With $\theta = \frac{\rho u_*^2}{g(\rho_s - \rho)d}$.

By making a logarithmic approximation on the points (solid line in figure 5.2), they found the following equation (5.2) :

$$\frac{\Delta z}{d_{50}} = 0.9937 \ln(\theta) + 6.0996 \quad (5.2)$$

But this equation is valid in a case of steady flow over mobile bed as explained in section 2.3.

Actually, Lac and Gilmard added their experiment results found with the slope method to a graph with same axis than the graph 5.2, and they found another curve. We can see on the figure 5.3 that the couple $\frac{\Delta z}{d_{50}} - \theta$ they found with the slope method does not really have the same trend as the couple found by Sumer et al. The trend seems rather to be the opposite:

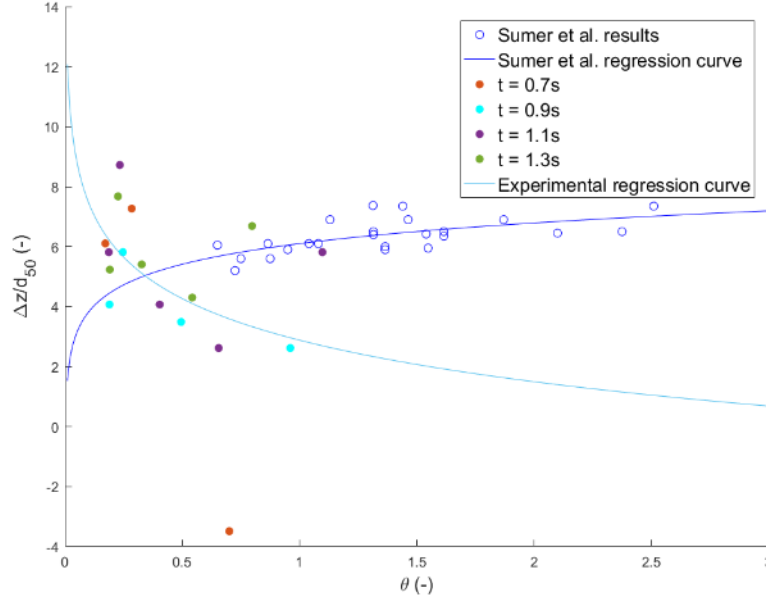


Figure 5.3: Experimental level of the theoretical bed Δz as a function of θ +
Results of Sumer et al. (1996).

Reference : Adlane Lac and Henri Gilmard's Thesis, 2018 [13].

Using a logarithm regression, they found the next equation:

$$\frac{\Delta z}{d_{50}} = -1.998 \ln(\theta) + 2.8804 \quad (5.3)$$

They concluded that this equation could be more specific to the dam-break flow. We analysed our own results using both equations and found better results with Lac and Gilmard equation up to a certain point (We will develop this later in the chapter).

The Δz corresponds to the theoretical level from $z_{b,low}$ towards which the velocity logarithmic law tends asymptotically as seen in figure 5.4, so, in order to follow the literature, Δz should never exceed the mobile sediment layer height. Sumer et al. equation tends to overestimate the Δz in a case of a dam-break event.

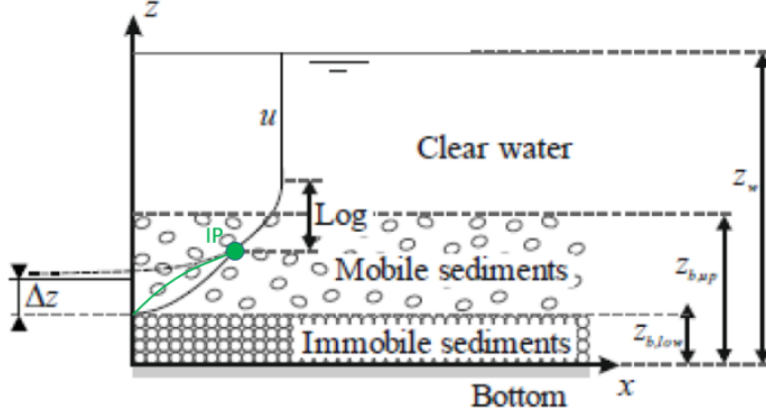


Figure 5.4: Definition of variables by Sumer et al. (1996), for a velocity distribution with logarithmic law near bed in presence of a sheet-flow layer.
Reference : Thesis of Ilaria Fent, 2018 [9].

So, most of our analysis will be based on Lac and Gilmard equation. It gave coherent results for most shear velocities u_* . However, when it became too small (in a case of small water height for example) we obtained results too close to the vertical asymptote ($\theta < 0.5$ in figure 5.3). With the curve of Adlane Lac and Henri Gilmard, the Δz is thus too big (higher than the $z_{b,up}$) and we could not use the equation.

5.1.3 Analysis procedure for the Clauser method

In order to extract the bed shear stress from the experiment results, we needed to fit the theoretical velocity profile onto the experiment points in a semi-logarithmic scale (see figures 5.5 and 5.6). Again it is important to define the reference level at $z_{b,low} + \Delta z$ because shifting horizontally a logarithm function in a standard axis system (with the parameter Δz , for example) is not equivalent to the horizontal shift of a linear function on a semi-logarithmic axis-system (see section 1.8).

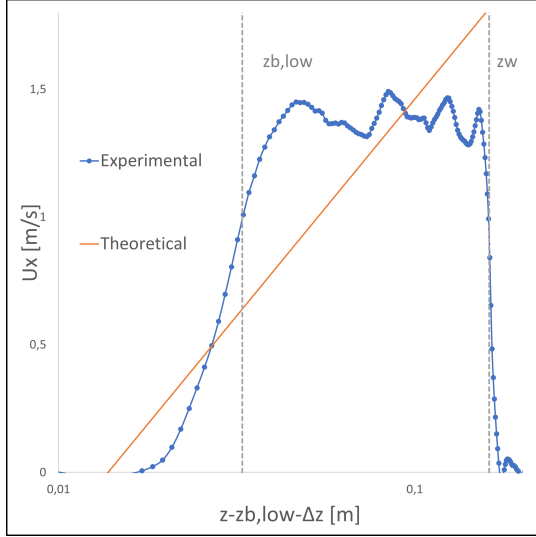


Figure 5.5: unfitted Curve at $t=0.9s$ for $H = 0.35m$.

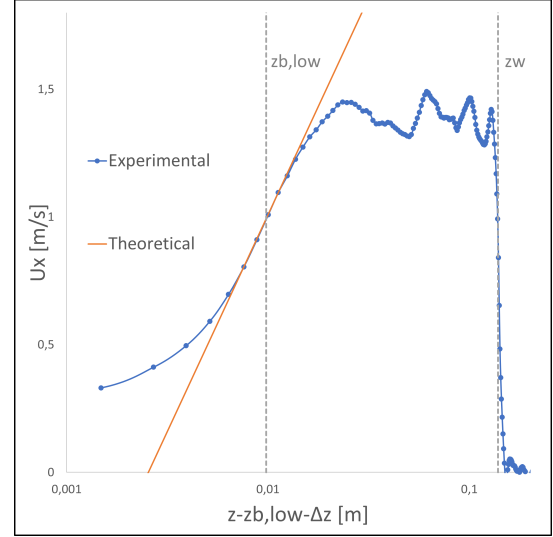


Figure 5.6: fitted Curve at $t=0.9s$ for $H = 0.35m$.

The first step is to check the repeatability as explained in section 4.1.5. Some samples could have been corrupted during the experiment. Thus before analysing a sample we need to be sure its velocity profile fits with the other samples with the same initial conditions. This shows the importance to have at least a few samples of the same dam-break experiment.

Then we can start to make a correlation between the theoretical profile and the experimental one by manipulating the graphs.

The way used to fit those curves together is to find the right angle by gradually increasing or decreasing the shear velocity u_* until the slopes match, then modifying B_r to catch the abscissae intersection.

This process must be done with hindsight. It is clear that for the same sample, more than one u_* can fit the log profile so the entire process needs more thoughts. For a value of u_* to be acceptable, it needs to fulfil a few criteria:

- The slopes need to match.
- The same value can be observed in different samples with the same conditions (Height, sand diameter, abscissa, times, etc).

- It needs to be coherent within certain time steps (For the same sample, u_* should evolve progressively along time and can not differ much from its value 0.3s before or after).
- The shear velocity can not lead to incoherent bed shear stress values.

Once the shear velocity is determined for given conditions, we can calculate the bed shear stress with the Clauser method as explained in section 1.7.1 with the equation (5.4):

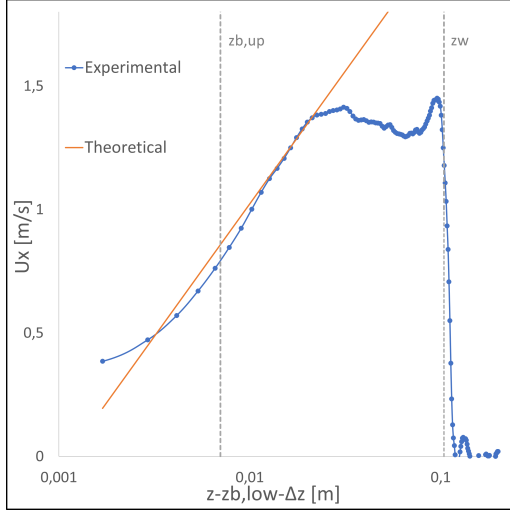
$$\tau_b = \rho u_*^2 \quad (5.4)$$

This value of τ_b is linked to a certain sand, water height and position and is ready to be compared with the other method.

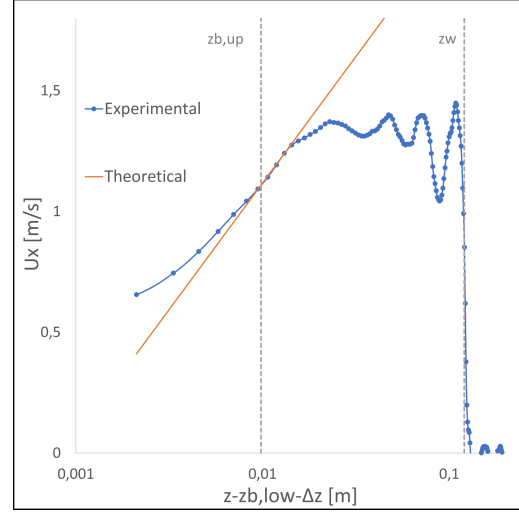
5.1.4 Results and conclusion of the Clauser method

The procedure described above provided us the shear velocity u_* and the bed shear stress τ_b for given conditions. In this section we will focus on the influences of these conditions over the Sumer et al. fitting procedure.

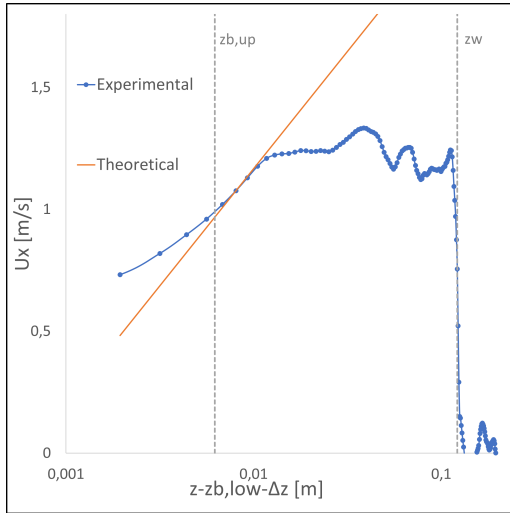
As explained, we wanted to see the effect of the water height and the sediment diameter. We took as reference an experiment with a water height equal to 0.30m and a sediment diameter of 0.00183m over 4 time steps: 0.6, 0.9, 1.2 and 1.5 seconds. This sample is shown on the figures 5.7.



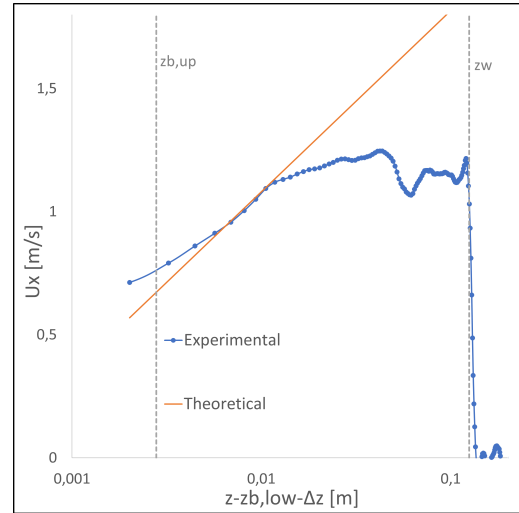
(a) $H=30\text{cm}$, $t=0.6\text{s}$ and $d=200\text{mm}$



(b) $H=30\text{cm}$, $t=0.9\text{s}$ and $d=200\text{mm}$



(c) $H=30\text{cm}$, $t=1.2\text{s}$ and $d=200\text{mm}$



(d) $H=30\text{cm}$, $t=1.5\text{s}$ and $d=200\text{mm}$

Figure 5.7: Experiment results for height $H=30\text{cm}$, sand diameter = 1.83mm and $d=200\text{mm}$.

We replicated this sample 3 times in the same conditions. We extracted the values for u_* , τ_b , Δz and B_r for each time step (shown in table 5.1). With those three experiments, we managed to identify some tendencies as far as the shear is concerned.

Sand 1: $d=1.83\text{mm}$ Height = 30cm $d = 200\text{mm}$	Time[s]	0.6	0.9	1.05	1.2	1.35	1.5
Experiment 1	$u_*[m/s]$	0.4	0.183	0.175	0.17	0.173	0.17
	$\tau_b[N/m^2]$	160	33.48	30.62	28.9	30.05	28.9
	$\Delta z[mm]$	-0.87	3.75	3.75	5.283	4.14	4.28
	$B_r[/math>$	-2	1.95	2.55	2.55	1.9	2.1
Experiment 2	$u_*[m/s]$	0.19	0.185	0.175	0.17	0.198	0.13
	$\tau_b[N/m^2]$	36.61	34.225	30.625	29.8	39.204	16.9
	$\Delta z[mm]$	4.483	4.674	5.074	5.283	5.186	7.21
	$B_r[/math>$	1.2	1.83	2.3	2.65	1.8	4.1
Experiment 3	$u_*[m/s]$	0.21	0.188	0.178	0.15	0.14	0.14
	$\tau_b[N/m^2]$	44.1	35.306	31.68	22.5	19.6	19.6
	$\Delta z[mm]$	2.66	3.26	4	5.083	6.679	6.679
	$B_r[/math>$	2.4	2.2	2.2	4	3.8	3.8

Table 5.1: Data for sand 1 $d_{50} = 1.83\text{mm}$, height $H = 0.30\text{m}$ and distance from gate $d = 200\text{mm}$.

The shear velocity (for a time range $t=0.9\text{s} - t=1.5\text{s}$) appears to vary from around 0.188 to 0.13 m/s and the bed shear stresses corresponding vary from around 35.306 to 16.9 N/m^2 for a sand diameter $d_{50} = 1.83\text{mm}$ and an initial water height $H = 0.30\text{m}$. When looking at figure 5.8, we realise that the bed shear stress (and therefore its corresponding shear velocity) tends to decrease constantly with time as the water column collapses.

The values taken before 0.9 seconds appear to be very different from an experiment to another. This is explained by the fact that those values are taken right after the gate opens. The water falls down from its initial position leading to important vertical components in the velocity profile. So the values taken too early in time tend to be distorted. This is why we do not use them for our analyses of the Clauser method.

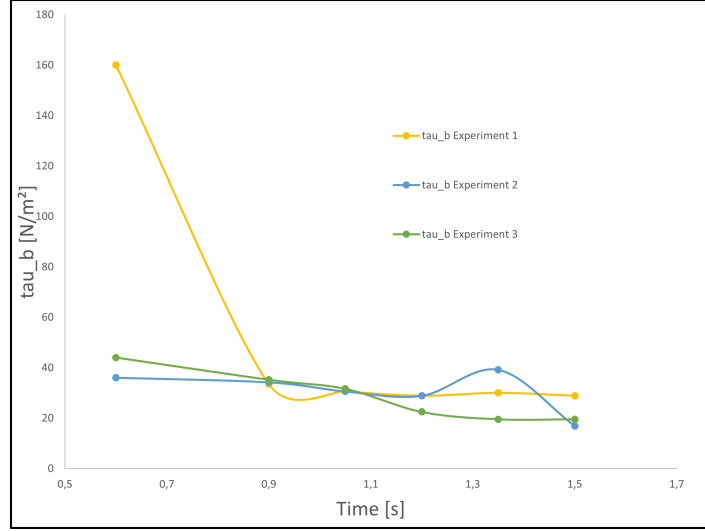


Figure 5.8: τ_b as a function of time at $d=200\text{mm}$ from the door as determined by the slope method

We can conclude with this first analysis that the bed shear stress is still hard to define at the start of a dam-break event due to uncertain behaviour of the flow but it settles to a certain value and slowly decreases over time. On the other hand, the B_r parameter seems to vary in an inversely proportional way to the shear velocity. As u_* decreases, B_r rises in response.

We will now use this first analysis to have an idea of the influence of our parameters in the following.

About the distance from the dam

We want to verify if the tendency observed for a distance $d = 200\text{mm}$ from the dam is still observable for other distances. So, from the same experiments, we extracted the value of u_* , τ_b , Δz and B_r for the same time step. We summarized them in the table 5.2 in order to observe and interpret the effect of the distance.

Sand 1: d=1.83mm Height = 30cm Experiment 1	Time[s]	0.6	0.9	1.05	1.2	1.35	1.5
d=150mm	$u_*[m/s]$	0.16	0.176	0.177	0.171	0.168	0.15
	$\tau_b[N/m^2]$	25.6	30.97	31.32	29.241	28.224	22.5
	$\Delta z[mm]$	4.719	3.733	4.993	5.24	4.86	6.183
	$B_r[/]$	3	1.9	2.48	1	0.8	1.5
d=200mm	$u_*[m/s]$	0.4	0.183	0.175	0.17	0.173	0.17
	$\tau_b[N/m^2]$	160	33.48	30.62	28.9	30.05	28.9
	$\Delta z[mm]$	-0.87	3.75	3.75	5.283	4.14	4.28
	$B_r[/]$	-2	1.95	2.55	2.55	1.9	2.1
d=250mm	$u_*[m/s]$	0.167	0.178	0.182	0.172	0.128	0.14
	$\tau_b[N/m^2]$	27.89	31.64	33.124	29.58	24.964	19.6
	$\Delta z[mm]$	5.411	3.65	4.762	5.199	4.81	5.479
	$B_r[/]$	3.1	1.8	1.89	2.45	2.75	3.54

Table 5.2: Data for sand 1 $d_{50} = 1.83mm$, height $H = 0.30m$ and distance from gate $d = 150mm, 200mm$ and $250mm$.

We notice that the experiment for all the distances have the same behaviour. u_* decrease with the time when B_r increase. So, for the rest of our analysis, we will hence be using the distance $d = 200mm$ which is right in front of the camera and right under the laser.

Effect of the variation of the initial water height

The first parameter we investigated is the initial water height. We took three different ones: $H=0.35m$, $H=0.30m$ (the reference) and $H=0.25m$.

The results for $H=0.35m$ are shown in the table 5.3. We can see that the values for u_* and τ_b rise in the first centiseconds then, they tend to slightly decrease over time similarly to the $H=0.30m$ case. However, we can notice that those values are marginally higher than this previous case. The shear velocity goes from 0.217 to 0.14 m/s and the corresponding bed shear stress is between 47.049 and 19.6 N/m^2 .

Sand 1: $d=1.83\text{mm}$ Height = 35cm $d = 200\text{mm}$	Time[s]	0.6	0.9	1.2	1.5
Experiment 1	$u_*[m/s]$	0.2	0.217	0.2	0.19
	$\tau_b[N/m^2]$	40	47.089	40	36.1
	$\Delta z[mm]$	4.11	3.52	4.11	4.48
	$B_r[/math>$	3.1	1.1	0.85	0.9
Experiment 2	$u_*[m/s]$	0.17	0.2	0.189	0.09
	$\tau_b[N/m^2]$	28.9	40	35.72	8.1
	$\Delta z[mm]$	5.28	4.11	4.52	8.65
	$B_r[/math>$	3.7	1.98	1.8	8.5
Experiment 3	$u_*[m/s]$	0.165	0.2	0.171	0.14
	$\tau_b[N/m^2]$	27.22	40	29.24	19.6
	$\Delta z[mm]$	5	4.11	4.04	5.67
	$B_r[/math>$	4.17	2.9	2.6	4.8
Experiment 4	$u_*[m/s]$	0.16	0.185	0.163	0.14
	$\tau_b[N/m^2]$	25.6	34.225	26.57	19.6
	$\Delta z[mm]$	5.11	4.17	4.78	6.17
	$B_r[/math>$	4.7	2.7	3.9	6.17

Table 5.3: Data for sand 1 $d_{50} = 1.83\text{mm}$, height $H = 0.35\text{m}$ and distance from gate $d = 200\text{mm}$.

The values extracted for $H=0.25\text{m}$ were more difficult: since the shear velocity rises with the water height, it should decrease in response to this case. However, this small shear velocity led to a small θ as explained in subsection 5.1.2. Thus, we found ourselves in a case close to the asymptotic part of Gilmard and Lac equation (see figure 5.3) leading to incoherent values with a Δz leaving the mobile sand layer. We decided then to use Sumer et al. equation (see equation (5.2)). It gave more coherent results as far as the literature is concerned but it pushed them towards steady flow results.

Those values can be seen in table 5.4. The shear velocity stays between 0.09 and 0.076 m/s while the bed shear stress stays in the range of 8.1 to 5.775 N/m^2 .

Sand 1: $d=1.83\text{mm}$ Height = 25cm $d = 200\text{mm}$	Time[s]	0.6	0.9	1.2	1.5
Experiment 1	$u_*[m/s]$	0.085	0.09	0.085	0.08
	$\tau_b[N/m^2]$	7.22	8.1	7.225	6.4
	$\Delta z[mm]$	8.85	8.65	8.45	8.23
	$B_r[/math>$	12	10	9.5	7.5
Experiment 2	$u_*[m/s]$	0.1	0.085	0.08	0.09
	$\tau_b[N/m^2]$	10	7.225	6.4	8.1
	$\Delta z[mm]$	9.03	7.45	8.23	8.65
	$B_r[/math>$	10.5	10.5	10.2	7.7
Experiment 3	$u_*[m/s]$	0.11	0.085	0.081	0.076
	$\tau_b[N/m^2]$	12.1	7.25	6.56	5.775
	$\Delta z[mm]$	8.41	7.45	8.278	8.05
	$B_r[/math>$	8.5	9.9	9	7.88

Table 5.4: Data for sand 1 $d_{50} = 1.83\text{mm}$, height $H = 0.25\text{m}$ and distance from gate $d = 200\text{mm}$.

The values seem to be decreasing over time as well but it is not as prominent as in the two other cases. Also, we can notice how small they are compared to the reference case $H=0.30\text{m}$. This could mean that a small shear velocity leads to an edge case where none of the $\Delta z - u_*$ relations determined so far correspond to reality.

For a more visual perspective of the initial water height influence, we compared on a graph each height cases for each time steps (see figure 5.9).

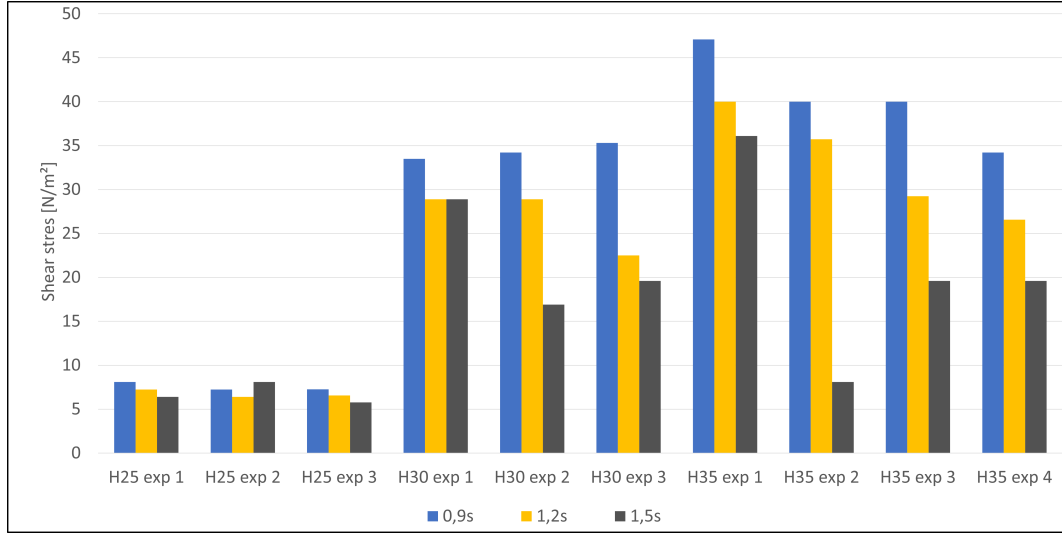


Figure 5.9: τ_b for different initial water heights as determined by the Clauser method.

As seen on the table 5.9, the variability of the results for $H = 0.35\text{m}$ is more accentuated. This comes from the procedure of analysis being non optimal when the time exceeded 1.2s. In all cases and it is more prominent for $H=0.35\text{m}$. Indeed, we sometimes could not find a very coherent matching slope so we have to take those results with hindsight.

The table 5.5 summarizes the maximum and minimum values of the shear stress for the different heights.

Sand 1 : $d=1.83\text{mm}$	$\tau_{b,max}$ [N/m ²]	$\tau_{b,min}$ [N/m ²]
H=0.25 [m]	8.1	5.775
H=0.30 [m]	35.306	16.9
H=0.35 [m]	47.049	19.6

Table 5.5: Maximum and minimum values of the shear stress for the different heights with a sand diameter $d=1.83\text{mm}$.

Effect of the standard deviation of the sand diameter

An element that is not a parameter by itself but that still deserves to be looked at, is the variance of the sediment diameter. As explained in subsection 3.5.4, our sand reference has a standard deviation $\sigma = 0.138\text{mm}$. We sieved a new

sand so that its standard deviation was equal to 0.02. We made three tests with this sand and found the following results (see table 5.6). We can observe that the shear velocity varies from 1.85 to 1.48 m/s and the bed shear stress from 33.67 to 21.90 N/m^2 .

Sand 2: d=1.83mm Height = 30cm d = 200mm	Time[s]	0.6	0.9	1.2	1.5
Experiment 1	$u_*[m/s]$	0.178	0.184	0.17	0.17
	$\tau_b[N/m^2]$	31.684	33.756	28.9	28.9
	$\Delta z[mm]$	4.95	3.713	5.28	4.28
	$B_r[/]$	3	1.95	2.1	1.6
Experiment 2	$u_*[m/s]$	0.185	0.1835	0.174	0.148
	$\tau_b[N/m^2]$	34.225	33.67	30.276	21.904
	$\Delta z[mm]$	4.67	4.73	5.115	6.27
	$B_r[/]$	3	2.5	2.4	3.3
Experiment 3	$u_*[m/s]$	0.198	0.185	0.17	0.18
	$\tau_b[N/m^2]$	39.204	34.225	28.9	32.4
	$\Delta z[mm]$	5.38	2.67	5.28	4.27
	$B_r[/]$	1.55	1.8	2.9	1.4

Table 5.6: Data for sand $d_{50} = 1.83\text{mm}$ and $\sigma = 0.02$, height $H = 0.30\text{m}$ and distance from gate $d = 200\text{mm}$.

Those values appear to be very close to the reference case. However, we can observe that the range of the shear stress is a bit reduced, the maximum value passed from 35.306 to 33.67 N/m^2 and the minimum value passed from 16,9 to 21.9 N/m^2 . So, a smaller standard deviation for the sand diameter could give us little more accurate results. Despite this we conclude from the study of the standard deviation that this factor does not influence significantly the bed shear stress. The standard deviation $\sigma = 0.138$ is considered to be a reasonable value for our experimental analysis.

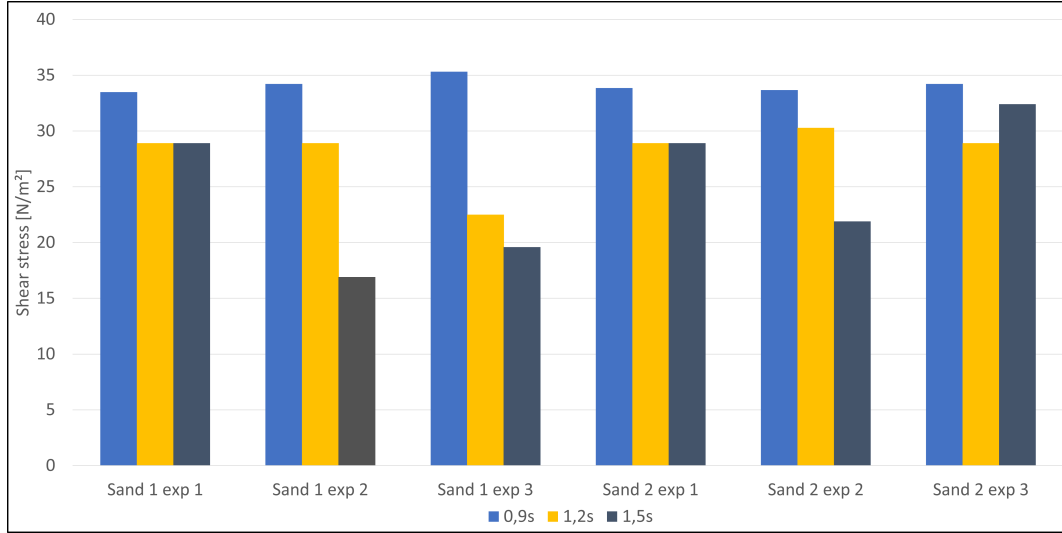


Figure 5.10: τ_b for different standard deviation of sand as determined by the Clauser method.

We can see on figure 5.10 that the results of the sand 1 is lightly more spread than the sand 2 but they are very similar.

Effect of the sediment diameter

The third parameter to be changed is the sand diameter. We took a smaller sand diameter of $d_{50} = 1.35mm$ in order to compare it to the first one $d_{50} = 1.83mm$.

This analysis showed an impressive variation in the results as highlighted by the table 5.7 (compare to table 5.1).

Sand 3: $d=1.35\text{mm}$ Height = 30cm $d = 200\text{mm}$	Time[s]	0.6	0.9	1.2	1.5
Experiment 1	$u_*[m/s]$	0.2	0.23	0.21	0.2
	$\tau_b[N/m^2]$	40	52.9	44.1	40
	$\Delta z[mm]$	2.309	1.55	2.046	2.3
	$B_r[/math>$	1.1	0.65	0.19	0.25
Experiment 2	$u_*[m/s]$	0.2	0.215	0.253	0.2
	$\tau_b[N/m^2]$	40	46.225	64.01	40
	$\Delta z[mm]$	2.3	1.91	1.04	2.3
	$B_r[/math>$	2.5	0.7	-1.05	0.25

Table 5.7: Data for sand $d_{50} = 1.35\text{mm}$, height $H = 0.30\text{m}$ and distance from gate $d = 200\text{mm}$.

The shear velocity lays between 0.253 and 0.2 m/s and its corresponding bed shear stress is between 64.01 and 40 N/m^2 . Those values are far higher than the reference case. Moreover, it appears that the shear velocity takes more time to settle with this sand diameter. In conclusion, the sediment average diameter has a major influence on the water velocity profile. With the data we collected, we can advance that a small sediment diameter induces a significant rise of the shear velocity.

The figure 5.11 illustrate this difference.

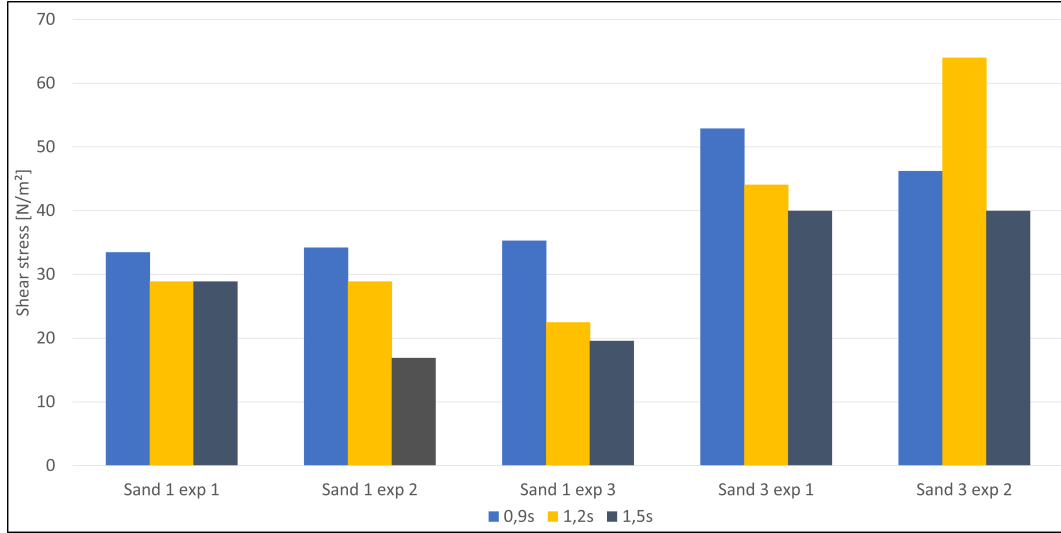


Figure 5.11: τ_b for different diameters of sand as determined by the Clauser method.

The table 5.8 summarizes the maximum and minimum values of the shear stress for different sand sizes with a same initial water height $H=30\text{cm}$

height: H30	$\tau_{b,max}$ [N/m²]	$\tau_{b,min}$ [N/m²]
Sand 1: d=1.83mm [m]	35.306	16.9
Sand 3: d=1.35mm [m]	64.01	40

Table 5.8: Maximum and minimum values of the shear stress for the different sand sizes with a same initial water height $H=30\text{cm}$.

Conclusion of the Clauser Method

The Clauser method seems to be reliable for a dam-break analysis when the data are taken not too close to the gate and not too early after this gate opens. The method also seems to give constant results around $t=0.9\text{s}$ and $t=1.2\text{s}$. For the time $t = 1.5\text{s}$ some of the samples water velocity seem to drop earlier than others.

We can explain it by the fact that the repeatability is less precise with time. The lightly errors can amplify with the time and create a larger difference between the experiments

Also, the results seem to vary as the sand diameter is modified which proves that it takes into account the sediment properties. The initial water height influence seems to manifest itself in the results as well, which is promising.

However, a small shear velocity seems to lead to edge cases that can not be correctly analysed. It also depends on a critical and sensitive relation between Δz and u_* . For more precise analysis, this relation would need deeper work in the future.

5.2 Slope method

The slope method initially presented in subsection 1.7.2 will be used in this section to compute the bed shear stress τ_b and the shear velocity.

We give our interpretations about these results and try to determine the effect of the experimental parameters that we chose to study.

Later, we will compare those results with those obtained by the Clauser method and the Δz of Adlane Lac and Henri Gilmard.

As a reminder, here are the following assumptions made before:

- The equation to compute τ_b :

$$\tau_b = \gamma R S_f \quad (5.5)$$

where $\gamma = 9810 N/m^3$, R is the hydraulic radius and S_f is the friction slope.

- As developed in section 1.5 the hydraulic radius R will be equal to h , the height between z_w and $z_{b,low}$ respectively the water free surface and the fix sediment layer height.
- The friction slope S_f will be found using the Manning equation (5.6):

$$S_{f,Manning} = n^2 U^2 R^{-4/3} \quad (5.6)$$

Where U is the average velocity along the vertical and n is the Manning coefficient and is equal to $0.0165 s/m^{1/3}$ according to Illaria Fent's work. We choose to use $S_{f,Manning}$ as Adlane Lac and Henri Gilmard's work on the different S_f suggests.

- The equation (5.5) becomes then:

$$\tau_b = \gamma h S_{f,Manning} \quad (5.7)$$

5.2.1 Analysis procedure for slope method equation

Starting from the equation (5.7), we need :

- γ which is assumed to be constant and equal to $9810N/m^3$
- h the height that we extract from the experiment data [m].
- $S_{f,Manning}$ that itself needs:
 - n the Manning coefficient, found by Illaria Fent [$s/m^{1/3}$].
 - U the average velocity with respect to the vertical that we extract from the experiment data [m/s].

Now that we have the bed shear stress τ_b found with the slope method, we can compute the shear velocity by isolating it in the equation (1.25):

$$\tau_b = \rho u_*^2 \quad (5.8)$$

$$u_* = \sqrt{\frac{\tau_b}{\rho}} \quad (5.9)$$

5.2.2 Results and conclusions for the Slope method

The slope method does not take into account, as much as the previous method does, the sediments properties or the shear velocity. It bases its calculations of the bed shear stress on Manning hypothesis only. The slope method has been proven to give good results for steady flows but here, we will see that the values of τ_b for the exact same experiments are quite different from what we just saw in the previous analysis with the Clauser method.

As for the Clauser method, we took the three experiments with $H=0.30m$ and $d_{50} = 1.83mm$ as reference to analyse the parameters influence on the method in order to be able to compare the two methods later. Here are the results for those 3 samples (see table 5.9).

Sand 1: d=1.83mm Height = 30cm d = 200mm	Time[s]	0.6	0.9	1.05	1.2	1.35	1.5
Experiment 1	$u_*[m/s]$	0.1054	0.092	0.088	0.0854	0.0837	0.083
	$\tau_b[N/m^2]$	11.114	8.568	7.75	7.29	7.011	6.918
Experiment 2	$u_*[m/s]$	0.101	0.096	0.0896	0.0887	0.088	0.84
	$\tau_b[N/m^2]$	10.202	9.276	8.034	7.876	7.75	7.071
Experiment 3	$u_*[m/s]$	0.119	0.0903	0.0892	0.0892	0.085	0.0846
	$\tau_b[N/m^2]$	14.077	8.165	7.953	7.964	7.268	7.162

Table 5.9: Data for sand $d_{50} = 1.83\text{mm}$, height $H = 0.30\text{m}$ and distance from gate $d = 200\text{mm}$.

As we can see, the bed shear stress varies from 11.077 to 6.918 N/m^2 . On the contrary to Clauser, this method does not have any trouble to find similar results right after the gate drops (around $t = 0.6\text{s}$) and the bed shear stress appears to diminish over time as we expected (see figure 5.12).

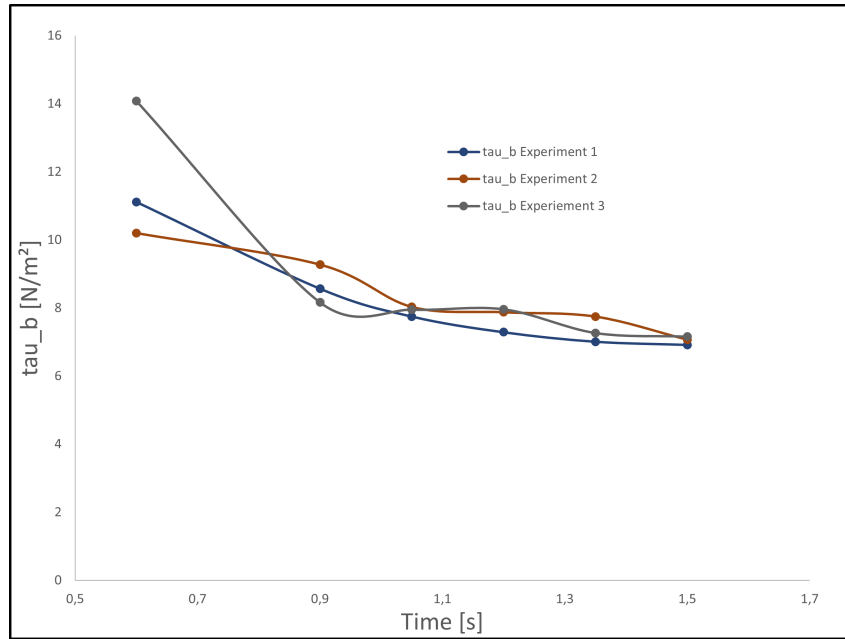


Figure 5.12: τ_b as a function of time at $d=200\text{mm}$ from the door as determined by the slope method.

Effect of the variation of the initial water height

For an initial water height of $H=0.35\text{m}$, the bed shear stress evolves from 12.58 to 7.292 N/m^2 as seen on the table 5.10. Those values are slightly higher than the reference case as we could have expected. However, these variations are quite minimal.

Sand 1: $d=1.83\text{mm}$ Height = 35cm $d = 200\text{mm}$	Time[s]	0.6	0.9	1.2	1.5
Experiment 1	$u_*[m/s]$	0.1122	0.0987	0.0907	0.0867
	$\tau_b[N/m^2]$	12.58	9.75	8.239	7.526
Experiment 2	$u_*[m/s]$	0.109	0.0959	0.089	0.086
	$\tau_b[N/m^2]$	12.027	9.204	8.011	7.4
Experiment 3	$u_*[m/s]$	0.103	0.095	0.088	0.085
	$\tau_b[N/m^2]$	10.66	9.052	7.85	7.292
Experiment 4	$u_*[m/s]$	0.106	0.0942	0.0905	0.082
	$\tau_b[N/m^2]$	11.354	8.876	8.204	6.87

Table 5.10: Data for sand $d_{50} = 1.83\text{mm}$, height $H = 0.35\text{m}$ and distance from gate $d = 200\text{mm}$.

Sand 1: $d=1.83\text{mm}$ Height = 25cm $d = 200\text{mm}$	Time[s]	0.6	0.9	1.2	1.5
Experiment 1	$u_*[m/s]$	0.1013	0.0899	0.0798	0.0764
	$\tau_b[N/m^2]$	10.278	8.096	6.37	5.84
Experiment 2	$u_*[m/s]$	0.105	0.0894	0.082	0.0783
	$\tau_b[N/m^2]$	11.049	8	6.82	6.132
Experiment 3	$u_*[m/s]$	0.1	0.0879	0.082	0.07629
	$\tau_b[N/m^2]$	10.08	7.73	6.69	5.821

Table 5.11: Data for sand $d_{50} = 1.83\text{mm}$, height $H = 0.25\text{m}$ and distance from gate $d = 200\text{mm}$.

When analysing the water height $H=0.25\text{m}$, we noticed a bed shear stress evolving from 11.049 to 5.82 N/m^2 . Those values are this time slightly lower than the reference case as seen on the table 5.11. Once again these variations are minor.

The initial water height directly influences the water velocity along the horizontal axis. The higher it is, the faster is the average velocity. The method seems to take this into account with our experiment. However, those changes are not always obvious or important. The figure 5.13 illustrates perfectly what we just said.

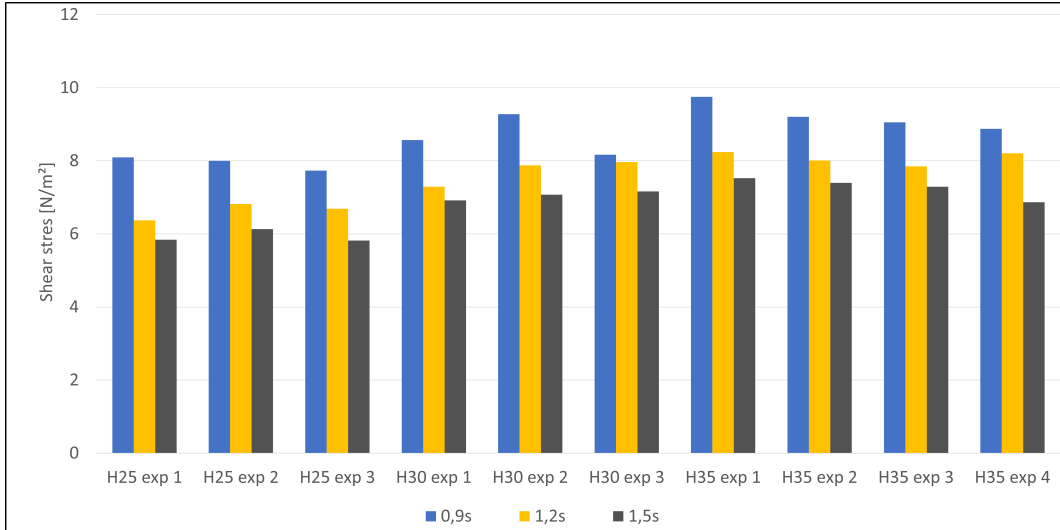


Figure 5.13: τ_b for different initial water heights as determined by the slope method.

Sand 1 : d=1.83mm	$\tau_{b,min} [\text{N/m}^2]$	$\tau_{b,min} [\text{N/m}^2]$
H=0.25 [m]	11.049	5.82
H=0.30 [m]	11.077	6.918
H=0.35 [m]	12.58	7.292

Table 5.12: Values of the maximum and minimum values of the shear stress for the different heights with a sand diameter $d=1.83\text{mm}$.

Effect of the variation of the sediment diameter

When looking at the slope method equations (5.6) and (5.7), we notice that the sediment properties are not present. Only the coefficient of Manning n is influenced by the bed but because the slope and the sediment material remained the same, we kept $n=0.0165 \text{ s}/\text{m}^{1/3}$ (In theory, the sediment diameter d_{50} should influence n , but the variation from 1.83mm to 1.35mm was not significant enough). We will verify if the change of this parameter still has an influence on the bed shear stress.

Using a sand with $d_{50} = 1.35\text{mm}$ gave the following results, see table 5.13. We can see that the bed shear stress vary from 12.3 to 7.41 N/m^2 .

Sand 3: $d=1.35\text{mm}$ Height = 30cm $d = 200\text{mm}$	Time[s]	0.6	0.9	1.2	1.5
Experiment 1	$u_*[m/s]$	0.109	0.102	0.09	0.086
	$\tau_b[N/m^2]$	11.805	10.49	8.11	7.51
Experiment 2	$u_*[m/s]$	0.11	0.093	0.087	0.086
	$\tau_b[N/m^2]$	12.304	8.69	7.673	7.419

Table 5.13: Data for sand $d_{50} = 1.35\text{mm}$, $H = 0.30\text{m}$ and distance from gate $d = 200\text{mm}$.

Those values are quite close to the reference case although the sediment average diameter was reduced by 26.2%. We can see it on the figures 5.14 and 5.15.

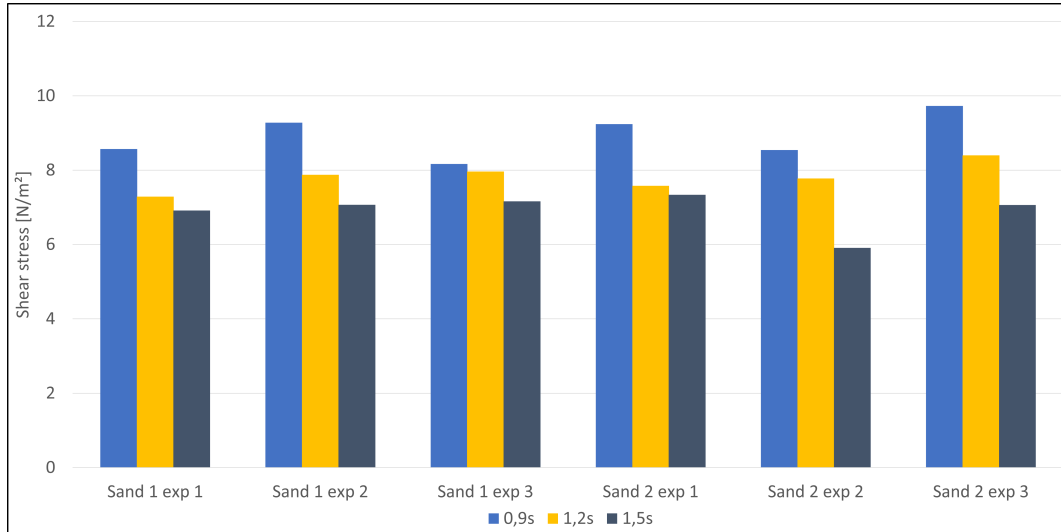


Figure 5.14: τ_b for different variances of sand as determined by the slope method.

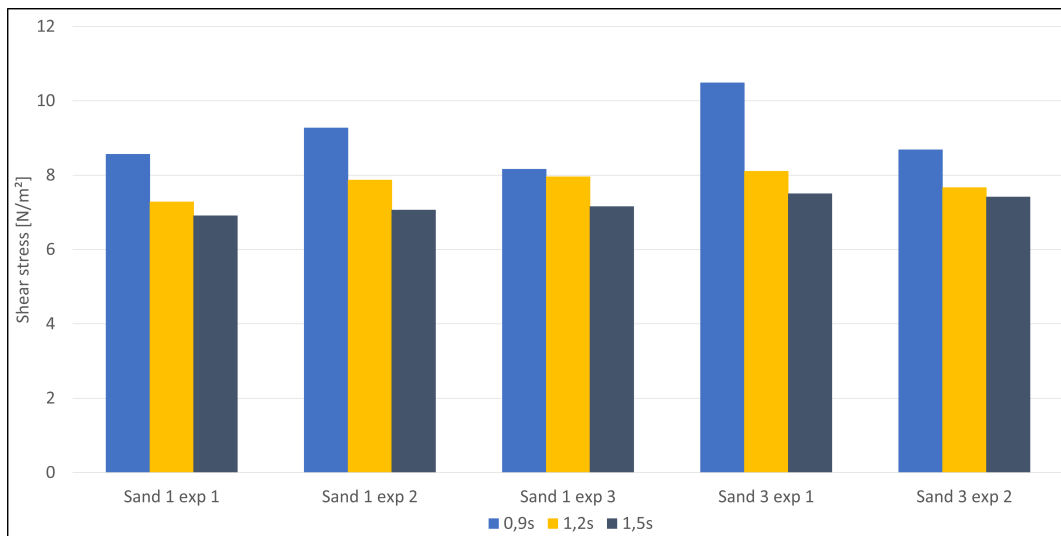


Figure 5.15: τ_b for different diameters of sand as determined by the slope method.

Taking into account the results obtained by the Clauser method, we can observe that the slope method has a flaw when taking into account the bed sediment.

Manning's coefficient needs to be adapted when his hypothesis is used in an unsteady flow over a mobile bed.

Conclusions of the Slope method

The slope method seems to be more adapted to steady flows. The bed shear stress τ_b seems to decrease over time as it should be. Nevertheless, it looks underestimated and does not vary much with the initial water height. Moreover, the fact that the sediment diameter does not influence its results is problematic.

The hypothesis of Manning are based on steady flow leading to noticeable mistakes in the analysis. An empirical coefficient taking the sediment diameter into account could be added to n .

5.3 Comparison of the two methods

Now that we have a bed shear stress calculated throughout graph manipulations of the Clauser method and a bed shear stress found with the slope method, we can compare these two methods to have a better understanding of the bed shear stress behaviour that occurs in a dam-break event.

The first part we can compare is the reference case that we have been using until now. We made a ratio $\frac{u_{*,ClauserMethod}}{u_{*,SlopeMethod}}$ of the shear velocities found by both methods for each sample (see figure 5.16).

The ratio varies from 1.54 to 2.025 and has an average of 1.91. This implies a ratio of the bed shear stresses going from 2.37 to 4.1. This difference is quite important, this put in evidence that one of the methods or both estimate wrongly the shear stress. Until now, we can not draw any conclusion.

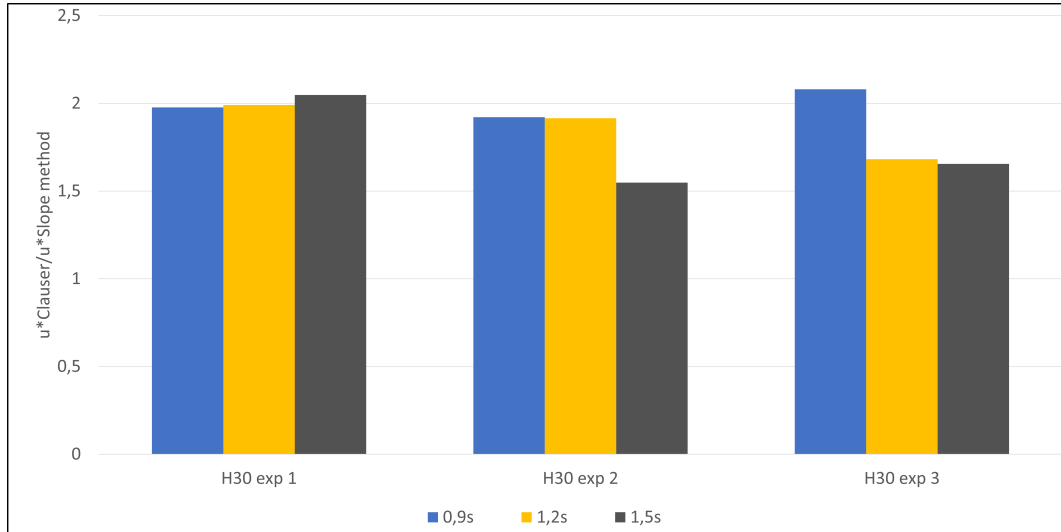


Figure 5.16: Ratio of the shear velocities found by the Clauser and the Slope method for H=0.30m.

For further comparisons, we took the same ratios for H=0.25m and H=0.35m as shown on figure 5.17.

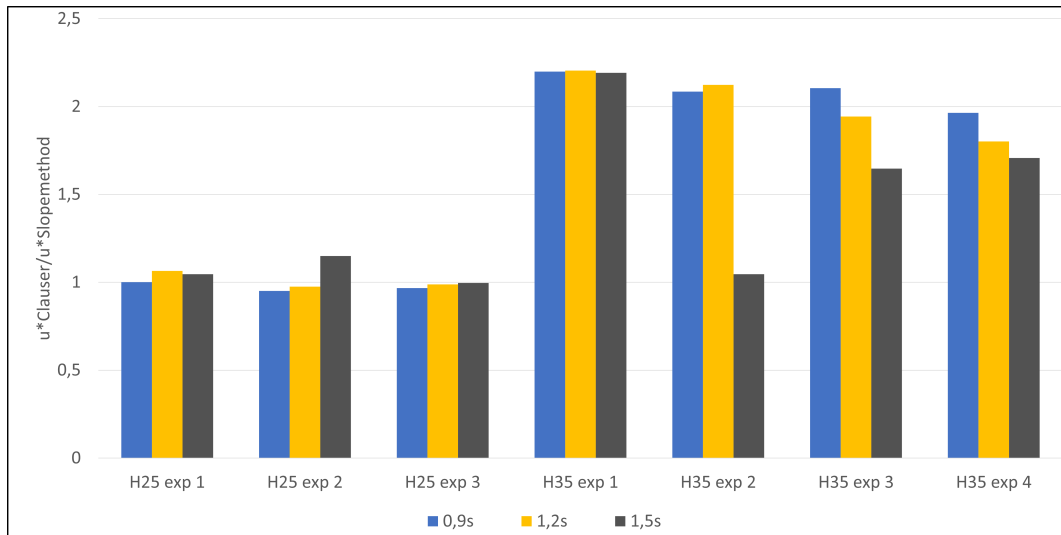


Figure 5.17: Ratio of the shear velocities found by the Clauser and the Slope method for H=0.25 and 0.35m.

For H=0.35m, the gap seems to be even more important with a ratio between

1.64 and 2.19 and an average of 2.04. This put in evidence that the ratio between the u_* is not constant in function of the height. Indeed, with an increasing height, the shear velocity obtained by the Clauser method increases a bit faster than the one obtained by the slope method. This leads to the conclusion that the slope method might not take the unsteadiness induced by the water height into account. $u_{*,Slope}$ rises because an higher water height induces an higher water velocity, but the unsteadiness factor seems to be ignored.

However, for $H=0.25m$, both methods give the same results with a constant ratio equal to 1. This could mean that the slope method is still adapted for low shear velocities induced by a lower water height. We should keep in mind that for those results, we used Sumer et al. equation (see equation (5.2)) instead of Lac and Gilmard equation (see (5.3)). This equation was made for steady flows over mobile bed. The fact that we used it for the Clauser method and that the low water height reduces the unsteadiness might explain why both methods gave the same results.

We already concluded that the Slope method does not seem to be influenced by the sediment diameter while the Clauser method gave higher shear velocities with the sand $d_{50} = 1.35mm$. We compared our results for this sand in the figure 5.18.

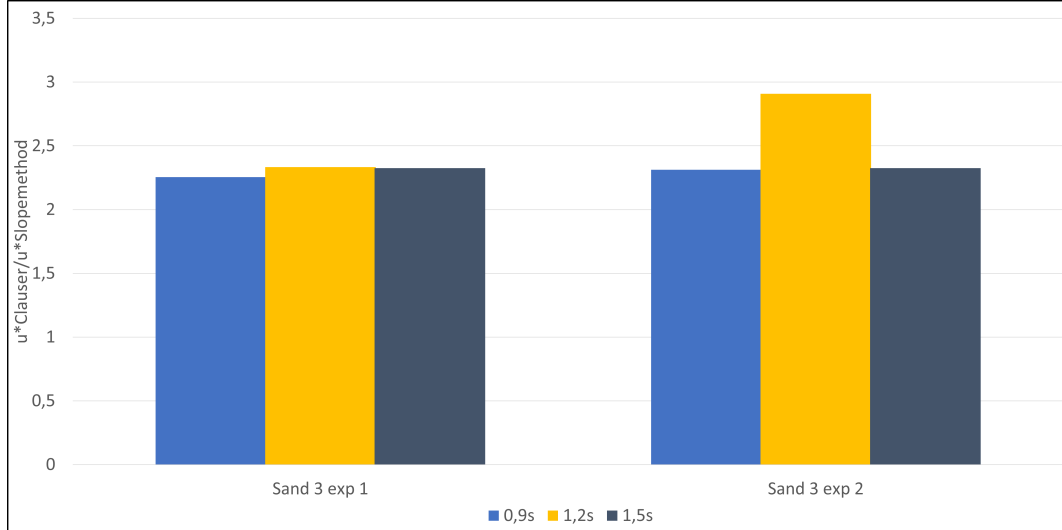


Figure 5.18: Ratio of the shear velocities found by the Clauser and the Slope method for a sand with $d_{50} = 1.35mm$.

As expected the ratio between the two methods is higher (between 2.25 and 2.34 and an average of 2.31). Obviously, it's because the shear velocity obtained by the Clauser method increase while the one obtained by the slope method do not change.

5.4 Experimental conclusions

The determination of the bed shear stress based on the velocity profiles went a step forward. We exposed the two methods and tried to specify their strengths and weaknesses, their effectiveness to find τ_b and their application limits.

The great amount of data we managed to collect this year throughout the experiments helped us to draw conclusion and to find tendencies in the general behaviour.

The Clauser method seems to be more adapted than the slope method for a dam-break event but the later could be adapted for such conditions.

5.4.1 Effectiveness of the methods for dam-break analysis

Clauser Method

The Clauser method give coherent results. It seemed to consider the sediment properties and the initial water height.

The unsteadiness of the flow has a direct impact on the shear velocity u_* , which lays at the base of the method procedure.

It appears that all the consequences of a dam failure have an impact on the results. This reinforced the conviction that the method is effective in our case.

However, the procedure has a few flaws. Firstly, it relies on a graph manipulation approach which might sometimes be subjective. Also, it happens that several values of u_* give a matching slope with the experimental curve. So it requires a subjective work to choose the most coherent value by looking at the entire sample.

In addition, some samples gave slopes that could not be matched with the experimental values throughout graph manipulations to obtain a good value for u_* especially in the first milliseconds of the event.

The procedure also needs a $\Delta z - u_*$ relation to be used. This relation is not well defined yet and does not work for small shear velocities as it happened for $H=0.25\text{m}$.

In conclusion, the method could be used to define a bed shear stress in a case of a dam-break event, but the procedure used need a great quantity of data to mitigate the mistakes caused by the subjective part of the analysis. It also does not work well in the early second or with a small shear velocity.

Slope Method

Unlike the Clauser method, the Slope method seems to have little to no edge cases. It does not require a subjective work to operate and gives constant results for any given time.

However, the bed shear stress seems to be underestimated when using the slope method. It gave results as if there was no sediment layer. This assumption verified itself when the change of sediment diameter had little to no impact on the results while the Clauser method was greatly influenced.

The slope method uses the coefficient of Manning n to determinate the the bed shear stress. For this method to be applicable to a dam-break event, it would need empirical coefficients adapted to the situation to correct n .

5.4.2 Analysis of the Manning coefficient n

We highlighted that the slope method did not give correct results but it had the right tendencies. The bed shear stresses increased with the initial water height and it decreased over time.

Because this method is easier to use than the Clauser method, we could apply an empirical coefficient to n as explained before so that both methods match values.

The slope method equations become then:

$$\tau_b = \gamma R S_f \quad (5.10)$$

$$S_{f,Manning} = (\lambda n)^2 U^2 R^{-4/3} \quad (5.11)$$

With λ [-] being the empirical coefficient. This coefficient would depend both on the initial water height and the sediment average diameter: $\lambda(H, d_{50})$.

For the experiments we made we could gather the value for λ for a few cases:

λ [-]	$d_{50} = 1.83$ [mm]	$d_{50} = 1.35$ [mm]
H=0.25 [m]	$\lambda \approx 1$	/
H=0.30 [m]	$\lambda \approx 1.91$	$\lambda \approx 2.31$
H=0.35 [m]	$\lambda \approx 2.04$	/

Table 5.14: Values for λ for each experimental cases.

More cases should be explored to give a better expression to λ .

5.4.3 Questioning the $\Delta z - u_*$ relation

The $\Delta z - u_*$ relation we used for most of our analysis was the equation (5.3) of Gilmard and Lac. They found this relation by calculating u_* throughout the slope method and then modifying Δz until the slope matched their experimental curve (see figure 5.19).

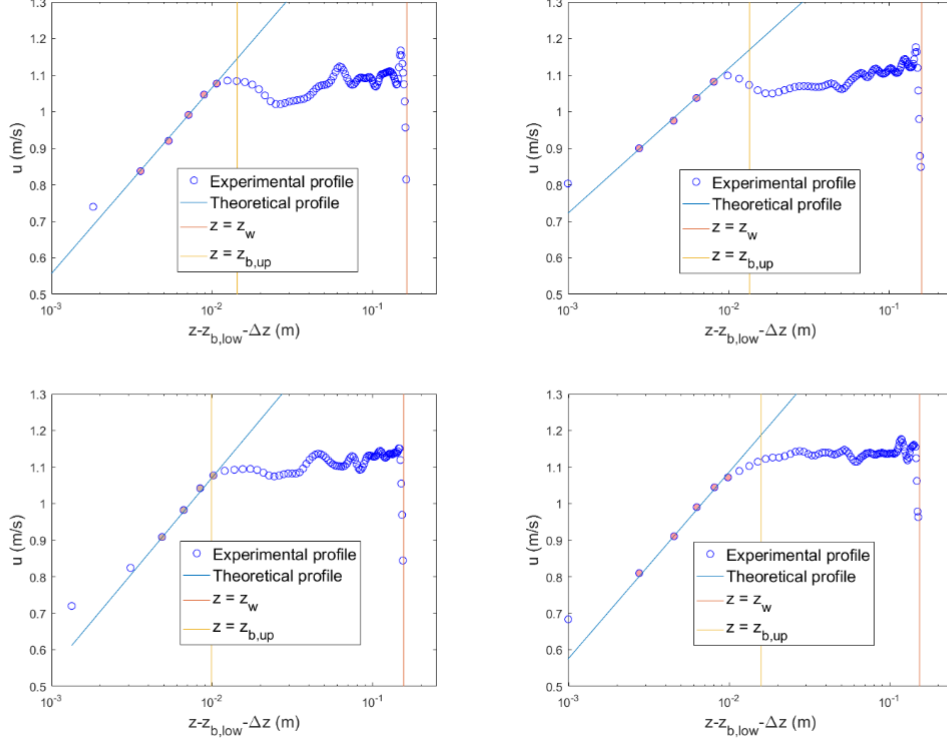


Figure 5.19: Theoretical profile fitting the experimental data.
Reference: Gilmard and Lac (2020). [13]

As seen during our analysis, the slope method tends to underestimate the actual shear velocity u_* . And the Δz corresponding becomes then larger. As a reminder, for the graph manipulation to work we have to set the reference level of the graph at $z_{b,low} + \Delta z$. Thus, having a Δz too important flattens the curves on the graphs up to a point that most of the points disappear.

The slope remains unchanged proportionally, but the lack of points may have led Gilmard and Lac to make a mistake.

Indeed, when looking at the literature, we found this: Illaria Fent found in her thesis that it was possible to calculate two bed shear stresses that act on the sediment layer surface (τ_s) and on the limit of the boundary layer (τ_w , water-water boundary). Those two bed shear stresses corresponded to two logarithmic parts that she observed on her experiments as seen on figure 5.20.

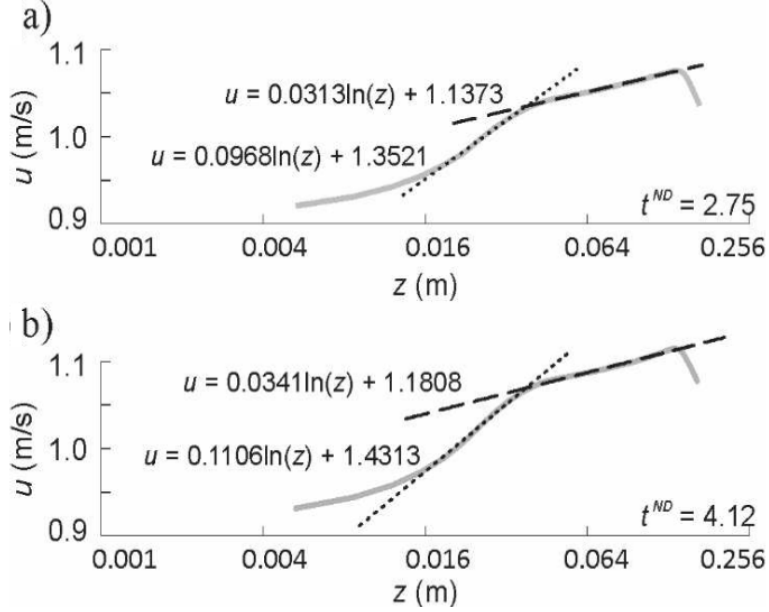


Figure 5.20: Shear stress on sediment layer τ_s and shear stress on the water-water boundary layer τ_w .
Reference: Illaria Fent (2018). [9]

Our analysis are aiming to determinate the bed shear stress τ_s in opposition to τ_w .

During our own experiments, most of our experimental curves presented those two logarithmic tendencies (see in Annex 2). The problem that Gilmard Lac faced was that, by flattening the curve too much with a large Δz , they matched the curve of the second shear stress τ_w .

The $\Delta z - u_*$ they deduced from this procedure contains thus a mistake because those two logarithmic parts are slightly different.

Although we used this relation for our analysis to find Δz , we did it with hindsight and had sometimes to correct it by 0.5 to 1 mm in order to have a matching slope.

The fact that we found coherent results shows that this error is minor but further researches as far as this relation is concerned should be done in the future.

Chapter 6

Conclusion

This master thesis was a little step towards the understanding of the phenomena that are dam-break events. The water velocity profiles appeared to have different tendencies comparing to regular flows and the actual behaviours seem to differ from the existing theory. We aimed to find better ways to define the bed shear stress to explain the reality behind the water-sediment layers interactions.

We contributed by many ways to this complex subject.

First of all, we improved the data processing. The process using our parameters gives more analysable data from the video and has more precise results available for the analysis. Thanks to this, we almost have twice the amount of points on our logarithmic lines. This involves a better fit of the theoretical curve on the experimental one. We propose, of course, to our successors to use all of our parameters when doing the data processing.

Secondly, we increased the number of experiments available for the study. We made no less than 28 new videos of the dam-break with different heights, different distances and different sands. Those videos and the results that we extracted from them could be used in the next thesis.

Thirdly, we computed the shear stress for numerous cases with the Clauser method and the slope method in order to compare them.

On the one hand, to compare them to the results that our predecessors found. We concluded that our results differed from theirs. Actually, we found (unfortunately lately) a mistake in their process. This was why they found similar results for the Clauser and the slope method.

On the other hand, to compare the results between them and draw some conclusions about them:

- The Clauser method gave a precise value to τ_b but it needed to be more deeply explored to give constant results.
- On the contrary, the slope method values were constant but slightly off. It would need to be corrected by an empirical factor applied on the Manning coefficient: λn . This correction would therefore need a large sample of experiments covering all the different cases possible.

Nevertheless, thanks to this methods, we discovered tendencies to the bed shear stress and its corresponding shear velocity. We concluded that τ_b increased when the initial water height was set higher. It appeared as well that the sediment properties played a significant role: taking a smaller sediment median diameter would increase the bed shear stress.

We also noticed that τ_d decreased over time, even right after the dam failure.

On a final notice, we discovered some facts as well as some dead ends and we would like to make a few suggestions to our potential successors. Those are the paths we wished to explore and that we think would be interesting to work on in order to contribute to the dam-break analysis:

1. The empirical coefficient λ we proposed to add to the slope method could be more deeply analysed. We propose to evaluate it, varying all kind of initial conditions in order to complete the table 5.14. The slope method gives constant results, so one experiment for each different case could be enough to have a good approximation for $\lambda(H, d_{50})$.
2. The Clauser method seems to give good results but still needs a lot of adaptation. The relation $\Delta z - u_*$ found by Sumer et al. did not work properly and the one found by Gilmard and Lac was a little off as explained in subsection 5.4.3. Researches for a better relation that suit the unsteadiness of a dam break event should be a first step.
3. During this thesis we varied the upstream water height and the sediment median diameter. Other parameters could influence the water velocity profiles and thus the bed shear stress. The characteristics of the sediment

like its density or its friction angle could be modified to study the changing behaviours.

4. Gilmard and Lac explored an automatic method to detect z_w , $z_{b,low}$ and $z_{b,up}$ respectively the water level, the lower level of the mobile sediment layer and the upper level of the mobile sediment layer. They were not convinced by this method and advised to opt for a manual detection. This manual detection is subject to the scale effect. So, it could be beneficial to research another way of detection. Make sure to read the thesis of Gilmard and Lac before.
5. We focused on two methods to calculate the bed shear stress because we wanted to analyse the effect of the variation of parameters. But the third one, the Reynolds Stress method was not explored and it could have advantages that the Clauser and the slope method did not have.

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Annex 1 : Operating mode for the dam break experiment

This annex explains the operating mode established by Ilaria Fent and completed by ourselves.

In fact, the operating mode made by Ilaria is already great but we think that it deserves someone to help at the good understanding of this last one in order to avoid a loss of time during the first steps of the master thesis.

This is the goal of this annex and it can be taken completely independently from the thesis.

Here are the steps to execute in order:

1. Turn on the switch at the control box so that the computer and the compressor start operating.
2. Place the high-speed camera on the previously marked spot. It is important for repeatability that it is always placed on the same exact place.
3. Connect the camera to the computer and start the PFV Fastcam software.
4. Install the laser above the studied spot. Keep in mind that it should be place on the same exact place for every experiment as well. The laser sheet should be parallel to the channel glass window and 3cm from it.
5. If it is the first set of experiment with the camera at the place, you need to calibrate it. This step should be done very carefully as it will be used for all your data. If the camera and the laser are conscientiously installed you only need one calibration. The calibration should be done under water so you need to use another gate to block the exit and fill it with water. Place the calibrating plate vertically right under the laser sheet (you may use a level for this step). Be careful to keep the stripes horizontal (see

figure 6.1). Then one can take a picture of the calibrating plate while the other one holds it. The lights should be on and you should clearly see all the white dots on your computer screen. This is a long process but if well done, it only needs to be done once.

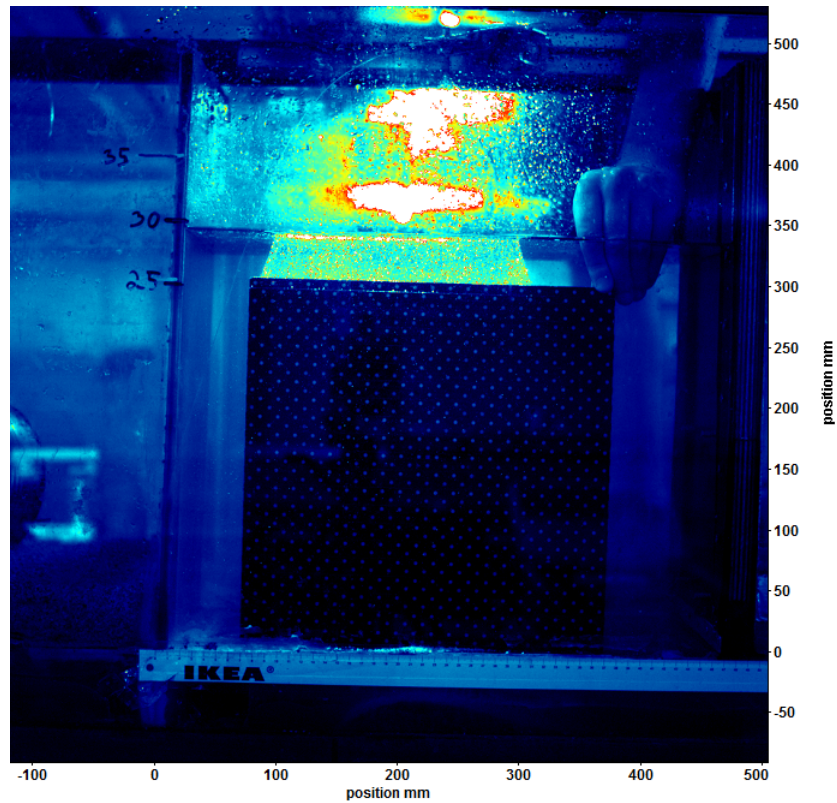


Figure 6.1: Calibration plate under water.

6. Place the sand you want to make your experiment with inside the channel. The layer height should be constant all long the channel.
7. Compact the sand with the mass system shown in figure 6.2.



Figure 6.2: Compacting mass system.

The compaction of the sand regularises the sand layer and accentuates the repeatability.

8. Fill the upstream part of the channel with the water from the blue container. If the container is empty, or if you don't have enough water, you can take it from the bigger canal. Be careful to fill it using the punctured tube (see figure 6.3) to prevent the water from digging in the sand layer.



Figure 6.3: Punctured tube.

9. Prepare the tracer solution. The tracer is the pliolite. It should be sieved with a $475\ \mu\text{m}$ sieve to have a perfect granulometry. Then mixed it with soap so it does not float due to surface tension. Pour your solution in the water and mix it for homogeneous distribution. For a correct seeding, try to have a particle density somewhere between the two figures: 6.4 and 6.5.

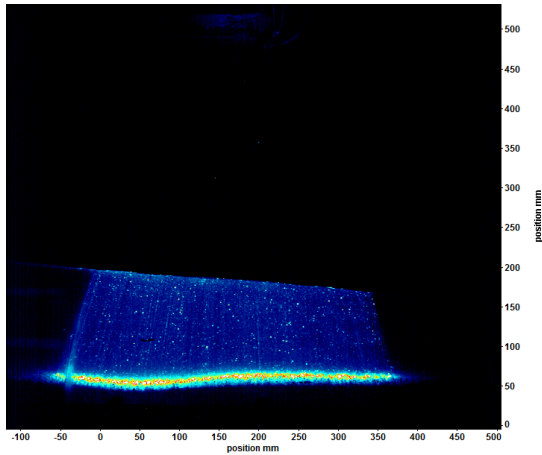


Figure 6.4: Poor seeding.

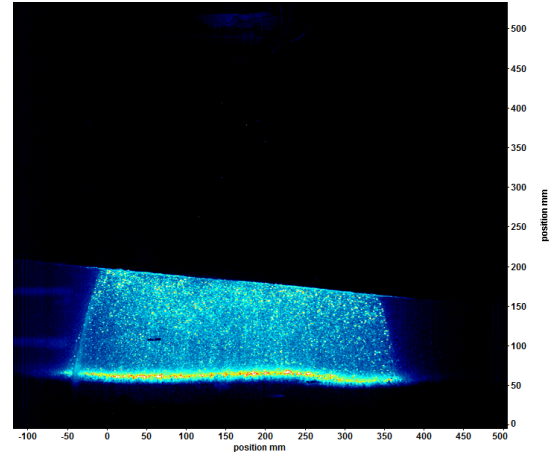


Figure 6.5: Maximum seeding.

Keep in mind that good seeding allows you to have better interrogation windows during the processing with the Davis8 software.

10. Turn off the lights so that only the laser lightens the channel.

Now that all the elements are set up. The experiment can be proceeded. Try to be rigorous and constant when going through those steps for accurate results. Before you start the recording, there are still a few things to check out:

1. The pressure of the tank should be at 8 bars for an instantaneous gate opening.
2. No extra light from the computer or the control box should interfere with the laser.
3. The green light on the control box should be on.
4. The sieve should be placed on the blue tank to prevent sand from falling inside it.

The gate opening is done by pressing the green button and the downward arrow simultaneously during 5 seconds on the control box. When the gate drops, this will automatically trigger the high-speed camera. The later will record 2.72seconds from the moment the gate is completely opened.

You can now, save the data in the .bmp format on the computer.

Finally here are some personal advises that will save you some times and prevent some of your samples to be unusable:

1. If the green light is off, the gate opening will not proceed. Check that the blue fence (see figure 6.6) in front of the gate system is correctly closed.



Figure 6.6: Protective blue fence

2. When you reuse the water from the blue tank, you may want to add some more pliotite solution to maintain to same particles density.
3. When you fill the channel with water, some foam might form at the water surface because of the soap used in the solution. You should remove this foam because it will block the laser sheet from lightening the water during the experiment.
4. Some pliolite particles might stick to the glass window where you want to record the flow. If so, clean it every now and then.

5. If the trigger system on the camera does not match the gate opening, this means you are using the wrong trigger cable. Switch it until you find the correct one.
6. You need clean water for your experiment. The blue tank should be cleaned up if your water gets dark.
7. When working with the pliolite, you should wear gloves and a mask. Avoid long duration contact with your skin. Avoid breathing it as well for that it might be toxic.

When you are done with your experiment and leave the laboratory you need to:

1. Empty the pressure tank down to 0 bars on the barometer as shown in figure 6.7.



Figure 6.7: Barometer

2. Leave the gate up.
3. Empty the water collector under the gate system with the pump to the sewer.
4. Turn off the compressor and the computer by switching them off on the control panel.

You might face other troubles than those described above and you will need to find a solution by yourself but this should help you to master the laboratory procedures in general. You may also want to watch Illaria Fent videos on how to use the laboratory to have a better understanding.

Annex 2 : Graphs of the experimental and theoretical curves

Here are all the graphs representing the theoretical curve of the velocity profile found by Sumer et al. as we fit them onto the experimental curve. This graphs represent all the work done during the analysis of the Clauser method as explained in the procedure subsection 5.1.3. Thosz graphs are also present in our digital annex

H=0.25m - Sand 1 ($d_{50} = 1.83mm$)

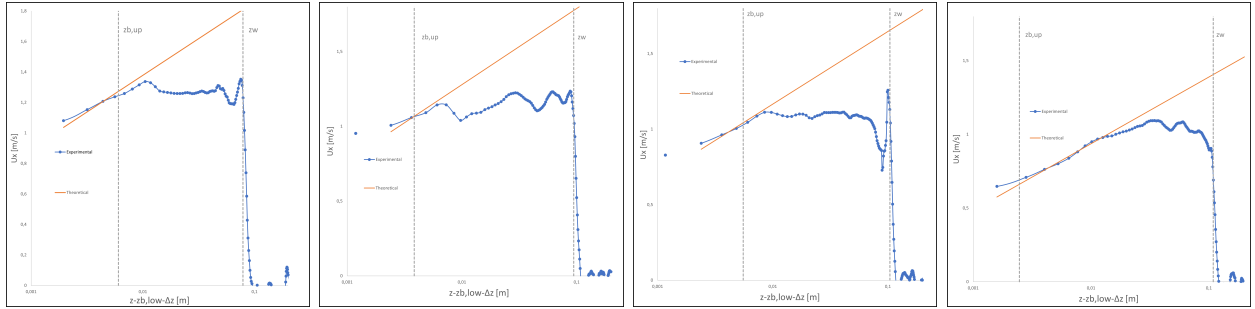


Figure 6.8: Experiment 1, $d_{50} = 1.83mm$, times $t= 0.6s, 0.9s, 1.2s$ and $1.5s$ (from left to right), $H=0.25m$ and position $d= 200mm$.

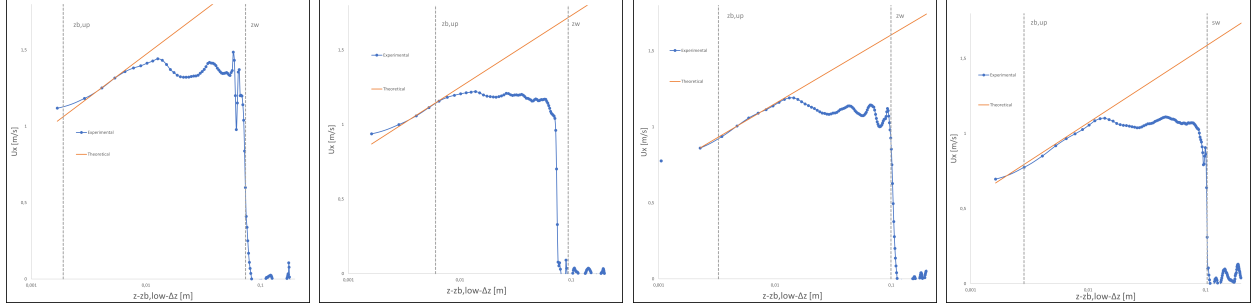


Figure 6.9: Experiment 4, $d_{50} = 1.83mm$, times $t = 0.6s, 0.9s, 1.2s$ and $1.5s$ (from left to right), $H=0.25m$ and position $d = 200mm$.

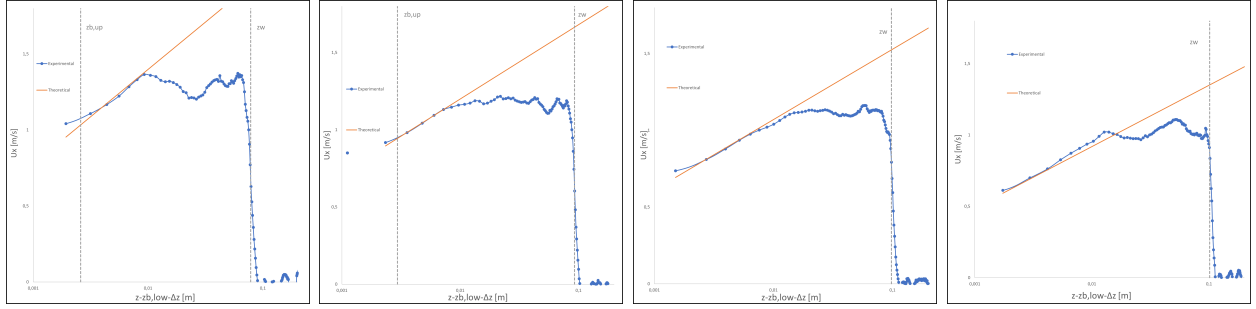


Figure 6.10: Experiment 5, $d_{50} = 1.83mm$, times $t = 0.6s, 0.9s, 1.2s$ and $1.5s$ (from left to right), $H=0.25m$ and position $d = 200mm$.

H=0.30m - Sand 1

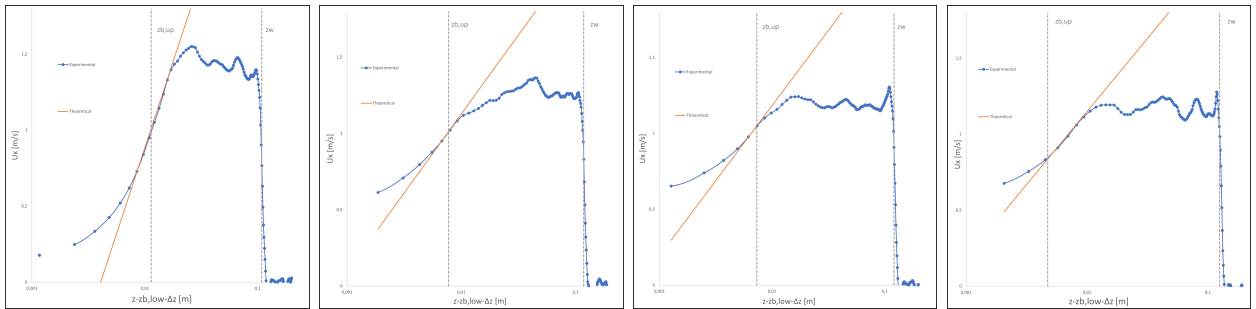


Figure 6.11: Experiment 1, $d_{50} = 1.83mm$, times $t = 0.6s, 0.9s, 1.05s$ and $1.2s$ (from left to right), $H=0.30m$ and position $d = 200mm$.

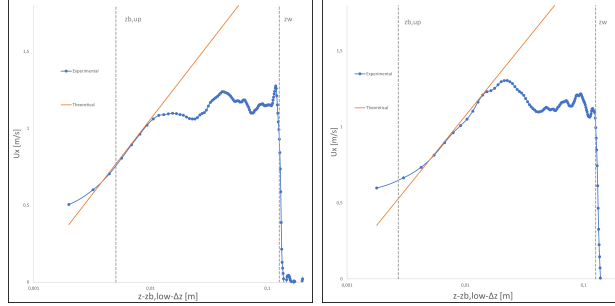


Figure 6.12: Experiment 1, $d_{50} = 1.83mm$, times $t= 1.35s$ and $1.5s$ (from left to right), $H=0.30m$ and position $d= 200mm$.

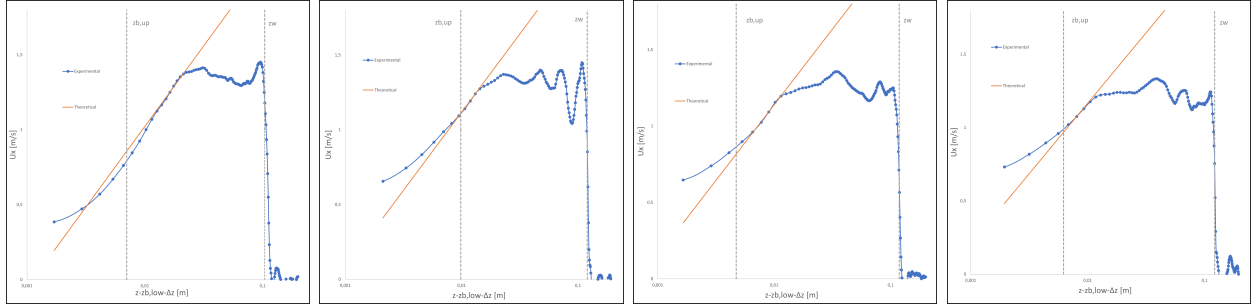


Figure 6.13: Experiment 2, $d_{50} = 1.83mm$, times $t= 0.6s, 0.9s, 1.05s$ and $1.2s$ (from left to right), $H=0.30m$ and position $d= 200mm$.

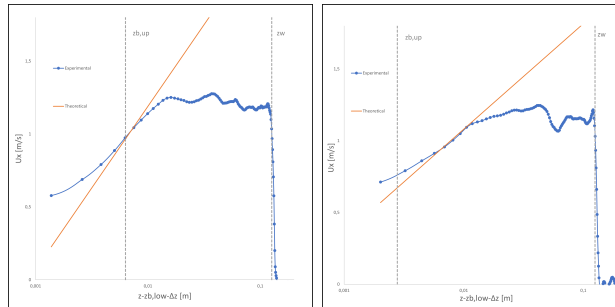


Figure 6.14: Experiment 2, $d_{50} = 1.83mm$, times $t= 1.35s$ and $1.5s$ (from left to right), $H=0.30m$ and position $d= 200mm$.

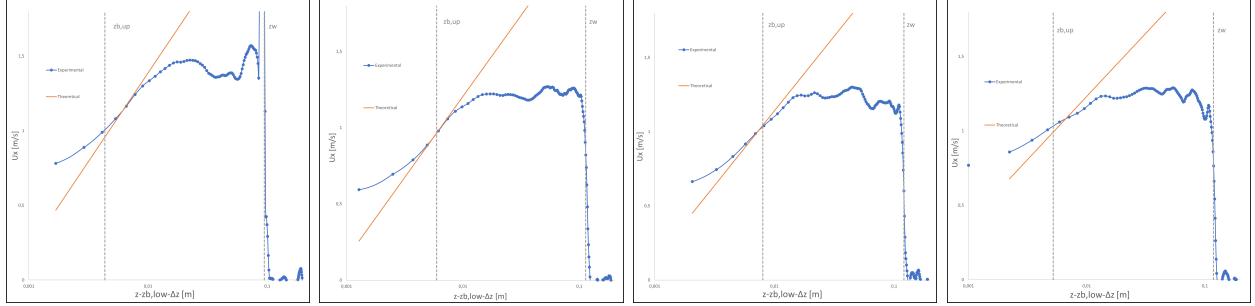


Figure 6.15: Experiment 3, $d_{50} = 1.83mm$, times $t= 0.6s, 0.9s, 1.05s$ and $1.2s$ (from left to right), $H=0.30m$ and position $d= 200mm$.

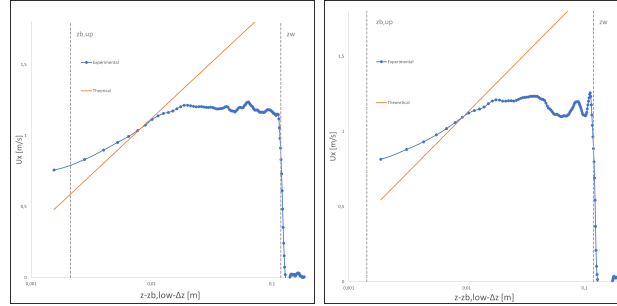


Figure 6.16: Experiment 3, $d_{50} = 1.83mm$, times $t= 1.35s$ and $1.5s$ (from left to right), $H=0.30m$ and position $d= 200mm$.

H=0.30m - Sand 2 ($\sigma \approx 0.02$)

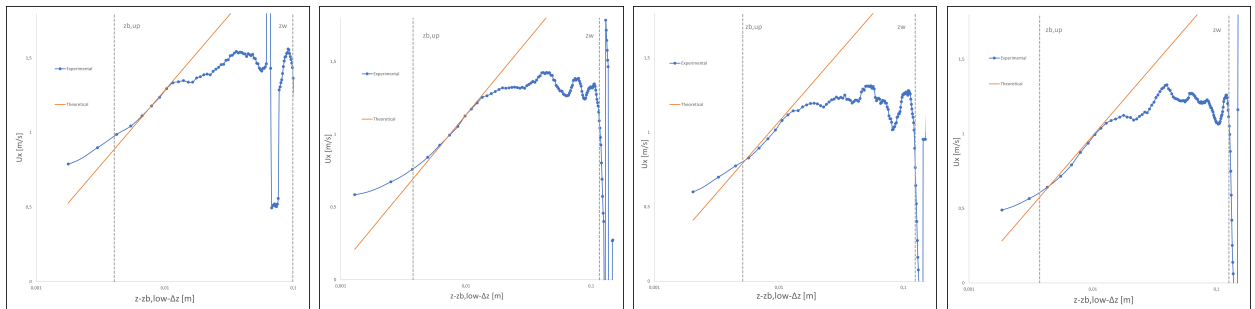


Figure 6.17: Experiment 1, $d_{50} = 1.83mm$, times $t= 0.6s, 0.9s, 1.25s$ and $1.5s$ (from left to right), $H=0.30m$ and position $d= 200mm$.

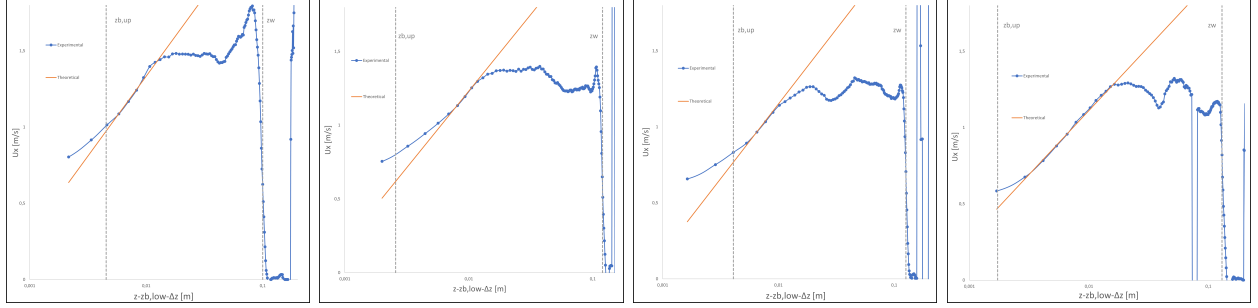


Figure 6.18: Experiment 2, $d_{50} = 1.83mm$, times $t= 0.6s, 0.9s, 1.25s$ and $1.5s$ (from left to right), $H=0.30m$ and position $d= 200mm$.

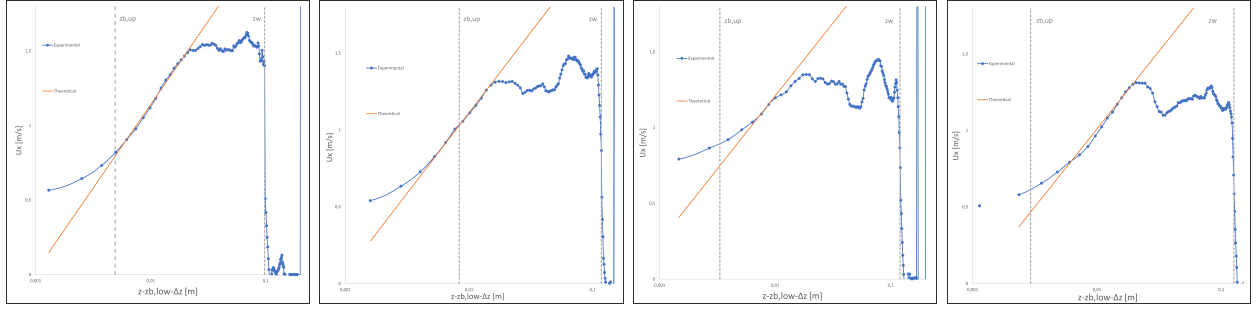


Figure 6.19: Experiment 3, $d_{50} = 1.83mm$, times $t= 0.6s, 0.9s, 1.25s$ and $1.5s$ (from left to right), $H=0.30m$ and position $d= 200mm$.

H=0.30m - Sand 3 ($d_{50} = 1.35mm$)

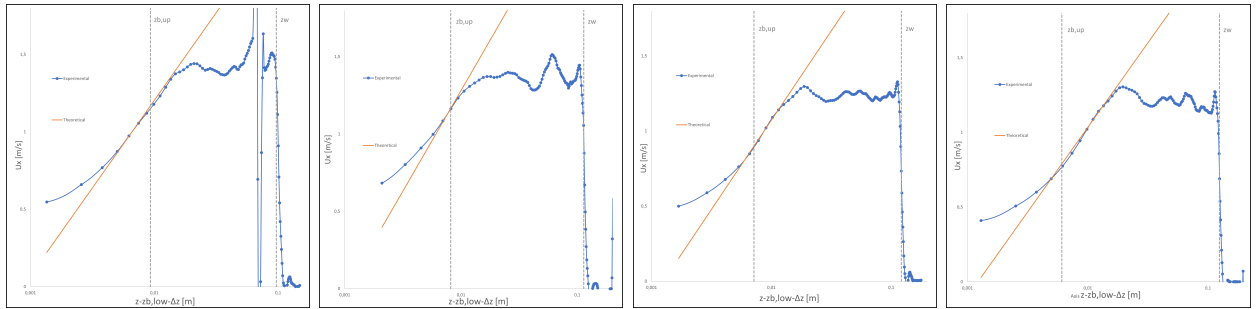


Figure 6.20: Experiment 1, $d_{50} = 1.35mm$, times $t= 0.6s, 0.9s, 1.25s$ and $1.5s$ (from left to right), $H=0.30m$ and position $d= 200mm$.

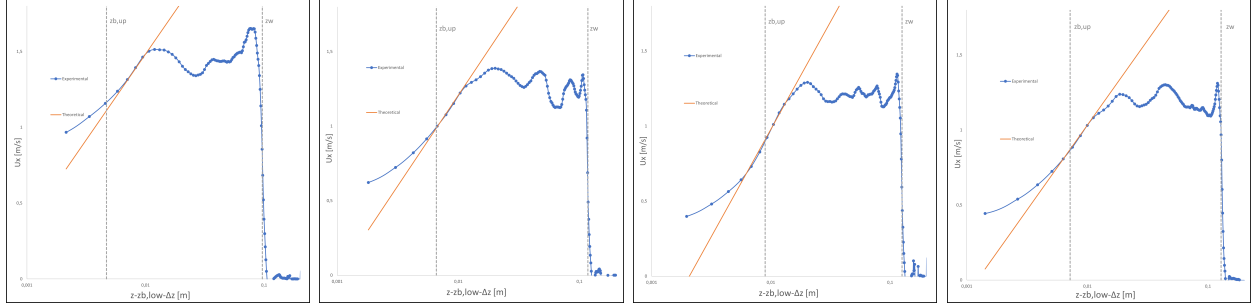


Figure 6.21: Experiment 2, $d_{50} = 1.35mm$, times $t= 0.6s, 0.9s, 1.25s$ and $1.5s$ (from left to right), $H=0.30m$ and position $d= 200mm$.

H=0.35m - Sand 1 ($d_{50} = 1.83mm$)

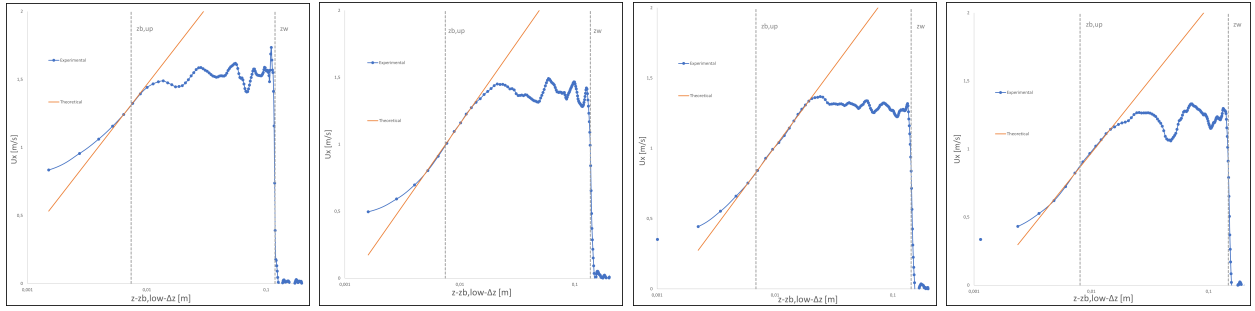


Figure 6.22: Experiment 1, $d_{50} = 1.83mm$, times $t= 0.6s, 0.9s, 1.25s$ and $1.5s$ (from left to right), $H=0.35m$ and position $d= 200mm$.

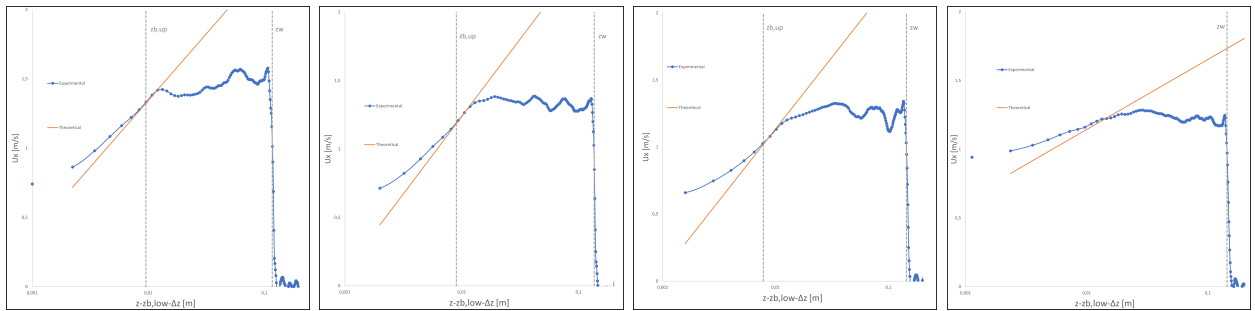


Figure 6.23: Experiment 2, $d_{50} = 1.83mm$, times $t= 0.6s, 0.9s, 1.25s$ and $1.5s$ (from left to right), $H=0.35m$ and position $d= 200mm$.

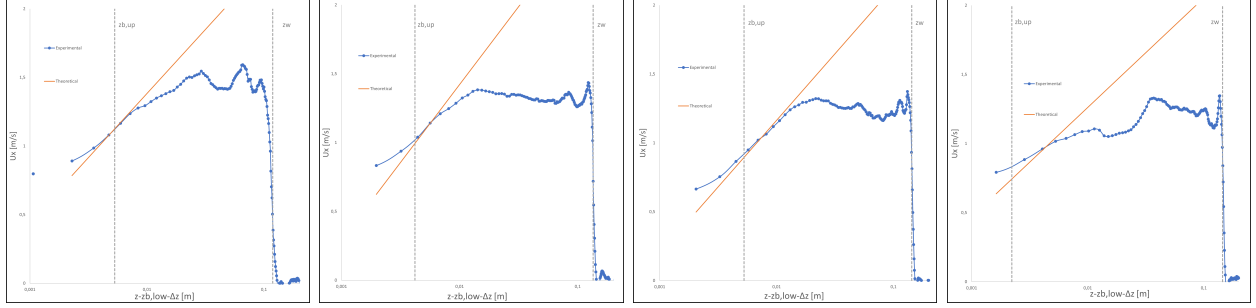


Figure 6.24: Experiment 3, $d_{50} = 1.83mm$, times $t = 0.6s, 0.9s, 1.25s$ and $1.5s$ (from left to right), $H = 0.35m$ and position $d = 200mm$.

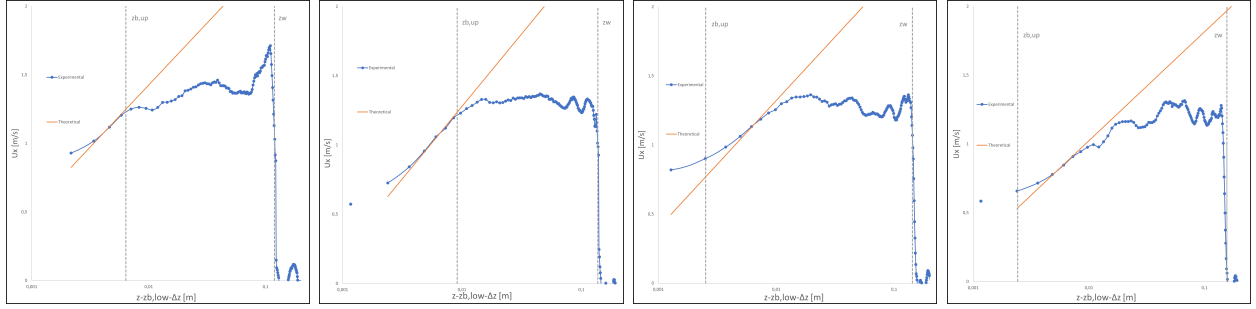


Figure 6.25: Experiment 4, $d_{50} = 1.83mm$, times $t = 0.6s, 0.9s, 1.25s$ and $1.5s$ (from left to right), $H = 0.35m$ and position $d = 200mm$.

Annex 3 : Digital Annex

In this annex, we enumerate all the files that we gave to our promoter Sandra Soares-Frazaõ:

- The videos of our 28 experiments.
 - 3 experiments in front of the door with the sand 1, sand diameter $d=1.83\text{mm}$, an initial water height $H=25\text{cm}$.
 - 3 experiments in front of the door with the sand 1, sand diameter $d=1.83\text{mm}$, an initial water height $H=30\text{cm}$.
 - 4 experiments in front of the door with the sand 1, sand diameter $d=1.83\text{mm}$, an initial water height $H=35\text{cm}$.
 - 3 experiments in front of the door with the sand 2, sand diameter $d=1.83\text{mm}$ (lower standard deviation), an initial water height $H=30\text{cm}$.
 - 3 experiments in front of the door with the sand 3, sand diameter $d=1.35\text{mm}$, an initial water height $H=30\text{cm}$.
 - 4 experiments at 50cm from the door with the sand 1, sand diameter $d=1.83\text{mm}$, an initial water height $H=25\text{cm}$.
 - 4 experiments at 50cm from the door with the sand 1, sand diameter $d=1.83\text{mm}$, an initial water height $H=30\text{cm}$.
 - 4 experiments at 50cm from the door with the sand 1, sand diameter $d=1.83\text{mm}$, an initial water height $H=35\text{cm}$.
- The excel sheets with the fitted curves for all the experiments in front of the door.
- The excel sheet summarizing all the important data ($z_w, z_{b,low}, z_{b,up}, u_{*,Clauser}, u_{*,Slope}, \tau_b, \Delta z, B_r$) of all the experiments in front of the door.
- Our Master's thesis .pdf

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