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Solid transport in flooding rivers with deposition on the floodplains : experimental and numerical investigations

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*Ce n'est pas dans le commencement d'une tâche, mais dans sa poursuite honnête et
sincère que réside la vraie gloire.*

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Abstract

Rivers present a threat on the neighbourhood, the houses and infrastructures when, following a major flood, they overflow their main channel and invade the floodplains. Consequent damage is often aggravated by muddy deposits, especially in urban areas.

In this context of growing flood risk, due to climate changes, studies centred on compound channel flows and sedimentation processes have undergone increasing interest.

The flow in compound channels is complex and present three-dimensional features which affect the transfer of suspended matter from the fast flowing main channel to the slower floodplain flow, separated by a strong shear layer.

This dissertation aims at a better understanding of convection and diffusion phenomena and fine sediment deposit formation on floodplains during overbank flow. The insights to be obtained should help into the development of a set of modelling tools useful on situations where such deposits are observed.

At first, transfer processes between the channel subsections due to turbulent exchanges are studied experimentally and numerically. A tracer (NaCl solution) was injected in a laboratory flume and the evolution of the plume was appreciated through conductivity measurements, providing experimental mixing reference data. The experiments mainly show an asymmetrical expansion of solute over the floodplains when injected in the shear layer, undoubtedly due to the strong turbulent structures observed in compound channels. Results from the various injection configurations are compared to numerical concentrations. The commercial CFD software *Telemac-2D* developed by EDF is used with its solute transport module to provide estimated diffusion coefficients which give a global indication of the mixing intensities in the different regions of the compound cross section. This *global* turbulent diffusion is compared to a second type of numerical simulations where large-scale horizontal structures are resolved using an Unsteady-RANS numerical method. Thereby, the role of these turbulent eddies in solute transport becomes more evident.

Lateral sediment transfer and subsequent deposition patterns are also investigated experimentally and numerically. Novel experiments were designed in a flume equipped with sediment-laden water recirculation and upstream sediment injection systems. Weighting of deposited sediment and digital imaging were used to evaluate the deposition forma-

tion. The study presents the influence of hydraulic parameters on sediment deposition rate, taken into account through the calculated stream-power over the floodplains. A quantitative and qualitative study of sediment deposition is also performed in correlation with various floodplain roughness or configurations.

Numerical simulations are performed with *Telemac's* sedimentological module *Sisyphe*. They show how the critical mobility parameter, together with the bed shear, control the erosion and deposition fluxes. A critical mobility parameter is defined in most sediment transport mathematical models, and theoretical values were investigated. Even if the spatial distribution lacks of realism due to the impossibility of the model to represent mobile deposition, modelled sediment deposition on the floodplains concurs with collected sediment mass during experiments, if however the critical mobility parameter is correctly adapted to the floodplain surface conditions (micro-topography).

Résumé

Les rivières présentent une menace pour les populations avoisinantes, leurs habitations et leurs infrastructures, lorsque, suite à une crue importante, elles débordent de leur lit mineur et envahissent leurs plaines d'inondation. De plus, les dégâts sont souvent aggravés par les dépôts boueux que laisse la rivière derrière elle lorsqu'elle se retire, ces conséquences étant particulièrement marquées en milieu urbain.

Dans un contexte d'augmentation du risque d'inondation, dû aux changements climatiques, des études touchant à l'écoulement en rivière à lit composé ainsi qu'aux phénomènes de sédimentation présentent un intérêt croissant.

L'écoulement dans les rivières à lit composé est complexe et présente de larges composantes tridimensionnelles qui affectent le transfert de matière en suspension depuis les eaux rapides du lit mineur vers l'écoulement plus lent des plaines d'inondation, séparés par une couche de cisaillement.

Cette thèse entend mieux comprendre les phénomènes de convection et diffusion ainsi que la formation de dépôts de sédiments fins sur les plaines d'inondation lors d'une crue. Les conclusions du travail serviront au développement d'outils de modélisation numérique utilisables pour des situations réelles où de tels dépôts sont observés.

Dans un premier temps, les phénomènes de transfert dus aux échanges turbulents entre les sous-sections d'un canal à lit composé sont étudiés expérimentalement et numériquement. Un traceur (une solution de NaCl) a été injecté dans un canal expérimental et l'évolution du panache a été appréciée via des mesures de conductivité, fournissant une base de données expérimentales de référence. Les expériences montrent principalement une expansion asymétrique du soluté au dessus des plaines d'inondation lorsqu'il est injecté dans la couche de cisaillement, sans aucun doute dû aux fortes structures turbulentes observées dans les géométries à lit composé.

Les résultats des configurations d'injection différentes sont comparés à des concentrations numériques. Le logiciel commercial *Telemac-2D* développé par EDF est utilisé avec son module de transport de traceur afin de fournir des estimations de coefficients de diffusion qui donnent une idée globale des intensités de mélange dans les différentes régions d'une section à lit composé.

Cette diffusion turbulente *globale* est comparée à un second type de simulations numériques où les structures horizontales de large échelle sont résolues en suivant une méthode

numérique Unsteady-RANS. De ce fait, le rôle de ces vortex turbulents sur le transport de soluté devient évident.

Le transfert latéral de sédiment et les schémas de dépôts qui s'ensuivent sont également investigués expérimentalement et numériquement. Des expériences innovantes ont été développées dans un canal équipé de systèmes de recirculation d'eau chargée en sédiments et d'injection de sédiments à l'amont. Le pesage des sédiments et l'imagerie digitale ont été employés afin d'évaluer la formation de dépôts. L'étude présente l'influence des paramètres hydrauliques sur le taux de sédimentation, pris en compte via le calcul de la puissance d'écoulement spécifique au-dessus des plaines d'inondation. Une étude qualitative et quantitative des dépôts de sédiments est également menée en rapport avec diverses configurations et rugosités des plaines d'inondation.

Des simulations numériques sont effectuées avec le module sédimentologique de *Telemac: Sisyphé*. Elles montrent comment le paramètre critique de mobilité, avec la contrainte de cisaillement au fond, contrôlent les flux d'érosion et de déposition. Ce paramètre est défini dans la plupart des modèles mathématiques de transport de sédiments, et ses valeurs théoriques ont été explorées. Même si la distribution spatiale manque de réalisme, vu la difficulté pour le modèle de représenter des dépôts mobiles, les dépôts de sédiment modélisés s'accordent avec la masse de sédiment récoltée durant les expériences, à condition que le paramètre critique de mobilité soit adapté aux conditions de surface (micro-topographie) des plaines d'inondation.

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Chapter 1

Introduction

1.1 Context

Fatality or human misunderstanding of nature's most expressive representation? Our present media recurrently reminds us of the threat imposed by rivers on the neighbourhood, the houses and infrastructures when, following a major flood, rivers overflow their main channel and invade the floodplains. The flood discharge is submitted to a compound cross section and the consequent damages are often aggravated by muddy deposits, especially in urban areas. Global warming and subsequent climate changes seem to favour more and more frequent floods. In this context of growing risk and need of floodplain restoration and management, studies centred on compound channel flows and sedimentation processes have undergone increasing interest.



Figure 1.1: *River invading its floodplains*

Mankind is constantly giving up relative safeness to tempting settlements on the river floodplains. Therefore, engineers are expected to be able to predict the extent of flooded areas and the maximum water levels, as well as to be able to assess the adequacy and efficiency of mitigation works and devices. Further, they are asked to anticipate the morphological response of the river to alleviation works. Understanding river morphological processes and assessing the impact of the long-term human interference with river systems definitely requires improved knowledge on transport and deposition of fine sediments in river systems including their floodplains.

1.2 Flow and transport in compound channels

Overbank flows are inherently complex: a region of high shear is formed at the interface between the fast main channel flow and the slower floodplain flow over usually rougher surface. This produces large lateral eddies (Figure 1.2) which transfer longitudinal momentum from the main channel to the floodplains (Sellin, 1964; Shiono & Knight, 1991). Secondary currents arising from non-isotropic turbulence (e.g. Naot *et al.*, 1993) in the shear layer are other three-dimensional features, which are usually recognized as significant in the influence of solute transport processes because they not only transport solute but also affect the momentum and solute exchange coefficients (Shiono *et al.*, 1997).

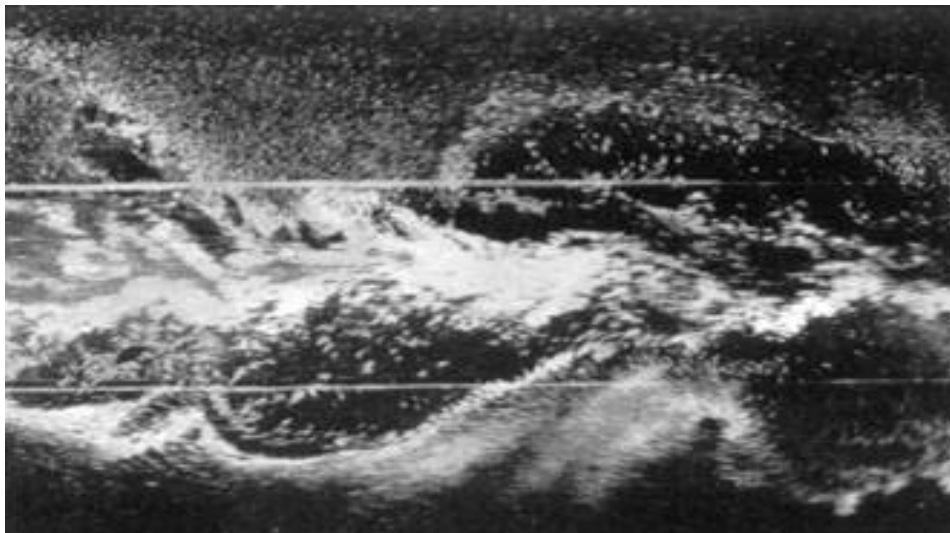


Figure 1.2: *Large vortices observed at the interface between the main channel and the floodplains of a compound channel (Sellin, 1964)*

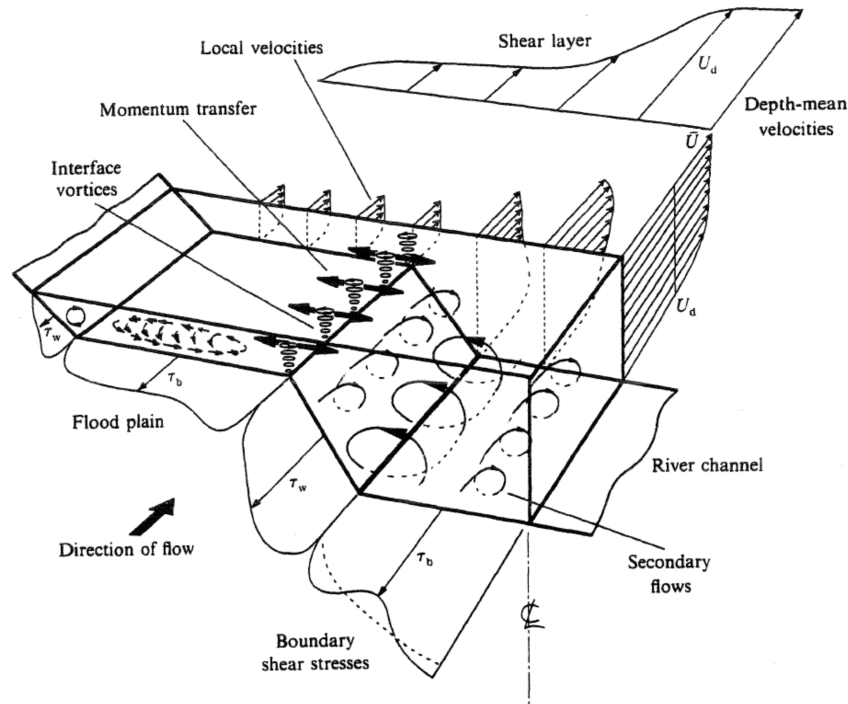


Figure 1.3: *Flow structure in a compound channel (Shiono & Knight, 1991)*

Due to this interaction, exchanges of suspended sediments and solute will be associated to water and momentum exchanges, leading to floodplain sedimentation. The larger sediment transport capacity in the main channel implies sediment concentration gradients between the two subsections resulting in a net transfer from the main channel to the floodplain. Once transferred to the plain region, sediments tend to settle out because of the reduced transport capacity of the flow in this region. As mentioned above, large floodplain sedimentation can become more problematic than flooding itself. For example, Woo & Kim (1997) report sediment deposits up to 1.5 m on floodplains used as recreational areas after a 3-day flood of river Han in Korea in 1990. Sediment deposition on floodplains also determines the morphological evolution of the river basin. As reported by Narinesingh *et al.* (1999) for a project in The Netherlands, this can significantly affect the long term results of re-naturalisation and flood control works. In rural area, floodplain deposits will be less critical, and are even appreciated for their potential fertilizing action. However, as soon as the river basin is industrialized, floodplains may further act as pollutant sinks, with sediments absorbing the major part of industrial wastes (Walling, 1999).

Transport in compound channels is governed by different processes, including convection

and turbulent diffusion; with intensities depending on the geometry of the floodplain and the layout of the main channel and floodplains system (Narinesingh *et al.*, 1999). Predicting the way sediments will settle on the floodplains thus requires analysis of dispersion in this highly three-dimensional flow area. Dye/tracing studies were initiated (Spence *et al.*, 1997; Shiono & Feng, 2003) and mainly show the non-linear turbulent diffusion in the shear layer, even if still no consensus over mixing intensities/coefficients in compound geometry exist.

If flow modelling in compound channel have undergone increasing interest in the past 20 years (Ackers, 1993 ; Shiono & Knight, 1991, Bousmar & Zech, 1999), overbank flows with mobile bed were extensively studied in the UK. Large scale experiments were performed in the Flood Channel Facility (FCF) at HR Wallingford. Most of them focused however on flow resistance due to bed form development (Knight & Brown, 2001 ; Tang & Knight, 2006). Deposition formation on the floodplains have been studied by very few authors : James (1985) reports the main experimental and numerical study on the subject. Fine sand transport and deposits appear to be controlled by an equilibrium between convection, turbulent diffusion and grain fall velocity in a 2-D vertical model, in a plane transverse to the main flow direction. This model relies on a number of empirical parameters but fulfills its objectives by giving good results compared to experiments where thickness and weight of sand deposits were measured on the floodplains with suspended sediment injected in the main channel.

Besides the experiments of James, only a few phenomenological tests were realized in the FCF. For prismatic channel, deposits form mainly near the main channel banks (Knight *et al.*, 1999). For meandering channels, deposits occur on the downstream part of the meander, just past the bend apex (Bathurst *et al.*, 2002).

Very few additional numerical approaches were published since the work by James (1985): Marriott (1996) developed a simplified version of James' model for a case study on long term prediction of River Severn (UK) meanders migration. Narinesingh *et al.* (1999) addressed the evolution of floodplains in non prismatic geometry with a model based only on convection and sediment fall velocity. Few other studies exist (e.g. Nicholas & Walling, 1997) but simulation requires the inclusion of further process models, as sometimes too severe simplifications decredibilize the results. There is a need for further research linking field studies and simulation models of sediment deposition on floodplains, including both floodplain topography and sediment distribution patterns, as performed by Hardy *et al.* (2000) with a two-dimensional approach.

Always more advanced turbulence models are developed to provide reliable compound flow predictions (Thomas & Williams, 1995) and even applied to understand solute dispersion (Lin & Shiono, 1995). Complexity and computational resource requirements rise together with the need of experimental data for models validation. This evolution should not happen at the expense of understanding the mechanics of sediment transport in compound channels, where very few studies feed the discussion, as most sediment transport experiments have been undertaken in simple channels with only inbank flows (Tang & Knight, 2006). All in all, novel 2-D modelling with thorough comprehension of transfer processes and gauging of existing deposition and erosion fluxes formulae (Celik & Rodi, 1988), even in prismatic geometry, would increase our knowledge of sediment transport in rivers in times of flood with sufficient complexity.

1.3 Objectives and outline

We aim at a better understanding of convection and turbulent diffusion phenomena and fine sediment deposit formation on floodplains during overbank flow. The insights to be obtained should help into the development of a set of modelling tools useful on situations where such deposits are observed.

The text is organised as follows. Chapter 2 presents a short state-of-the-art of research on sediment transport and deposition on the floodplains. This chapter also illustrates the complexity of the flow structures in compound channels, as it is a pre-requisite to treating sediment problems in such geometry. Hopefully, this will give the reader the framework and the interest of the present study.

Two different methodologies of investigation are followed for studying solute and sand transport in this thesis: laboratory experiments and numerical modelling. Numerical models are based on a mathematical description of both flow and transport, which is performed in Chapter 3. The three-dimensional fluid motion general purpose model (the Navier-Stokes equations) and its two-dimensional simplification (the Shallow Water equations) are presented, understanding by the same occasion the underlying degrees of abstraction in either case. An appropriate description of nature's basic transfer processes in water leads to models for solute and mass transport. All definitions given in this chapter are used to describe the mechanics of sediment transport, the main scope of this study.

Experiments are carried out in prismatic compound channel geometry to identify the mechanisms that govern mass transfer between the channel's subsections and the deposi-

tion process on the floodplains. Experimental data is used on the other hand for validation of numerical modelling. Chapter 4 deals with this experimental approach. The extensive modifications brought to the pre-existent experimental setup and the novel measurement techniques are described, as much work was requested to introduce morphodynamic experiments in the UCL compound channel flume.

The dispersive capacity of the flow in compound geometry is investigated in Chapter 5 through saline (NaCl) solution injection and tracing of the plume via concentration measurements using a conductivity meter specially designed to avoid wall and free surface effects. Mixing intensities are compared to results collated from published field studies. The water depth together with turbulence intensity in the shear layer are shown to influence the transverse dispersion and the lateral transfer of mass on the floodplains. Observations are brought one step further with sediment transport experiments, in which sand deposition patterns are analyzed using digital imaging. The study presents the influence of hydraulic parameters on sediment deposition rate with various floodplain roughness or configurations.

The lateral transfer is further investigated in Chapter 6, with the aid of numerical simulations using *Telemac 2-D*, in which the large scale depth-averaged motion is explicitly resolved using a coherent structure capturing approach. The horizontal shear layer turbulent structures are recognized to play a great role in solute and fine sediment transport, and an approximative quantification of their effect is attempted. A fine sediment transport model for compound channels is further built, based on *Telemac's* morphological module *Sisyphe*, featuring no explicit turbulent structures modelling. In the light of experimental observations, efforts concern the identification and calibration of the most relevant numerical parameters which condition the deposition formation process and deposits morphology, with a focus on erosion and deposition fluxes.

Finally, in Chapter 7, conclusions are drawn and the new insights are discussed. This includes the applicability of contributions as well as recommendations for further research.

Chapter 2

State of the art

A full review of past studies on compound channel flow and transport phenomena would exceed the scope of this work. Selected previous works relative to compound channel flow modelling will instead illustrate the complexity of the flow in such geometry and explain how it hardens any transport study. This short state-of-the-art also intends to present the most significant contributions to fine sediment transport study during overbank flow, emphasizing on the limitations of the few existing numerical models which tend to model sediment deposition on the floodplains.

This work is concerned by many domains of expertise and requests a proper investigation of several closely related - or not - research activities. The comprehensive work on flow and sediment transport in compound channels from Japanese and UK research teams recounted in the IAHR monograph published by Ikeda & McEwan (2009) provides an updated state-of-the-art review. When it comes to mixing processes in rivers, the interested reader should refer to the work of Rutherford (1994), which provides rule of thumb relations, together with literature values on mixing intensities in many geometries.

2.1 Compound channel flow

As a river changes from inbank to overbank flow, there is a significant increase in the complexity of flow structures. Whereas inbank flows may be treated as if they were predominantly one-dimensional flows in the streamwise direction, despite known three-dimensional mechanisms being present in all flows, overbank flows must be treated differently as certain three-dimensional processes begin to be especially important, particularly the main channel/floodplain interaction.

The velocity in the floodplains is generally lower than in the main channel, with shallower water level and higher roughness due to the presence of vegetation. The difference in velocity may produce strong lateral shear layers which generates large scale coherent structures due to shear instabilities, as already seen on Figure 1.2. This turbulent motion causes momentum to be transferred from the high velocity side to the low velocity side and results in a smaller velocity gradient.

Turbulence is further expressed in the form of a series of helicoidal vortices rotating in the streamwise direction. Those vortices are generally present in all turbulent flows in straight conduits with a non-circular cross section, due to anisotropic turbulence, and are closely related with secondary flow of Prandtl's 2nd kind.

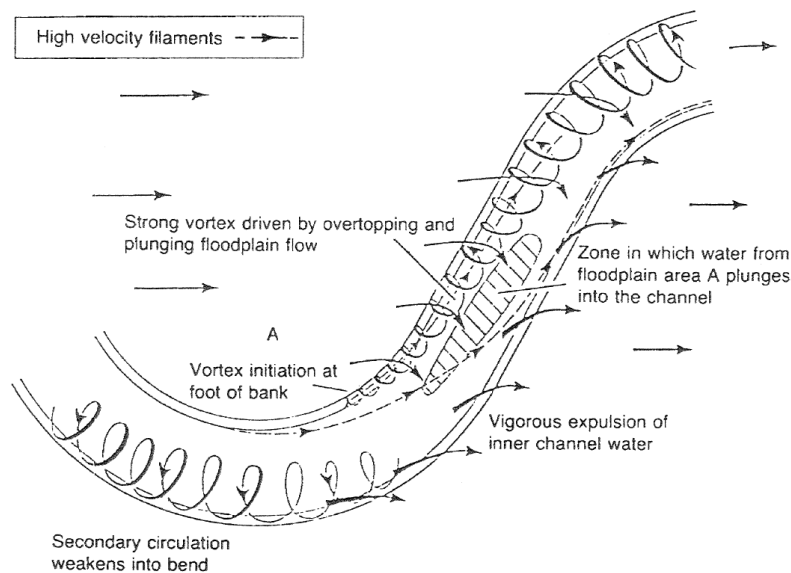


Figure 2.1: Flow features within a flooded meandering channel (Sellin et al., 1993)

Complex interactions of planform and streamwise vortices, rotating about either vertical or horizontal axes, are presented schematically on Figure 1.3, together with their influence on key hydraulic parameters, such as velocity and boundary shear stress. The individual effect of the distinct turbulent mechanisms is impossible to isolate, but their relative intensity will depend on geometrical parameters, such as the relative depth h_r (floodplain depth/total depth). For instance, higher relative depths will remove the strong lateral shear influence. The maximum interaction between the river and the floodplain usually occurs at relative depths of around 0.1-0.3 (Knight & Shiono, 1996).

When the analysis is extended to non-prismatic geometry such as overbank flow in a meandering channel, flow structures and interactions appear even more complex, as 3-D flow structures gain in importance (Figure 2.1). In fact, the flow analysis must be treated very differently, but it gains realism as the situation corresponds to actual river geometry as observed in nature.

2.1.1 Shear layer development

Hydrodynamic instabilities gradually grow due to the transverse gradient of the stream-wise velocity into large scale coherent structures. Instability is reached in case of an inflection point in the transverse profile (Drazin & Reid, 1981). This vortex generation mechanism is known as *internal transverse shear instabilities* (Jirka, 2001). The term internal is used to emphasize that there is no external forcing, as for coherent structures which develop on the lee side of an object. Such organized planform vortices are illustrated in Figure 2.2.

Once generated, coherent structures develop depending on the turbulent kinetic energy production and dissipation mechanisms. The coherent structures are fed by the transverse shear and decay under the influence of bottom turbulence, partly noticeable as a bottom shear stress and partly as an effective eddy viscosity.

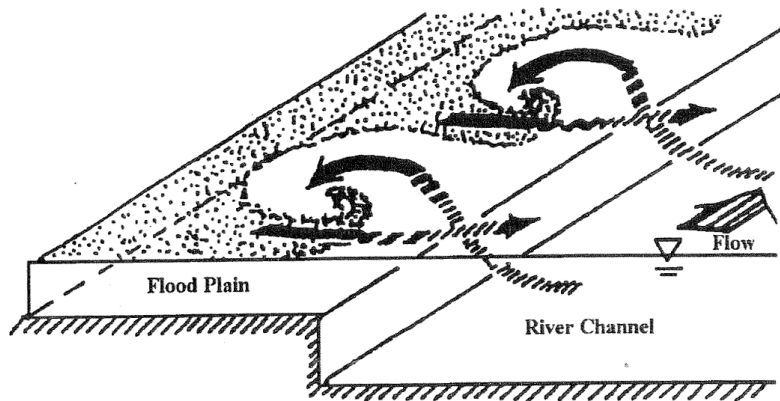


Figure 2.2: Large-scale vortex structure associated with overbank flow (Fukuoka & Fujita, 1989)

Interfacial turbulent vortices with vertical axis were first described by Sellin (1964) who investigated experimentally the behaviour of the uniform flow in a compound channel.

Since then, many researchers have studied the origins and effects of organized planform vortices, e.g. Tamai *et al.* (1986), Fukuoka & Fujita (1989), Nezu & Nakayama (1997), Bousmar & Zech (2002, 2003), among others. Analysis went from experimental visualization and numerical simulation to hydrodynamic stability analysis. Bousmar & Zech (2002) observed the periodicity of those large-scale vortices and captured the vortices wave-length and width to help characterizing the momentum transfer between main channel and floodplains.

Figure 2.3 presents local Reynolds (turbulent) stresses τ_{yx} (on vertical planes) which illustrates the effect of large-scale planform vortices on Reynolds stresses and momentum transfer in a compound channel. These are taken from the straight channel results of the Flood Channel Facility (Wallingford, England) Phase A data (Knight, 1992). At the channel centreline, $y = 0$, $\tau_{yx} = 0$, as would be expected for a symmetric channel. For $0.5 < y < 1.5m$, there is evidence of a strong shear region, in which these Reynolds stresses are significant, reaching a maximum at $y = 0.9m$, i.e. at the top of the main channel bank and floodplain/main channel interface.

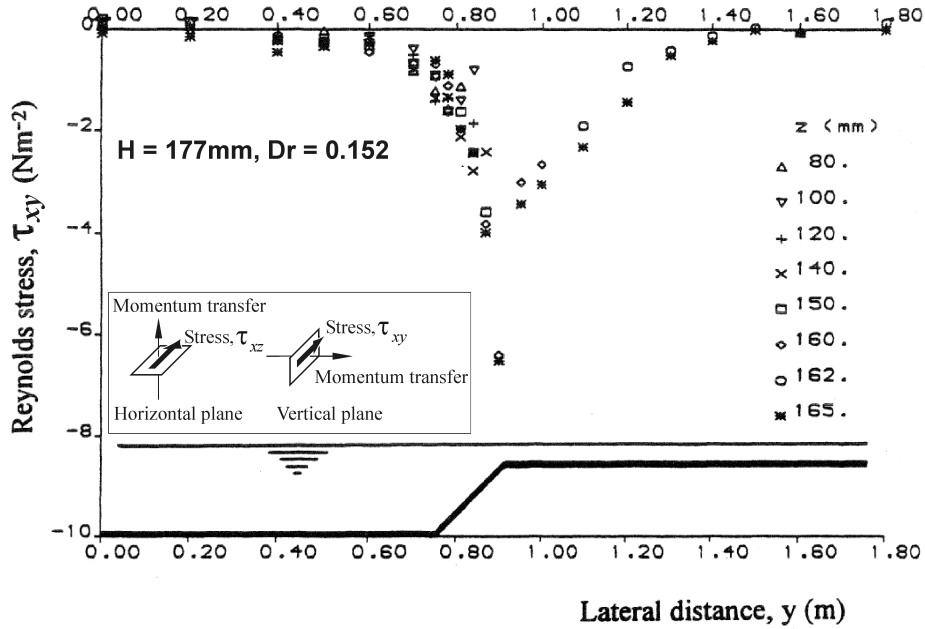


Figure 2.3: Lateral variation of Reynolds stresses, τ_{yx} recorded in the FCF (Knight, 1992)

PIV and PTV techniques have been employed to obtain the instantaneous 2D flow field of the planform vortices (e.g. Nezu *et al.* 1994; Bousmar, 2002) while numerical calculation

using SDS-2DH (Sub-Depth-Scale-2D Horizontal) turbulence model (e.g. Bousmar & Zech, 2003) are able to model explicitly those large scale structures.

2.1.2 Secondary flow in compound geometry

It is well known that longitudinal vorticity is present in all turbulent flows in non-circular sections (e.g. Einstein & Li, 1958; Perkins, 1979) and that these cause perturbations in any lateral distribution of boundary shear stress, and affect the pattern of isovels (Knight *et al.*, 1994).

Turbulence anisotropy is the primary mechanism for the generation of turbulence-driven secondary motions in planes normal to the main flow direction (Prandtl, 1926). These motions are generally quite small (4 % of the maximum streamwise velocity) but are nevertheless quite influential in determining the overall flow behaviour.

Recent research has thus highlighted the three-dimensional nature of the flow in compound channels (e.g. Tominaga & Nezu, 1991), and many authors have performed full three-dimensional analysis (Naot & Rodi, 1982; Lin & Shiono, 1992, 1994; Cokljat & Younis, 1995). Numerical modelling using the RANS equations requests turbulence models which are able to cope with turbulence anisotropy, such as the 3-D Algebraic Stress Model (Naot *et al.* 1993) or Reynolds Stress Modelling (Demuren & Rodi, 1984). Indeed, Boussinesq's linear stress-strain constitutive relationship needs to be abandoned in favour of those alternative means for representing the unknown Reynolds stresses (Younis, 1996).

Advanced 3-D flow measurements techniques such as LDA system are requested to investigate the nature of time-averaged longitudinal vortices in compound channels (Nezu, 1996; Shiono *et al.*, 2003). Figure 2.4 presents such measurements, together with 3-D numerical modelling of the transverse and vertical flow components in symmetrical compound channel using a RSM turbulence model (Cokljat & Younis, 1995). Of particular significance is a strong up-flow of secondary flow from the junction between the main channel and the floodplain, towards the free surface (see also Figure 2.6). As for horizontal structures, the relative depth h_r influences the intensity of those longitudinal vortices. Isovels distortions, marked as lines bulge from the junction edge, are generated by the secondary flow (Figure 2.5) as pointed out by Tominaga & Nezu (1991), Shiono & Knight (1991), Nezu (1996), among others.

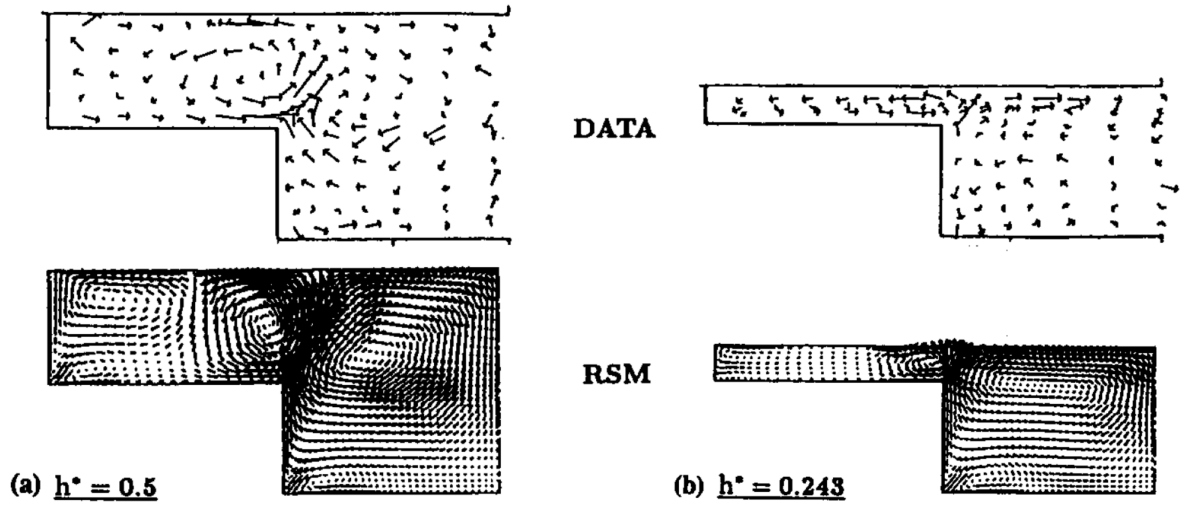


Figure 2.4: Predicted (RSM) and Measured (DATA) secondary flow of compound channels: (a) $h_r = h^* = 0.5$; (b) $h_r = 0.243$ (Cokljat and Younis, 1995)

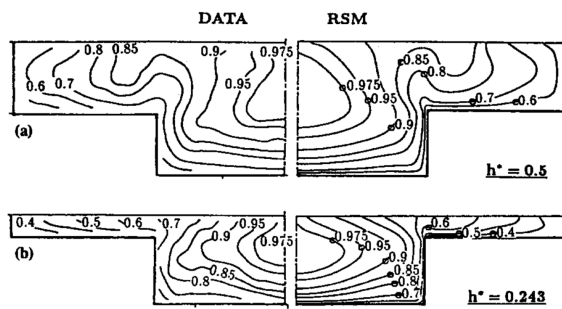


Figure 2.5: Contours of the main flow velocity ($\bar{u}/\bar{u}_{max}$) in compound channels : (a) $h_r = h^* = 0.5$; (b) $h_r = 0.243$

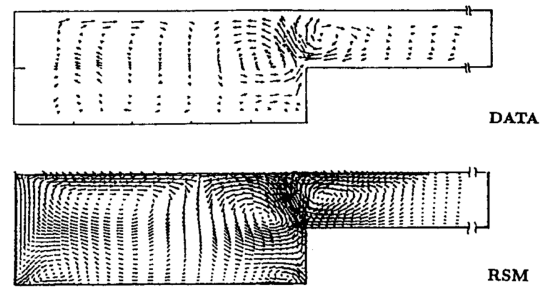


Figure 2.6: Predicted (RSM) and Measured (DATA) Secondary flow in asymmetric compound channels for $h_r = 0.5$

2.1.3 Stage-discharge prediction in compound channels

First hydraulicians understood the challenge underlying the predictions of water levels as a function of the flood discharge and the river's cross section. If this task has somewhat become an easily handled problem in simple geometry channels, compound channel flow has been investigated quite extensively in the last years due to its inherent complexity. Momentum transfer from the main channel towards the floodplains decreases the former's conveyance, while the floodplain's one increases significantly.

One-dimensional models are often preferred to two-dimensional models due to drastically reduced computational costs and data requirement. But such models which intend to estimate the stage-discharge relation in a compound section must clearly take this momentum transfer into account, as we will see.

Traditional methods

When it comes to water stage prediction using channel flow resistance relations, pioneer investigations since the 18th century provided a number of algebraical or empirical formula for uniform flow calculation. Among those relations commonly used in practice, we find the Manning equation (1889):

$$Q = \frac{AR^{2/3}}{n} S_0^{1/2} \quad (2.1)$$

with Q = discharge; A = section area; R is the cross-section hydraulic radius (ratio of the section area to the wetted perimeter P); S_0 is the channel bed slope, equivalent to the friction slope in uniform flows; and n is the dimensional Manning roughness coefficient ($s/m^{1/3}$). Another form of this relation introduces the concept of *conveyance*:

$$S_f = \left(\frac{Q}{AR^{2/3}/n} \right)^2 = \left(\frac{Q}{K} \right)^2 \quad (2.2)$$

where S_f is the friction slope and K is the section conveyance. Extensive tables of Manning's n values for different channel characteristics are given in Chow (1959) and Yen (1991). From a fluid mechanics perspective, one drawback of the Manning formula is that it is dimensionally nonhomogeneous. As an improvement Yen (1992) among others proposed a dimensionally homogeneous form of Manning's equation, which will not be detailed here.

When applied to a compound cross-section, Eq. (2.1) tends to underestimate the discharge for a given flow depth. Indeed, when the water starts to flow on the floodplains, the wetted perimeter increases suddenly and the hydraulic radius decreases accordingly. The so-called Single Channel Method (SCM) is preferably restricted to channels with uniform velocity distribution.

Manning's formula needs to be adapted for application to compound channels, where manifest velocity discrepancy between main channel and floodplains suggests to divide the channel cross-section in subsections where the velocities are more homogeneous. A classical subsection division suggests to draw a vertical line between the main channel and the two floodplains (Lotter, 1933), and will be used throughout this work. The Divided Channel Method (DCM) computes the total discharge from the addition of the subsection discharges Q_i , estimated separately using Eq. (2.1)

$$Q = \sum_i Q_i = \sum_i \frac{A_i R_i^{2/3}}{n_i} S_0^{1/2} = \sum_i K_i S_0^{1/2} \quad (2.3)$$

where i refers to a given subsection. The DCM improves the flow prediction, but the discharge is overestimated, as the reduced global conveyance due to momentum transfer is not accounted for.

Alternative methods

As DCM showed manifest weaknesses, some alternative methods were proposed. An exhaustive review may be found in Bousmar (2002). Ackers (1993) developed empirical formulae to correct the computed discharge with the DCM. Lambert & Myers (1998) proposed an alternative way to divide the channel (vertical or diagonal lines from the main channel edge) in subsections and weighted addition of the various discharge computations. Indeed, Yen & Overton (1973) suggested that the best division line would be a division line where the shear-stress equals zero, which appeared almost diagonal but evolving with given discharge and water depth.

Later, Bousmar & Zech (1999) focused on this momentum transfer, as it seems to be the key to accurate one-dimensional stage-discharge predictions in compound channels. They attempted to model explicitly the momentum transfer and developed the so-called Exchange Discharge Model (EDM). The EDM accounts for momentum transfer between channel subsections, estimated as proportional to the velocity gradient and to the discharges exchanged through the interface; where two main processes are identified (Figure 2.7): (1) the turbulent exchange, due to the shear-layer development; and (2) the ge-

ometrical transfer, due to cross-sectional changes. The EDM was further developed by Bousmar *et al.* (2006) for main channel with mobile bed.

Throughout this work, the EDM was used to predict subsectional discharges which produce uniform flow conditions in the laboratory compound channel flume. The development of this method is summarized in the next paragraph.

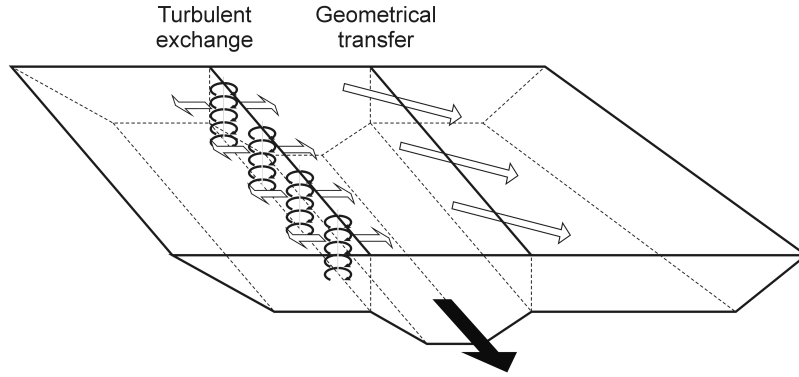


Figure 2.7: Flow exchange at interfaces between main Channel and floodplains (Bousmar & Zech, 1999)

EDM governing equations

Each subsection of a compound channel can be considered as a single channel submitted to a lateral flow per unit length q_l . The conservation of mass may be written:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q_l = q_{in} - q_{out} \quad (2.4)$$

where q_{in} is the inflow discharge; and q_{out} is the outflow discharge.

The momentum equation for a unit length also accounts for the lateral flow (Fig. 2.8):

$$\frac{\partial}{\partial t} (\rho AU) = -\frac{\partial}{\partial x} (\rho AU^2) + \rho q_{in} u_l - \rho q_{out} U + \rho g A (S_0 - S_f) - \rho g A \frac{\partial h}{\partial x} \quad (2.5)$$

where ρ is the density of water; $U = Q/A$ = mean velocity; h is the water depth; and u_l is the velocity of the lateral inflow in the direction of the main flow.

Assuming steady flow, subtracting the continuity equation (2.4) multiplied by U from (2.5), replacing the sum of the bed slope and the depth gradient by the water level gradient $\partial z/\partial x$, and dividing by gA , the momentum equation reduces to:

$$\begin{aligned} S_e &= -\frac{\partial}{\partial x} \left(z + \frac{U^2}{2g} \right) \\ &= S_f + \frac{q_{in,r} (U - u_{l,r}) + q_{in,l} (U - u_{l,l})}{gA} = S_f + S_a \end{aligned} \quad (2.6)$$

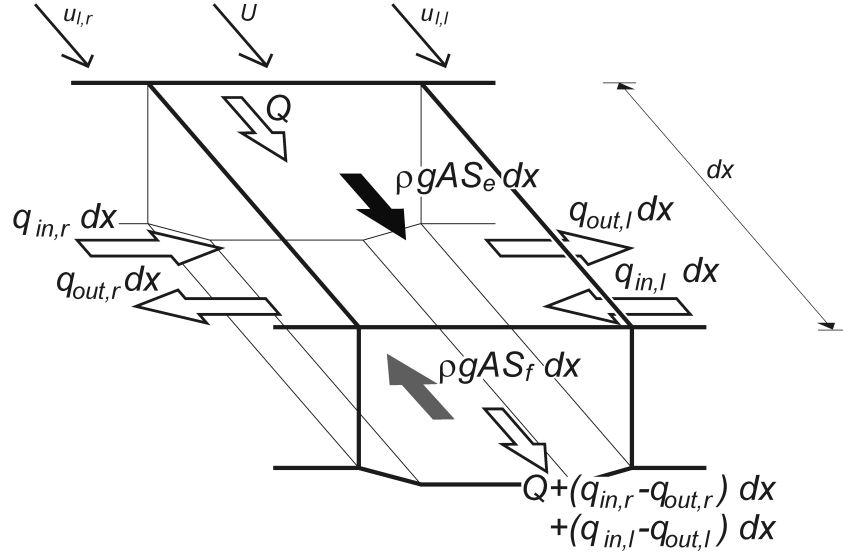


Figure 2.8: Momentum equilibrium for a compound channel subsection (Bousmar & Zech, 1999)

where the lateral flow has been divided into right-side r and left-side l inflow; S_e is defined as the total head loss per unit length; and S_a = additional loss due to lateral inflow, i.e. momentum transfer effect.

The lateral flows represent turbulent exchange and geometrical transfer (Fig. 2.7). The turbulent exchange discharge, q^t , is the mass of fluid which oscillates between main channel and floodplain, due to large vortical structures that develop in the shear layer at the interface, and may be modelled using a mixing length. It is proportional to the velocity difference between subsections $|U_c - U_f|$ and to the interface area per unit length $(h - h_f)$, where c and f stands for main channel and floodplain, respectively; and h_f is the bank level:

$$q_{cf}^t = q_{fc}^t = \psi^t |U_c - U_f| (h - h_f) \quad (2.7)$$

where $\psi^t = 0.16$ = proportionality factor (determined using experimental data). Because turbulent exchange is an oscillating discharge, the lateral inflows q_{cf}^t from main channel to floodplain and the dual inflow q_{fc}^t are equal. Accordingly, the turbulent exchange does not affect the conservation of mass (2.4) but has a significant effect on the momentum balance (2.6).

The geometrical transfer, q^g , corresponds to the floodplain conveyance variation due to a non-prismatic geometry or to a gradually varied flow in a prismatic geometry:

$$q_{fc}^g = \psi^g \kappa_{fc} \frac{dK_f}{dx} S_{ff}^{1/2} ; q_{cf}^g = \psi^g \kappa_{cf} \frac{dK_f}{dx} S_{ff}^{1/2} \quad (2.8)$$

where κ_{fc} and κ_{cf} take the appropriate values, respectively (0, 1) for $dK_f/dx > 0$, i.e. increasing floodplain conveyance, with a transfer from main channel to floodplain; and (-1, 0) for $dK_f/dx < 0$, i.e. decreasing floodplain conveyance, with a reverse transfer. A proportionality factor $\psi^g = 0.50$ has also been included.

Using the momentum equation (2.6) to correct the friction equation (2.2), and assuming that the head loss S_e is the same in all subsections, the subsection discharge is given by:

$$Q_i = \frac{A_i R_i^{2/3}}{n_i} S_{fi}^{1/2} = K_i S_{fi}^{1/2} = K_i \left(\frac{S_e}{1 + \chi_i} \right)^{1/2} \quad (2.9)$$

where $\chi_i = S_{ai}/S_{fi}$. The total discharge is obtained by addition of subsection discharges.

EDM extension

The practical evaluation of (2.9) requires the estimation of χ_i values, using (2.6), (2.7) and (2.8). Combining these expressions together, gives:

$$\begin{aligned} \chi_1 &= \frac{1}{gA_1} \left[\psi^t (h - h_1) \left| \frac{R_2^{2/3}}{n_2} \left(\frac{1 + \chi_1}{1 + \chi_2} \right)^{1/2} - \frac{R_1^{2/3}}{n_1} \right| + \psi^g \kappa_{21} \frac{dK_1}{dx} \right] \\ &\quad \left[\frac{R_1^{2/3}}{n_1} - \frac{R_2^{2/3}}{n_2} \left(\frac{1 + \chi_1}{1 + \chi_2} \right)^{1/2} \right] \\ \chi_2 &= \frac{1}{gA_2} \left[\psi^t (h - h_1) \left| \frac{R_2^{2/3}}{n_2} \left(\frac{1 + \chi_1}{1 + \chi_2} \right)^{1/2} - \frac{R_1^{2/3}}{n_1} \right| + \psi^g \kappa_{12} \frac{dK_1}{dx} \right] \\ &\quad \left[\frac{R_2^{2/3}}{n_2} \left(\frac{1 + \chi_1}{1 + \chi_2} \right)^{1/2} - \frac{R_1^{2/3}}{n_1} \right] \left(\frac{1 + \chi_2}{1 + \chi_1} \right) \\ &\quad + \frac{1}{gA_2} \left[\psi^t (h - h_3) \left| \frac{R_2^{2/3}}{n_2} \left(\frac{1 + \chi_3}{1 + \chi_2} \right)^{1/2} - \frac{R_3^{2/3}}{n_3} \right| + \psi^g \kappa_{32} \frac{dK_3}{dx} \right] \\ &\quad \left[\frac{R_2^{2/3}}{n_2} \left(\frac{1 + \chi_3}{1 + \chi_2} \right)^{1/2} - \frac{R_3^{2/3}}{n_3} \right] \left(\frac{1 + \chi_2}{1 + \chi_3} \right) \\ \chi_3 &= \frac{1}{gA_3} \left[\psi^t (h - h_3) \left| \frac{R_2^{2/3}}{n_2} \left(\frac{1 + \chi_3}{1 + \chi_2} \right)^{1/2} - \frac{R_3^{2/3}}{n_3} \right| + \psi^g \kappa_{23} \frac{dK_3}{dx} \right] \\ &\quad \left[\frac{R_3^{2/3}}{n_3} - \frac{R_2^{2/3}}{n_2} \left(\frac{1 + \chi_3}{1 + \chi_2} \right)^{1/2} \right] \end{aligned} \quad (2.10)$$

where subscript 2 stands for the main channel; subscripts 1 and 3 stands for the floodplains; and h_1 and h_3 = main channel bank level on floodplain 1 and 3 side, respectively.

The system of equations (2.10) defines the values of the ratios χ_i in an implicit form, and as a function only of water depth, geometry and roughness. An analytical solution

of (2.10) for straight symmetrical uniform flow is given by Bousmar (2002), and Bousmar and Zech (1999) proposed a numerical solution procedure for the general case.

2.2 Fine sediment transport in overbank flows

Traditionally the sediment load in rivers is divided into bed load and suspended load (see Chapter 3). Bed load includes the rolling and gliding sediments of larger size near the river bottom. Usually bed load is considered as the most important factor determining the morphological characteristics of the river (Jansen, 1979). Suspended load is however recognized to play an important role in river morphology during overbank flow. Indeed, floodplain levels in the river are closely linked to sedimentation rates of fine sediments transported by suspension.

As seen in previous sections, complex compound channel flow structures modify the 3-D flow fields compared with those found in open channels of simpler cross-section, and in particular the near-bed conditions, which have a profound influence on sediment mobility. Momentum transfer is dominated by interfacial (vertical axis) vorticity for prismatic channels or by spillage of water towards the floodplains near meander bend apex for meandering compound channel flow. In both cases, lateral transfer of water will be associated to lateral transfer of sediment (and other dissolved and suspended material). When entering a new environment, deposition will occur when the transporting capacity of the water is reduced. Floodplains, therefore, exhibit sediment deposits after the recession of inundating floods (Walling *et al.*, 1996).

Increasing economic impact of floods, together with the behaviour of floodplain acting as pollutants sink (Walling, 1999) justifies the interest for sediment transport and deposition during overbank flows. Different approaches are adopted in literature: field observations, laboratory experiments and numerical simulations. This section provides an overview of previous studies on transport phenomena in such geometry.

2.2.1 Field observations

Marriott (1996) performed field observations and examined sediment samples collected after a significant flood on River Severn, UK (Figure 2.9). He appreciated the quantities deposited, sediment composition, textural patterns, and variations of grain size with

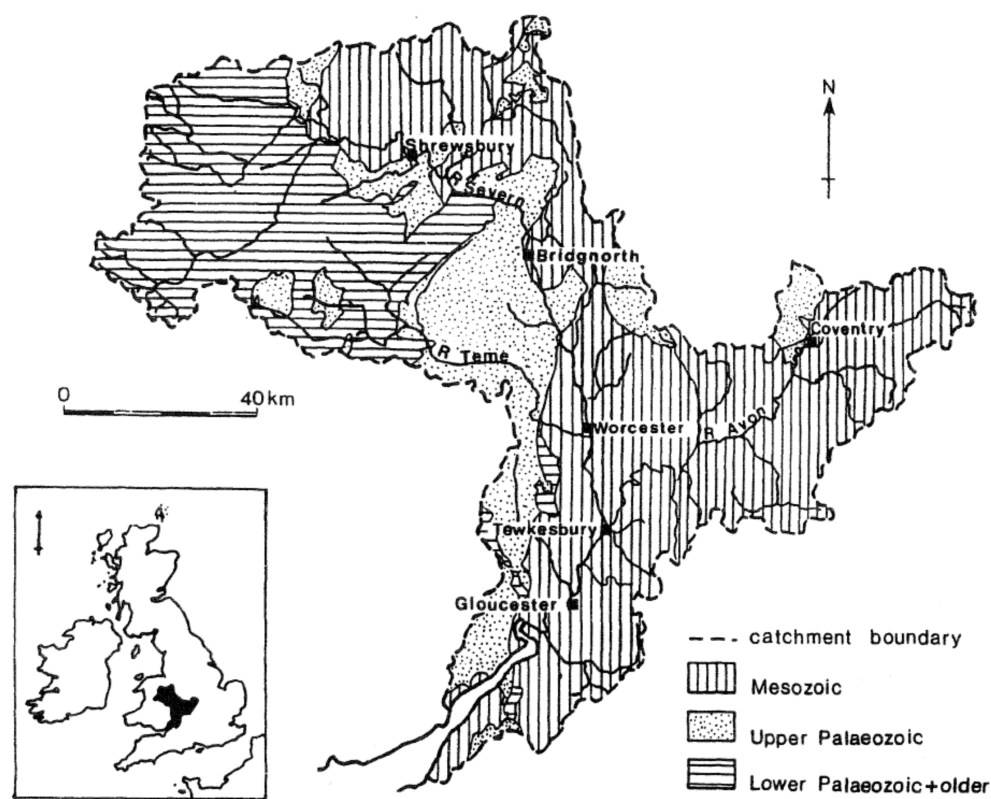


Figure 2.9: *The Severn catchment, showing bedrock geology and main rivers (Marriott, 1996)*

distance from the main channel bank. Deposits appeared near the banks of the main channel with a reduction in grain size with increasing distance from this bank. The mean grain size decreases sharply at around 20 meters from the channel bank. Close to the bank, the sediment consists mainly of sand and appears to be well sorted. At greater distances from the channel, finer sediments are collected, presenting also wider grain size distribution (Figure 2.10). Walling (1999) also observed a decreasing sedimentation rate with the water depth and increasing distance from the main channel. Studies by Kesel *et al.* (1974), Pizzuto (1987), Simm & Walling (1998), Nicholas & Walling (1997), Asselman (1997), Middelkoop (1997) and Dunne *et al.* (1998) further point out that deposition will depend on the micro-topography of the floodplain and the presence of secondary flow routes, especially floodplain channels (Narinesingh *et al.*, 1999). Moreover, all studies agree that overbank deposition is a complicated phenomenon.

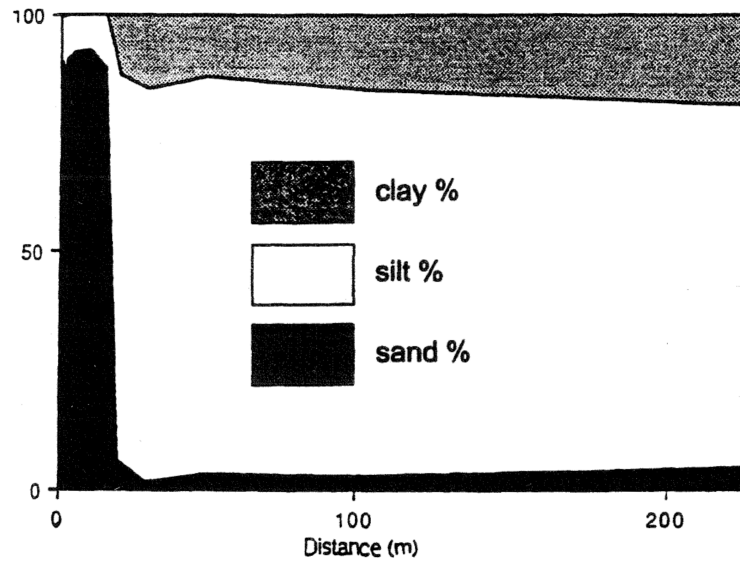


Figure 2.10: *The variation in sand, silt and clay with distance from the channel bank near Worcester (Marriott, 1996)*

2.2.2 Laboratory investigations

Field observations are typically site-specific. Laboratory flume experiments enable methodical study of deposition patterns as function of the flood intensity, channel planform, channel sediment load and sediment characteristics (James, 1985). Some exploratory studies were performed in the Flood Channel Facility (FCF) at HR Wallingford, UK (Benson *et al.*, 1997; Knight *et al.*, 1999; Bathurst *et al.*, 2002). Highest floodplain deposits were observed near the main channel banks, with little sediment transferred further onto the floodplain. Figure 2.11 presents the rippled overbank deposition of fine sand in suspension ($d=100\ \mu\text{m}$) for FCF runs by Knight *et al.* (1999). Figure 2.12 indicates the deposition of the same size fine material on the FCF floodplain (Benson *et al.*, 1997).

More complex deposition patterns were drawn in non-prismatic geometry: in wide meandering channel, deposition spread across the entire floodplain tongue between successive meanders, deposits generally occurring in decreasing velocity areas. Figure 2.13 and 2.14 present such deposition for experiments in the FCF by Bathurst *et al.* (2002).



Figure 2.11: Overbank deposition of 0.1mm sediment in suspension in the FCF Wallingford (Knight et al., 1999)

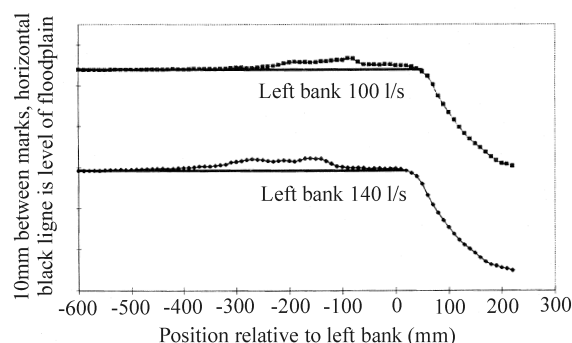


Figure 2.12: Floodplain deposition of suspended sediment introduced close to left bank in main channel (Benson et al., 1997)

2.2.3 Numerical modelling of transport during overbank flows

Few authors managed to construct sediment transport and deposit simulation models. James (1985) and Pizzuto (1987) both developed a numerical transport model to predict the transverse distribution of deposits over the floodplain and the grain size distribution, assuming steady uniform flow. Accordingly, they deal with floodplain deposition due to lateral diffusion only. Convection due to secondary currents is not expressed explicitly, but is encompassed in the transverse diffusion coefficient given by Rajaratnam & Ahmadi (1979) and Lau & Krishnappan (1977). These models are therefore essentially applicable to simplified conditions such as straight channels with continuous prismatic floodplains on both sides. However, these models, validated by experiments, provide results that concurs previous laboratory and field observations: (1) an exponential decrease in the deposition rate with distance to the main channel; (2) a decrease in the sediment particle size deposited with the distance from the main channel; and (3) a depositional trend, which depends on the micro-topography of the floodplain.

Marriott (1996) attempted to include those overbank deposition models to a finite difference computer model for the simulation of alluvial architecture for the Severn River in UK (Crane, 1982). Unsatisfactory results proved that turbulent diffusion alone could not account entirely for the deposition on the floodplains, even if relative distributions of sediment obtained were fairly consistent with the grain size distribution of the sediment

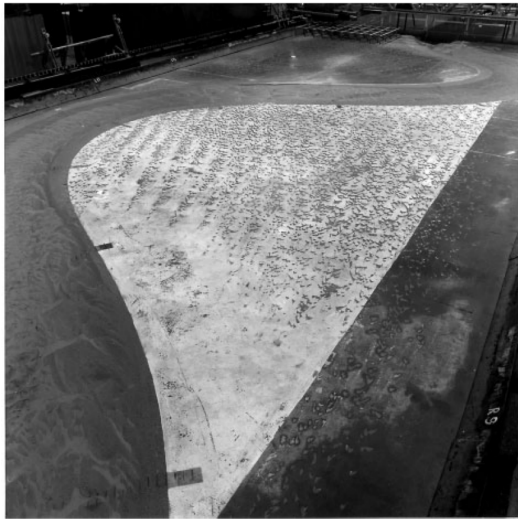


Figure 2.13: *Overbank deposition pattern for meandering channel flow (Bathurst et al., 2002)*

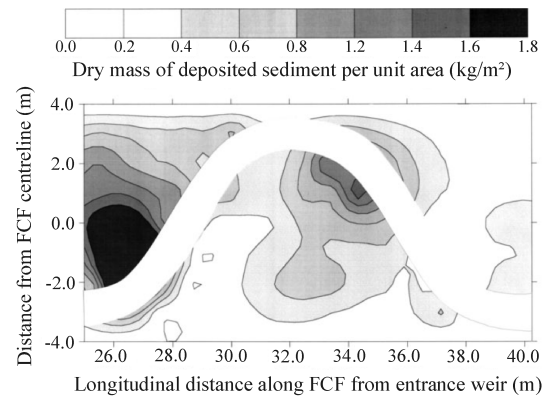


Figure 2.14: *Spatial distribution by mass of the overbank sand deposit (Bathurst et al., 2002)*

samples of Severn River 1990 flood at some locations. Lateral relative deposition profiles of James' model are sketched on Figure 2.15 using the River Severn flow data near Tewkesbury. Nicholas & Walling (1997) adopted a quasi-similar approach to a 600m reach of the River Culm, Devon, UK. They expressed the complexity of inundation sequence and how patterns of suspended sediment dispersion and deposition are controlled by the complex floodplain topography.

The works of Middelkoop (1997), Asselman (1997), Nicholas & Walling (1997), Dunne *et al.* (1998), Simm & Walling (1998), and, more recently, Hardy *et al.* (2000) indicate that a major part of the sediment deposited in overbank flows is transported to the floodplain and into the floodplain channels from the main channel via a convective process and that diffusion at the interface between main channel and floodplain plays a minor role only in the transfer of sediment towards the floodplain. The diffusion process however, distributes the sediment laterally on the floodplain. In addition to sediment properties (like settling velocities and the corresponding particle size), floodplain topography including floodplain channels, lakes, levees and forests play a major role in determining the actual distribution of the sediment load on the floodplain surface.

Since sediment transport and deposition rely on mixing processes in compound channels, predicting the way sediments will settle on the floodplains requires analysis of the diffusion/dispersion and convection phenomena occurring during overbank flow. Researchers

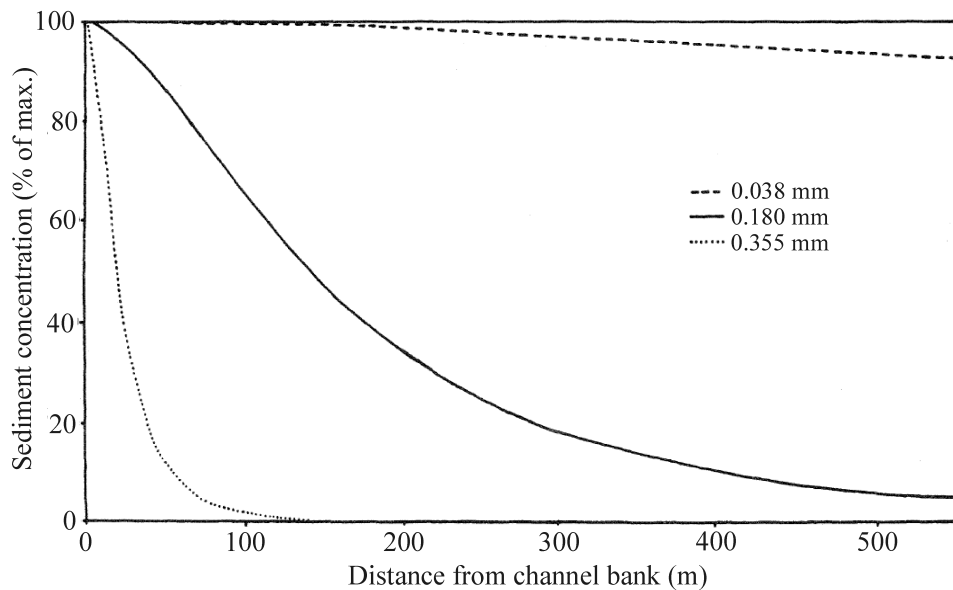


Figure 2.15: Output from the James (1985) numerical model, using different grain sizes, the River Severn channel cross-section and floodplain dimensions and depth of flow at site WG near Tewkesbury. The curves show relative values of predicted suspended sediment concentrations as distance from the channel bank increases. Concentrations are shown as percentages of the maximum for each grain size from the values of the finite difference grid in the layer immediately above the floodplain surface (Marriott, 1996)

thus investigated diffusion processes in compound channel involving dye/solute concentration measurements instead of sediments. Those efforts are also of great help for the control of pollution levels in rivers and the design of waste treatment facilities (Arnold *et al.*, 1985). The presence of strong turbulent features at the interface between the subsections enhanced mixing in this region and lateral transfer towards the floodplains by the same occasion, as explained by Guymer *et al.* (1998). Respective role of turbulence and secondary currents on transport rate is rather complex. Shiono & Feng (2003) showed how the transport rate varies with the different mechanisms along the channel.

Chapter 5 will present the author's work (Fraselle *et al.*, 2008), who investigated diffusion processes in the UCL compound channel flume involving solute concentration measurements instead of sediments, and discussing the results in the light of a consistent literature review on mixing processes in natural rivers (Rutherford, 1994) or compound channel laboratory flume.

Chapter 3

Theoretical description

In this chapter, a mathematical description of a general purpose model of fluid flow (the Navier-Stokes equations) and its two-dimensional simplification (the shallow water equations) is presented. Those governing equations of fluid flow represent mathematical statements of the conservation laws of physics (conservation of mass, momentum and energy) and relies on an appropriate description of nature's basic transport processes: convection and diffusion.

The first section of this chapter will be dedicated to write the conservation equation for mass, which is neither created nor destroyed in the flows of engineering interest. The following section will apply those conservation principles on a *control volume* (CV) in order to describe the transport of a general property φ , and of a solute. The *control volume approach* is chosen in fluid flows because it is more convenient to deal with the flow within a certain fixed spatial region, rather than in a parcel of matter which quickly passes through the region of interest. The latter approach uses a *control mass* (CM) definition (or system) and is used to study the dynamics of solid bodies, where the CM is easily identified.

Later in this chapter, the general transport equation will then be used to describe the three-dimensional fluid motion and its bi-dimensional simplification. The definition of those equations is necessary to understand the complexity of fluid dynamics modelling and the degrees of abstraction used in this work to model all turbulent artefacts appearing in compound channel flow. Moreover, this 2-D version of the equations for free-surface flows is implemented in the Telemac-2D software, which is used in this work for hydrodynamic and sedimentological simulations.

Finally, all previous definitions will be used to describe the mechanics of sediment transport, the main scope of this study. The exposed theoretical or empirical relations in

this chapter have all been used in a way throughout this dissertation and are a result of well validated theories. Sometimes, more recent literature exists, but are often not widely accepted, and will only be mentioned to understand how research has evolved.

3.1 Mass conservation

A typical small element of fluid with sides dx, dy and dz submitted to flow velocity (u, v, w) and presented on Figure 3.1 is considered. The mass balance for incompressible

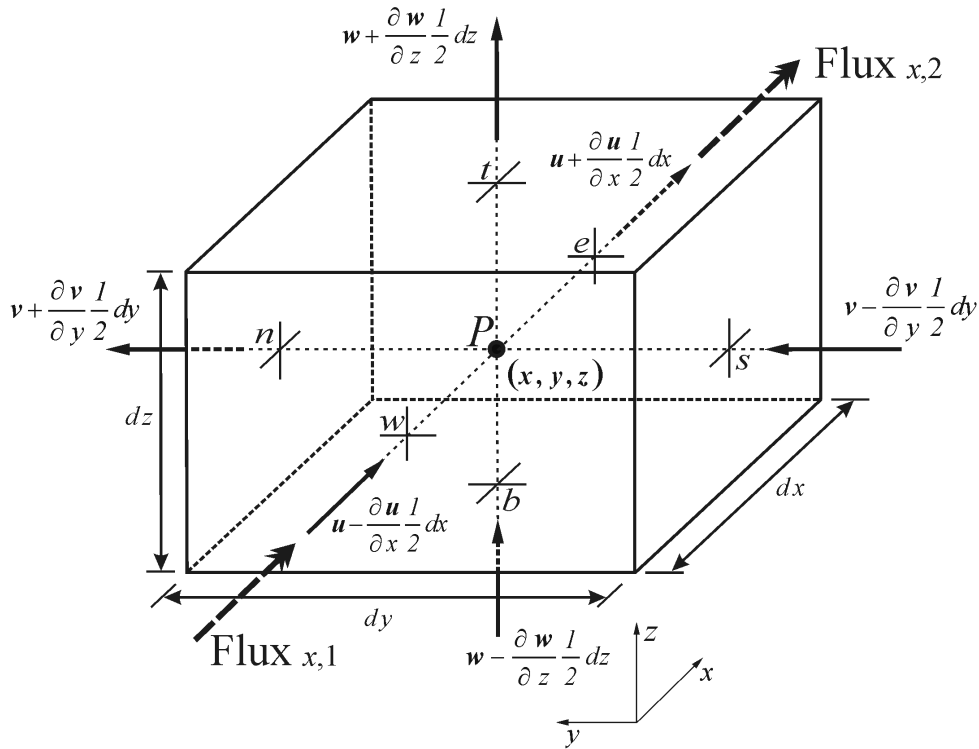


Figure 3.1: Fluid element for conservation laws. Faces are labelled n, s, e, w, t, b which stands for north, south, east, west, top and bottom

flow (acceptable for all liquids) is expressed as :

$$\frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} + \frac{\partial w}{\partial z} = 0 \quad (3.1)$$

This equation is also called the *continuity* equation and is written in more compact vector notation with $\mathbf{u}$ the velocity vector:

$$\text{div} \mathbf{u} = 0 \quad (3.2)$$

3.2 Transport processes

3.2.1 General transport equation

We introduce a general variable φ , which is considered as any conserved *intensive* property (for mass conservation, $\varphi = 1$; for momentum conservation, $\varphi = \mathbf{u}$; for conservation of a scalar, φ represents the conserved property per unit mass). *Intensive* variables are independant of the amount of matter considered (e.g. density ρ and velocity $\mathbf{u}$ which is momentum per unit mass), contrary to extensive properties, like mass, momentum and energy. The most useful form of the conservation laws for fluid flow computation are concerned with change of flow property for a fluid element stationary in space. Conservation of the general property φ will be expressed as the rate of increase of this property in the typical fixed control volume presented on Figure 3.1.

A systematic account of changes in the φ of the fluid element due to fluid flow across its boundary and, where appropriate, due to the action of sources inside the element, leads to the general transport equation, under *integral* form (Ferziger & Peric, 1997):

$$\frac{\partial}{\partial t} \int_{\Omega} \rho \varphi d\Omega + \int_S \rho \varphi \mathbf{u} \cdot \mathbf{n} dS = \sum f_{\varphi} \quad (3.3)$$

where t is the time, Ω stands for volume occupied by the CV while S represents the volume surface enclosing CV (typically, all faces together) and $\mathbf{n}$ is the unit vector orthogonal to S . The first term of the equation expresses the rate of change of the amount of the property within the control volume while the second represents the net flux of it through the CV boundaries due to fluid motion relative to the CV boundary. This term is also called the *convective* flux of φ through the CV boundary. Convection is therefore defined as transport at the fluid velocity. f_{φ} represents transport of φ by mechanisms other than convection and any sources or sinks. *Diffusive* transport constitutes part of it, and expresses transport due to φ gradients in the flow which tends to homogenisation. It is usually described by a gradient approximation e.g. *Fourier's* law for heat diffusion and *Fick's* analogy for mass diffusion:

$$f_{\varphi}^d = \int_S \Gamma \text{grad} \varphi \cdot \mathbf{n} dS \quad (3.4)$$

and Γ stands for diffusivity for the φ quantity. Considering (3.4), equation (3.3) may be re-written as the integral form of the generic conservation equation:

$$\frac{\partial}{\partial t} \int_{\Omega} \rho \varphi d\Omega + \int_S \rho \varphi \mathbf{u} \cdot \mathbf{n} dS = \int_S \Gamma \text{grad} \varphi \cdot \mathbf{n} dS + \int_{\Omega} q_{\varphi} d\Omega \quad (3.5)$$

which in words means that the rate of increase of φ plus the net rate of decrease of φ due to convection across the boundaries are equal to the rate of increase of φ due to diffusion across the boundaries plus the net rate of creation of φ due to sources (q_φ term). The decreasing nature of the convective term comes from the fact that the $\rho\varphi\mathbf{v} \cdot \mathbf{n}$ term expresses the flux component of property φ due to fluid flow along the outward normal vector $\mathbf{n}$. The increasing nature of the diffusion term needs to consider that a diffusive flux is positive in the direction of a negative gradient of the fluid property (along direction $-\text{grad}\varphi$). The product $(\Gamma\text{grad}\varphi) \cdot \mathbf{n}$ which is also written $(-\Gamma\text{grad}\varphi) \cdot (-\mathbf{n})$ is interpreted as a positive diffusion flux in the direction of the inward normal vector $-\mathbf{n}$ into the fluid element.

This integral form is central to the finite volume CFD method (Versteeg & Malalasekera, 1995). By applying the Gauss' divergence theorem to the surface integrals to be transformed into volume integrals and allowing the control volume to become infinitesimally small leads to a differential coordinate-free form of the general transport equation:

$$\frac{\partial(\rho\varphi)}{\partial t} + \text{div}(\rho\varphi\mathbf{u}) = \text{div}(\Gamma\text{grad}\varphi) + q_\varphi \quad (3.6)$$

In cartesian coordinates and tensor notation, the differential form of this equation is:

$$\frac{\partial(\rho\varphi)}{\partial t} + \frac{\partial(\rho u_i \varphi)}{\partial x_i} = \frac{\partial}{\partial x_i} \left(\Gamma \frac{\partial \varphi}{\partial x_i} \right) + q_\varphi \quad (3.7)$$

where u_i are velocity components in direction x_i .

As we will see in the following sections, this conservation equation is the general form for all coming equations that determine solute and sediment transport. Special features of the continuity and momentum equation that form once gathered the so-called Navier-Stokes equation for fluid flow will be described afterward as an extension/reduction of this generic equation.

3.2.2 Solute transport in water

The prediction of solute dispersion in a flow field is based upon the solution of the solute mass conservation equation, which is expressed by the *Reynolds-averaged* convection/diffusion equation (Rutherford, 1994). This expression can be found starting with the generic equation defined in the previous section. For solute conservation, the chosen intensive variable will be c which is the solute concentration in the flow ($-$). Equation (3.7), divided by density ρ , is used with $\varphi = c$ and simplifying the convection terms with

the continuity equation (3.1) to produce the solute transport equation in laminar flows:

$$\frac{\partial c}{\partial t} + u \frac{\partial c}{\partial x} + v \frac{\partial c}{\partial y} + w \frac{\partial c}{\partial z} = \varepsilon_m \left(\frac{\partial^2 c}{\partial x^2} + \frac{\partial^2 c}{\partial y^2} + \frac{\partial^2 c}{\partial z^2} \right) \quad (3.8)$$

where ε_m is the *molecular* diffusion (m^2/s). The appearance of ε_m as replacement for the $\Gamma (\times 1/\rho)$ coefficient expresses that, in laminar flow, diffusive solute fluxes are expressively due to molecular homogenisation which appear even in hydrostatic conditions.

Turbulent transport

On the other hand, turbulent flows, which are often classified through the importance of their *Reynolds number*, will experience irregular velocity fluctuations which will enhance diffusion. To understand this, the turbulence phenomenon needs further explanations and a few analogies are needed to describe its effect. Hinze (1959) explained that turbulent fluid motion is an irregular condition of flow in which the various quantities show a random variation with time and space coordinates, so statistically distinct average values can be discerned. Reynolds' idea (Osborne Reynolds, the famous physicist) was to differentiate averages and fluctuations with $u_i = \bar{u}_i + u'_i$ and $c = \bar{c} + c'$. Variables with overbars mean that we take temporal averages of the instantaneous value while u'_i and c' represent deviations of the instantaneous values toward this average and are respectively called the velocity fluctuations and concentration fluctuations. This process is also called *Reynolds' averaging*. Hence, Eq. (3.8) becomes:

$$\frac{\partial \bar{c}}{\partial t} + \bar{u}_i \frac{\partial \bar{c}}{\partial x_i} = \varepsilon_m \frac{\partial^2 \bar{c}}{\partial x_i \partial x_i} + \frac{\partial}{\partial x_i} (-\overline{c' u'_i}) \quad (3.9)$$

and the $(-\overline{c' u'_i})$ expresses additional mixing due to turbulent fluctuations. Elder (1959) assumed a fickian law for those turbulent fluxes and defined a turbulent diffusion coefficient:

$$-\overline{c' u'_i} \approx \varepsilon_{ij}^t \frac{\partial \bar{c}}{\partial x_j} \quad (3.10)$$

The left term, multiplied by the density ρ , represents mass fluxes per unit surface ($kg/m^2 s$) and are also called *Reynolds' fluxes*. Finally, assuming the analogy to Fick's law to be valid and the scale of the random motion of turbulence much greater than that of molecular motion, the three-dimensional turbulent solute transport equation in flow of a passive contaminant can be written as:

$$\frac{\partial c}{\partial t} = -u_i \frac{\partial c}{\partial x_i} + \frac{\partial}{\partial x_i} \left(\varepsilon_i \frac{\partial c}{\partial x_i} \right) \quad (3.11)$$

where t is the time, u_i are velocity components in direction x_i and ε_i are the diffusivities in the corresponding directions. In Eq. (3.11) the overbars denoting time averages are no longer needed and therefore dropped ; the scalar ε_i has replaced ε_{ij} , commonly referred to as the diffusion tensor, by taking the axes of the coordinate system as the principal axes of the diffusion tensor. This equation is the most general one, applying equally well to solid particles or to solute. However, for particles with higher mass and increasing fall velocity, care must be given to the effective field velocity which convects the particles and, similarly, to the effective diffusivity.

3.3 Navier-Stokes equations

As announced, Navier-Stokes equations can be derived from equation (3.7). The so-called Navier-Stokes equations gathers the conservation of mass, the conservation of momentum and the conservation of energy, irrelevant in this work. Equation (3.1) already presents the conservation of mass, where $\varphi = 1$. Momentum conservation replaces φ by the velocity vector $\mathbf{u}$ and expresses the forces which may act on the fluid in a CV (surface forces like the pressure, the normal and shear stresses or body forces like gravity, centrifugal and Coriolis forces). Since the form of the final equations is common, the complete derivation of these equations will not be given here (see e.g. Rodi, 1980).

The Reynolds-Averaged Navier-Stokes or RANS equations are the following :

$$\frac{\partial \bar{u}}{\partial x} + \frac{\partial \bar{v}}{\partial y} + \frac{\partial \bar{w}}{\partial z} = 0 \quad (3.12a)$$

$$\begin{aligned} \frac{\partial \bar{u}}{\partial t} + \frac{\partial}{\partial x} (\bar{u} \bar{u}) + \frac{\partial}{\partial y} (\bar{u} \bar{v}) + \frac{\partial}{\partial z} (\bar{u} \bar{w}) = & -\frac{1}{\rho} \frac{\partial \bar{p}}{\partial x} + \nu \nabla^2 \bar{u} \\ & - \left(\frac{\partial}{\partial x} \overline{u'u'} + \frac{\partial}{\partial y} \overline{u'v'} + \frac{\partial}{\partial z} \overline{u'w'} \right) \end{aligned} \quad (3.12b)$$

$$\begin{aligned} \frac{\partial \bar{v}}{\partial t} + \frac{\partial}{\partial x} (\bar{v} \bar{u}) + \frac{\partial}{\partial y} (\bar{v} \bar{v}) + \frac{\partial}{\partial z} (\bar{v} \bar{w}) = & -\frac{1}{\rho} \frac{\partial \bar{p}}{\partial y} + \nu \nabla^2 \bar{v} \\ & - \left(\frac{\partial}{\partial x} \overline{v'u'} + \frac{\partial}{\partial y} \overline{v'v'} + \frac{\partial}{\partial z} \overline{v'w'} \right) \end{aligned} \quad (3.12c)$$

$$\begin{aligned} \frac{\partial \bar{w}}{\partial t} + \frac{\partial}{\partial x} (\bar{w} \bar{u}) + \frac{\partial}{\partial y} (\bar{w} \bar{v}) + \frac{\partial}{\partial z} (\bar{w} \bar{w}) = & -\frac{1}{\rho} \frac{\partial \bar{p}}{\partial z} + \nu \nabla^2 \bar{w} \\ & - \left(\frac{\partial}{\partial x} \overline{w' u'} + \frac{\partial}{\partial y} \overline{w' v'} + \frac{\partial}{\partial z} \overline{w' w'} \right) \end{aligned} \quad (3.12d)$$

where p is the pressure; g is the gravity constant and ν the molecular viscosity. A Reynolds averaging of the local velocity components has been used, where $\bar{u}$, $\bar{v}$ and $\bar{w}$ are the Reynolds averaged velocities; and u' , v' and w' are their turbulent fluctuations, whose products define Reynolds turbulent stresses. As presented, those equations neglect the shear stresses due to molecular viscosity, since they are usually several order of magnitude smaller than the Reynolds stresses. Body forces other than gravity are not stated in these expressions as no use will be made of e.g. Coriolis in this work.

The instantaneous continuity and N-S equations form a closed set of four equations with four unknowns u, v, w and p but are difficult to solve directly (Versteeg & Malalasekera, 1995). Engineers are content to focus their attention on certain mean quantities. However, in performing the time-averaging operation on the momentum equations we throw away all details concerning the state of the flow contained in the instantaneous fluctuations. As a result we obtain six additional unknowns, the Reynolds stresses, in the time averaged momentum equations. The complexity of turbulence usually precludes simple formulae for the extra stresses. It is the main task of turbulence modelling to build a sufficiently reliable way to compute them.

Boussinesq eddy viscosity concept

In the RANS equations presented above, no clue is given about the value of the Reynolds turbulent stresses, $\tau_{ij} = -\overline{\rho u'_i u'_j}$ in tensor notations ($u_1 = u$, $u_2 = v$ and $u_3 = w$). This term formally describes the impact of the turbulence on the mean flow field and for which an expression in terms of mean flow quantities has to be found in order to close the equations.

The *Boussinesq eddy-viscosity concept* is applied in most turbulence models and express a proportionality between those turbulent stresses and the local velocity gradients, in analogy with molecular viscosity:

$$\frac{\tau_{ij}}{\rho} = \nu_t \left(\frac{\partial \bar{u}_i}{\partial x_j} + \frac{\partial \bar{u}_j}{\partial x_i} \right) - \frac{2}{3} \delta_{ij} k \quad (3.13)$$

where ν_t is called the *turbulent viscosity* and $k = 1/2 (\overline{u'^2} + \overline{v'^2} + \overline{w'^2})$ is the kinetic

turbulent energy. The second term on the right hand side is normally a very small term and can often be neglected, as it will be in this study.

3.4 Shallow water equations

At the time being, few industrial tools for solving directly the Reynolds-averaged 3-D Navier-Stokes equations for free surface flows are available. One of the main difficulties is the free surface itself, which causes the computational domain to vary in time. Following the level of approximation required, and the complexity of the flow to model, a depth-averaged version of the 3-D RANS equations may be sufficient for hydrodynamic modelling. Those equations are the *shallow-water equations* given by Barré de Saint-Venant one century ago, which present a hydrostatic pressure assumption. The derivation of the Shallow Water equations is mainly based on the Leibnitz rule (Hervouet & Van Haren, 1996) and the new variables obtained are mean values over the depth. The two components of the horizontal depth-averaged velocity, U and V , will thus be:

$$U = \frac{1}{h} \int_{z_b}^{z_w} \bar{u} dz \text{ and } V = \frac{1}{h} \int_{z_b}^{z_w} \bar{v} dz \quad (3.14)$$

where z_b is the bottom elevation and z_w is the free surface. The depth h is equal to $z_w - z_b$ (see figure 3.2) and is unknown.

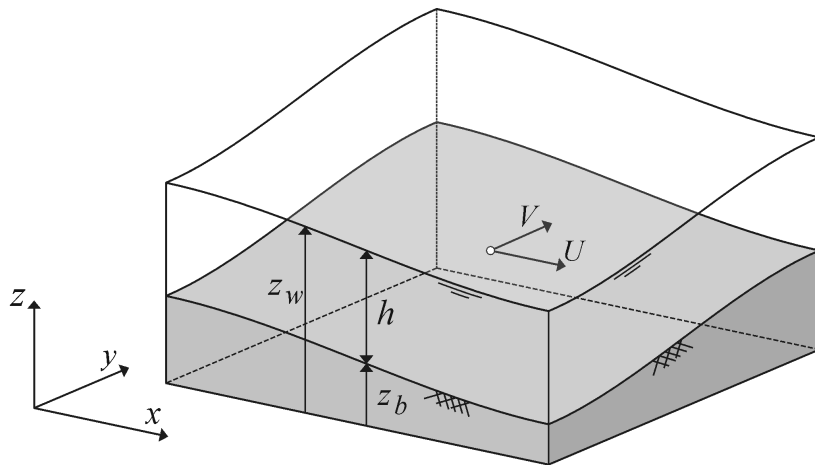


Figure 3.2: Definition of bottom topography and free surface

3.4.1 Preliminaries

Hypothesis

The full derivation of the equations would be far too long, but recalling the underlying hypothesis and approximations is important. First, the vertical accelerations and shear stresses are neglected in equation (3.12d), and this goes along with the hydrostatic pressure assumption:

$$-\frac{1}{\rho} \frac{\partial \bar{p}}{\partial z} - g = 0 \quad (3.15)$$

i.e. $\bar{p} = \rho g (z_w - z) + p_a$ where the pressure p_a at the free-surface is the (constant) atmospheric pressure. As a consequence the vertical velocity will be assumed to remain small and will have no specific equation. An often overlooked consequence is that steep slopes should be avoided when they are facing the flow.

Further, the bottom and the free surface are considered impermeable:

$$\frac{\partial z_w}{\partial t} + \mathbf{U}_w \cdot \mathbf{n} = 0 \text{ and } \frac{\partial z_b}{\partial t} + \mathbf{U}_b \cdot \mathbf{n} = 0 \quad (3.16)$$

where $\mathbf{U}_w$ is the velocity vector at the free surface and $\mathbf{U}_b$ at the bottom. $\mathbf{n}$ is a vector normal to the surfaces with components $(\partial z / \partial x, \partial z / \partial y, -1)$ for the free surface.

As announced already, Leibnitz rule is a major tool in the derivation of the shallow water equations, where it is used to invert the integration and differentiation operators :

$$\frac{\partial}{\partial x} \int_{z_b}^{z_w} f(x, y, z) dz = \int_{z_b}^{z_w} \frac{\partial f}{\partial x} dz + f(x, y, z_w) \frac{\partial z_w}{\partial x} - f(x, y, z_b) \frac{\partial z_b}{\partial x} \quad (3.17)$$

Most of the development is made by simply integrating the Navier-Stokes equations along the depth z and applying all the prescribed hypothesis. However, the averaging procedure of the non-linear terms will give rise to expressions still containing the original 3-D variables. For instance, after some algebraic operations:

$$\frac{\partial}{\partial y} \int_{z_b}^{z_w} \bar{u} \bar{v} dz = \frac{\partial}{\partial y} (h U V) + \frac{\partial}{\partial y} \int_{z_b}^{z_w} (\bar{u} - U) (\bar{v} - V) dz \quad (3.18)$$

The second part of this expression cannot be reduced further when there are heterogeneities in the velocity over the vertical. These terms are the so-called *dispersion* terms and are generally treated as additional diffusion. Indeed, even if these are advection terms arising from the inhomogeneity of the horizontal velocity over the vertical, they express

fluctuations around the mean (here spatial) and are analogous to Reynolds stress. The existence of dispersion is clearly a limitation of the Shallow water equations: average values have no physical meaning if the horizontal velocity varies too much along the vertical.

3.4.2 Synthesis

After some transformations, the final expressions read (Hervouet, 2003):

Continuity

$$\frac{\partial h}{\partial t} + \frac{\partial}{\partial x_j} (hU_j) \quad (3.19)$$

Momentum in x and y directions

$$\frac{\partial (hU_i)}{\partial t} + \frac{\partial (hU_i U_j)}{\partial x_j} = -hg \frac{\partial z_w}{\partial x_i} + \frac{\partial}{\partial x_j} \left(h\nu_t \left(\frac{\partial U_i}{\partial x_j} + \frac{\partial U_j}{\partial x_i} \right) \right) + \underbrace{\left(\tau_{iz}^{surf} - \tau_{iz}^{bot} \right)}_{hF_i} \quad (3.20)$$

In this equation the subscripts i and j stand for 1 or 2 with $U = U_1$ and $V = U_2$ (respectively x and y directions). ν_t combines the effect of kinematic and turbulent viscosity, but also dispersion, for whom no explicit modelling is proposed. The term hF_i contains the friction forces and several supplementary source terms, like the effect of wind, etc..., that stem from the depth-averaging procedure.

3.4.3 Bottom friction modelling

The shear stress at the free surface is considered negligible, but a very important term in the equations, especially in river flows, is the bottom friction. Friction force appears in the momentum equation as a source term and the bed shear stress τ_b in the direction x is expressed as:

$$\frac{\tau_b}{\rho} = \tau_{xz}^{bot} = ghS_{fx} \quad (3.21)$$

with S_{fx} the energy slope in the x direction (Yen, 2002). Flow resistance has several origins, as classified by Rouse (1965), but this work will focus on the effect of surface or skin friction and form resistance or drag. The general 1-D expression of bed friction is given by the formula $\tau_b = \rho C_f U^2$ ($kg/m\ s^2$) and C_f is a dimensionless friction coefficient and U is, in this case, the mean section velocity. The 2-D equivalent of this formula reads $\boldsymbol{\tau}_b = \rho C_f \|\mathbf{U}\| \mathbf{U}$. The above formula is often found in literature as $\tau_b = 1/2 \rho c_f U^2$, with c_f the dimensionless friction coefficient (see e.g. Liggett, 1994).

The friction coefficient C_f is not frequently used in hydraulics and many models of the head loss due to bed friction, expressed as energy slope S_f have been proposed.

Among them we find Manning's equation, developed empirically (Manning, 1889) or Darcy-Weisbach's resistance formula. Expressed in uniform flow ($S_f = S_0$, energy slope equal to the bed slope) and relating open-channel cross-sectional flow velocity, U , to resistance coefficients, they read:

$$U = \frac{R^{2/3}}{n} S^{1/2} \text{ (Manning)} \quad (3.22)$$

$$U = \sqrt{\frac{8g}{f}} \sqrt{RS} \text{ (Darcy-Weisbach)} \quad (3.23)$$

in which n and f are the Manning and Darcy-Weisbach resistance coefficients, respectively; R = hydraulic radius of the flow and S = slope. From the previous expressions, one can deduce the general link between the resistance coefficients:

$$(C_f)^{1/2} = \sqrt{\frac{f}{8}} = \frac{n\sqrt{g}}{R^{1/6}} = \frac{\sqrt{gRS}}{U} \quad (3.24)$$

Mostly, the value of the friction coefficients will be unknown and will have to be estimated by means of measurements or by using catalogues (e.g. Chow, 1959 for n or the Moody diagram for f). Manning's n has the advantage of being nearly a constant almost independent of the flow depth, Reynolds number or the relative roughness $\mathbf{K} = k_s/R$ (where k_s is the equivalent wall surface roughness) for fully developed turbulent flow over a rigid rough surface. On the other hand, the Darcy-Weisbach relation reflects more clearly the relation between roughness and turbulence regime. It appears appropriate to refer Weisbach's f for point resistance while Manning's n for cross sectional and reach resistance coefficients.

According to the manning equation, the friction force will appear in the momentum equations (3.20) as a source term having the following form:

$$F_x = -gS_{fx} = -g \frac{n^2 U \sqrt{U^2 + V^2}}{h^{4/3}}, \quad F_y = -gS_{fy} = -g \frac{n^2 V \sqrt{U^2 + V^2}}{h^{4/3}} \quad (3.25)$$

since in 2-D flows, R is replaced by the water depth h , which has an impact on n values.

Turbulence models, as presented in the next subsection, often request another friction parameter which is a kinematic shear measure called the bed shear velocity, defined for 1-D flows as:

$$u_* = \sqrt{\frac{\tau_b}{\rho}} = \sqrt{gRS_0} \quad (3.26)$$

Or computed differently, for 2-D flows:

$$u_* = \sqrt{C_f (U^2 + V^2)} \quad (3.27)$$

3.4.4 Turbulence modelling

As explained already, the action of turbulence on mean flow quantities is associated with a turbulent diffusion process, where the turbulent shear stresses are assumed to be proportional to the Reynolds-averaged velocity gradients (Eq. (3.13)). Turbulence modelling now consists of finding an expression for the turbulent viscosity. The turbulent, or "eddy", viscosity ν_t is either assumed constant; or estimated by an algebraic equation or by a one- or two-equations model. The interested reader might refer to Rodi (1980) for a full literature review.

Zero-equation turbulence models

One major simplification to turbulence modelling consists of specifying a constant turbulent viscosity for the whole flow field. For shallow-water flows, one often considers a direct link between turbulence and bed friction:

$$\nu_t = \lambda u_* h \quad (3.28)$$

where the proportionality constant in the latter case not only takes into account the real turbulence but also the vertical flow inhomogeneities; a value of $\lambda = 0.135$ has been prescribed by Rodi (1980) for wide laboratory flumes; while Wark *et al.* (1990) found 0.16 for laboratory flumes or 0.6...2.0 in natural rivers.

This model is only valid for simple flows, such as the calculation of quasi-2-D large waterbodies, where turbulence only plays a minor role. When the turbulence model becomes important, in more complex flow situations, the constant eddy viscosity model is far too simple. A special form of (3.28) was proposed by Elder (1959), who extended Taylor's analyses to open channel:

$$\nu_{t,long} = \alpha_L u_* h \text{ and } \nu_{t,lat} = \alpha_l u_* h \quad (3.29)$$

where lateral ($\alpha_l \cong 0.23$) and longitudinal ($\alpha_L \cong 6$) turbulent mixing are dissociated.

Mixing length model

If transverse shearing is suspected as the main turbulence source, the algebraic turbulence

model proposed by Prandtl through dimensional analysis, the Prandtl's mixing length model, may be used:

$$\nu_t = l_m^2 \left| \frac{\partial U}{\partial y} \right| \quad (3.30)$$

where the mixing length l_m is analogous to the mean free path of molecular motion and, hence, to some turbulent scale. The hope is that the mixing length can be related to the scale of the problem. Some work (e.g. Rodi, 1980) has been done to determine appropriate values of the mixing length for different flow situations.

One- and two-equation (k - ϵ) models

The k - ϵ , by Launder & Spalding (1974), is the most commonly used turbulence model, because it is sufficiently general to treat many practical flow applications. Rastogi & Rodi (1978) proposed a depth-averaged version which is implemented in *Telemac-2D*.

The principle of the k - ϵ model is to express the turbulent viscosity as a function of the turbulent kinetic energy k and its dissipation rate ϵ :

$$\nu_t = c_\mu \frac{k^2}{\epsilon} \quad (3.31)$$

Here one has to solve extra transport equations to estimate the depth-averaged turbulent kinetic energy and dissipation. Compared to the 3-D equations, the vertical-averaging procedure gives rise to supplementary production terms in both equations, $P_{k\nu}$ for the kinetic energy and $P_{\epsilon\nu}$ for the dissipation rate, due to bed shear:

$$\frac{\partial k}{\partial t} + U \frac{\partial k}{\partial x} + V \frac{\partial k}{\partial y} = \frac{1}{h} \frac{\partial}{\partial x} \left(h \frac{\nu_t}{\sigma_k} \frac{\partial k}{\partial x} \right) + \frac{1}{h} \frac{\partial}{\partial y} \left(h \frac{\nu_t}{\sigma_k} \frac{\partial k}{\partial y} \right) + P_h - \epsilon + P_{k\nu} \quad (3.32a)$$

$$\frac{\partial \epsilon}{\partial t} + U \frac{\partial \epsilon}{\partial x} + V \frac{\partial \epsilon}{\partial y} = \frac{1}{h} \frac{\partial}{\partial x} \left(h \frac{\nu_t}{\sigma_\epsilon} \frac{\partial \epsilon}{\partial x} \right) + \frac{1}{h} \frac{\partial}{\partial y} \left(h \frac{\nu_t}{\sigma_\epsilon} \frac{\partial \epsilon}{\partial y} \right) + \frac{\epsilon}{k} (c_{1\epsilon} P_h - c_{2\epsilon} \epsilon) + P_{\epsilon\nu} \quad (3.32b)$$

where σ_k , σ_ϵ , $c_{1\epsilon}$ and $c_{2\epsilon}$ are calibrated constants, whose values are given in Table 3.1 and P_h is the production of depth averaged kinetic turbulent energy by shear:

$$P_h = \nu_t \left[2 \left(\frac{\partial U}{\partial x} \right)^2 + 2 \left(\frac{\partial V}{\partial y} \right)^2 + \left(\frac{\partial U}{\partial y} + \frac{\partial V}{\partial x} \right)^2 \right] \quad (3.33)$$

Table 3.1: *The constants of the depth-averaged k - ϵ model (Rodi, 1993)*

c_μ	σ_k	σ_ϵ	$c_{1\epsilon}$	$c_{2\epsilon}$
0.09	1.0	1.3	1.44	1.92

The above-mentioned source terms $P_{k\nu}$ and $P_{\epsilon\nu}$ are assumed to be related to the shear velocity (Rodi, 1993), since it expresses turbulence production due to bed friction:

$$P_{k\nu} = c_k \frac{u_*^3}{h} \text{ with } c_k = \frac{1}{\sqrt{C_f}} \quad (3.34a)$$

$$P_{\epsilon\nu} = c_\epsilon \frac{u_*^4}{h^2} \text{ with } c_\epsilon = 3.6 \frac{c_{\epsilon 2} \sqrt{c_\mu}}{C_f^{3/4}} \quad (3.34b)$$

where C_f is the friction coefficient and u_* is the bottom friction velocity, that has been defined earlier.

A simplification of the k - ϵ turbulence model is worth to mention in this work, as it will be referred to later on (Chapter 6). Nadaoka & Yagi (1998) developed a depth-averaged modelling of periodical turbulent structures, founded on a one-equation model that reads:

$$\nu_t = c'_\mu \sqrt{k} L_d \quad (3.35)$$

The turbulent kinetic energy k is estimated through the transport equation (3.32a) ; while the turbulence length scale L_d is assumed to be proportional to the water depth h , as the bed friction is the main turbulence source. The dissipation ϵ , requested to solve (3.32a), is given by:

$$\epsilon = c_d \frac{k^{3/2}}{L_d} \quad (3.36)$$

where c_d is a constant (a value of 0.17 is recommended) and $c'_\mu = c_\mu / c_d = 0.17 / 0.09$ relates directly k to the length-scale L_d .

3.5 Sediment transport in open channel flows

Flowing water exerts hydrodynamic forces on sediment particles and it is the interplay between the resultant of these forces and the particle's weight which represent the physical mechanism of the sediment motion. Many efforts have been made in the past century to

describe theoretically this complex phenomenon, and only the most relevant elements will be given here. It is however necessary for the reader to be able to understand (1) what is the mode of transport? ; (2) When and why does erosion and deposition begin to occur on the bed? ; (3) how much solid material is transported and what are the model laws able to describe it? Further, a few words are requested to describe the effect on the flow of various bedforms, since they appeared in the sedimentological experiments led in the laboratory flume. It will indeed be useful to distinguish skin friction effects from form drag.

All definitions given in this section will become useful throughout this dissertation, especially when describing the sedimentological numerical model *Sisyphe*, presented in Chapter 6.

3.5.1 Transport mode classification

Sediment transport loads can usually be classified as per sediment transport mode. The total load which corresponds to all bed sediment particles moving with flowing water can be divided into *bed load* and *suspended load* according to the mechanism for transport. The bed load consists of sediment particles that slide, roll, or saltate in the layer several particle sizes above the bed surface. The suspended load is composed of sediment particles that move in suspension in the water column above the bed load layer. Its weight is continuously supported by the turbulence of flow.

Bed load

Due to the weight of water, the flow produces on the bottom a tangential force per unit bed area known as the bed shear stress τ_b already defined earlier in equation (3.26). The bed shear stress or *unit tractive force*, and thus the shear velocity, provides a direct measure of the flow intensity and its ability to entrain and transport sediment particles. This shear stress on the channel bed is used in most of the bed load predictive formulas. When all of the transport is directed in the streamwise direction, the volume bed load transport rate per unit width is given by q_b [$L^3/L/T$]. In general, bed load transport is vectorial and $\mathbf{Q}_b \equiv (Q_{bx}, Q_{by})$ are the x (longitudinal) and y (lateral) direction components respectively [L^3/T].

It should be noted that this transport mechanism does not occur for all sediment sizes ; or at least is more the prevailing mode of transport for gravel beds than fine sand beds. Garcia (2008) argues that for sediment diameter sizes $d_{50} < 180\mu m$ (fine sand and finer)

at initial motion, the moving particles go directly into suspension following initiation of motion. The d_{50} is called the median diameter and corresponds to the diameter for which 50% in mass of the sediments are smaller.

Suspended load

There obviously exists an active interchange between the suspended load and bed load, but also between the bed load and the bed itself. When the entire motion of solid particles is such that they are surrounded by fluid, they are said to move in suspension. These particles are kept in suspension due to turbulence. The action of turbulence on sediments may be assumed to be analogous to a diffusion-dispersion process (Graf, 1971) and described in the previous sections. Suspended particle fluxes have thus two components: one associated with the mean flow motion and one associated with the turbulence of the flow. Mass conservation of suspended sediment of uniform size in a turbulent flow is then described with an adaptation of equation (3.11):

$$\begin{aligned} \frac{\partial c}{\partial t} + u \frac{\partial c}{\partial x} + v \frac{\partial c}{\partial y} + (w - w_s) \frac{\partial c}{\partial z} = \\ \frac{\partial}{\partial x} \left(\varepsilon_{s,x} \frac{\partial c}{\partial x} \right) + \frac{\partial}{\partial y} \left(\varepsilon_{s,y} \frac{\partial c}{\partial y} \right) + \frac{\partial}{\partial z} \left(\varepsilon_{s,z} \frac{\partial c}{\partial z} \right) \end{aligned} \quad (3.37)$$

Once again, the Reynolds flux $\overline{u'_i c'}$ is assumed diffusive in nature, with the hypothesis that they are proportional to gradients in sediment concentration. The sediment eddy diffusivity tensor is assumed to represent a diagonal matrix and equation (3.37) accounts for anisotropic turbulence, with $\varepsilon_{s,i}$ the kinematic eddy diffusivity for the sediment particles in x, y, z directions. The main assumption in the above equation is that the sediment particles follow the fluid particles, except in the vertical z direction, where the effect of gravity introduces a relative velocity denoted by the sediment fall velocity w_s in quiescent water. Stokes' law for falling spheres is applicable to very fine sand, as used in this study and gives (Chang, 1988):

$$w_s = \frac{gd^2}{18} \left(\frac{\rho_s - \rho}{\mu} \right) \quad (3.38)$$

with ρ_s the mass density of the submerged sediment (kg/m^3) and μ the dynamic viscosity of water ($kg/m \cdot s$).

Consider the local axis defined in Figure 3.3, with u which denotes the mean flow velocity (m/s) at a point located at a distance z' normal to the bed, and c which is the mean volumetric concentration of suspended sediment (m^3/m^3), with a a reference level.

The streamwise volume transport rate of suspended sediment per unit width is given by:

$$q_s = \int_a^h u c dz' \quad (3.39)$$

Vertical distribution of suspended matter

In a wide, rectangular open channel with erodible bed and uniform steady flow where the z' -coordinate is quasi-vertical, implying a low channel slope S , the equilibrium concentration c is function of z alone, as showed in Fig. 3.3. Under this assumption, the diffusion equation (3.37) reduces to:

$$w_s \frac{dc}{dz} + \frac{d}{dz} \left(\varepsilon_{s,z} \frac{dc}{dz} \right) = 0 \quad (3.40)$$

After integration, this equation indicates that $(w_s c + \varepsilon_{s,z} dc/dz) = \text{Constant}$ in the water column above $z = a$. The net vertical flux is reduced to 0 at the water surface which makes that the constant equals zero and thus

$$w_s c + \varepsilon_{s,z} \frac{dc}{dz} = 0 \quad (3.41)$$

The $w_s c$ term represents the downward rate of deposition of suspended sediment under the influence of gravity ; the second term represents sediment transport due to turbulent diffusion which is directed upward as long as $dc/dz < 0$. Turbulence diffuses sediment from zones of high concentration (near the bed) to zones of low concentration (near the water surface). General boundary conditions for Eq. (3.41) are given by Fredsoe & Deigaard (1992):

$$-\varepsilon_{s,z} \frac{dc}{dz} \Big|_{z=a} = w_s E_s \quad (3.42a)$$

$$\varepsilon_{s,z} \frac{dc}{dz} + w_s c \Big|_{z=h} = 0 \quad (3.42b)$$

with E_s the dimensionless entrainment of bed sediment into suspension ($E_s \equiv E_b/w_s$), defined later.

An estimation of the sediment diffusivity is requested to find a solution to Eq. (3.41). von Karman (1934) involved the use of the Reynolds analogy, and pointed out that the diffusion coefficient for mass transfer was related to that for momentum transfer in turbulent flow. The momentum flux is equivalent to the shear stress and related to the velocity gradient (Chang, 1988) by:

$$\tau_{xz} = \rho \nu_{t,z} \frac{du}{dz} = \text{momentum flux} \quad (3.43)$$

Fig. 3.3 also represents the hypothetical logarithmic vertical velocity profile established, both experimentally and from dimensional arguments (Schlichting, 1979 ; and Nezu & Rodi, 1986):

$$\frac{u}{u_*} = \frac{1}{\kappa} \ln \left(\frac{z}{z_0} \right) \quad (3.48)$$

where κ is the von Karman constant (approximated at $\kappa \cong 0.41$) and z_0 is the bed roughness length which depends on the hydraulic regime. In reality, the logarithmic velocity profile as presented is not fully valid up to $z = h$ (see e.g. Bradshaw, 1976). It is strictly valid only in the so-called *inner region* defined by Graf (1998) as $z/h \leq 0.2$. However, if this profile is used, Eqs (3.43) - (3.45) give a parabolic vertical distribution of eddy viscosity by (Graf, 1971):

$$\varepsilon_{s,z} = \sigma_{t,z} \nu_{t,z} = \sigma_{t,z} \kappa u_* \frac{z}{h} (h - z) \quad (3.49)$$

Substituting Eq. (3.49) in Eq. (3.41) and integrating provides the equilibrium vertical concentration developed by Rouse (1937):

$$\frac{c}{c_a} = \left(\frac{h - z}{z} \frac{a}{h - a} \right)^{z_*} \quad (3.50)$$

and z_* denotes the dimensionless Rouse Number:

$$z_* = \frac{w_s}{\kappa u_*} \quad (3.51)$$

The Rousean formulation predicts sediment concentration relative to a near-bed concentration c_a , which has to be estimated as a function of flow parameters and sediment characteristics. Profiles of suspended sediment obtained from laboratory observations are plotted in Rousean form in Fig. 3.4 (Vanoni, 2006). The Rouse number z_* stands as parameter for drawing the different curves. The vertical distribution of concentration is more uniform for small values of z_* , that is, for finer sediments or more turbulent flow. The Rouse solution presents serious shortcomings, as it is based on a dubious parabolic expression for $\varepsilon_{s,z}$. No experimental evidence supports a sediment concentration going down to zero at the water surface as expressed by Eq. (3.50). Alternatives to the Rousean formulation exist, based on different estimations for $\varepsilon_{s,z}$ (Verbanck et al., 2002), but will not be detailed here.

Bed evolution

A formulation of a sediment bed level evolution was first proposed by Exner (1925), but requests to understand how sediment mass conservation can be formulated. Consider the

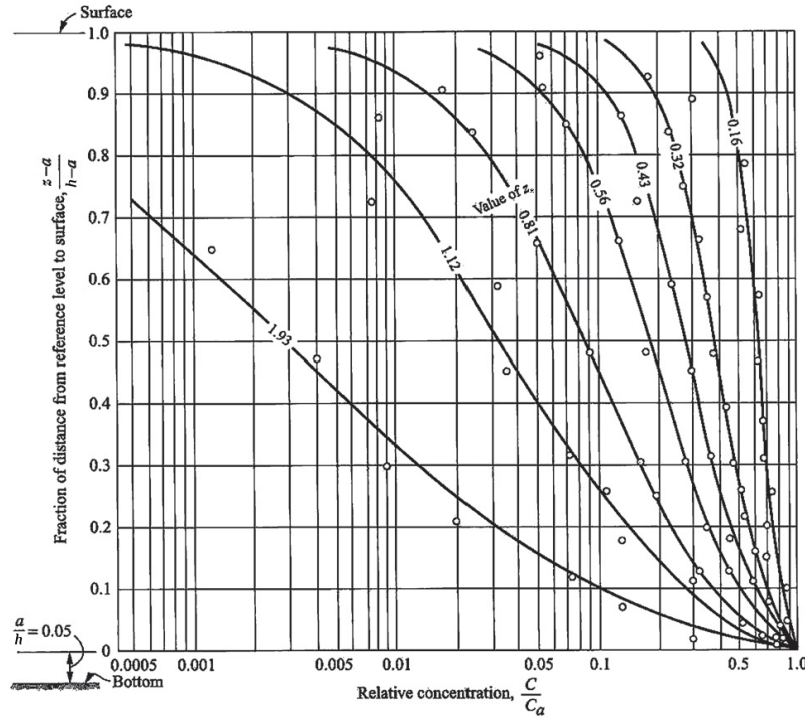


Figure 3.4: *Laboratory observations of suspended sediment distribution (following Vanoni, 2006)*

definition diagram for a sediment-laden, uniform, open-channel flow shown on Fig. 3.3 where it becomes clear that bed load transport only happens in a small part of the sand-water column, the so-called *bed layer*. This diagram formulates the interaction between bed sediment and the water column through erosion and deposition. The volume rate of erosion of bed material into suspension per unit time per unit bed area is denoted as E_b [$L^3/L^2/T$]. On the other side, deposition on the bed is by means of settling. The deposition rate D_b (m/s) at the bed is obtained by computing the component of vertical downward flux that is actually directed normal to the bed:

$$D_b = w_s c_a \quad (3.52)$$

where c_a is a near-bed value of the volumetric sediment concentration.

Because of variations in flow conditions and channel properties, the sediment transport in natural rivers usually is not in states of equilibrium. Bed elevation change in a portion of river bottom where the bed material is taken to have a constant porosity n and taking into account both bed load transport and sediment erosion into and from suspension gives,

using a control volume approach:

$$(1 - n) \frac{\partial z_b}{\partial t} + \text{div}(\mathbf{Q}_b) = (D_b - E_b) \quad (3.53)$$

To compute this equation, bed load transport needs to be determined, along with the near-bed suspended sediment concentration c_a and sediment entrainment into suspension E_s .

3.5.2 Erosion and deposition criterias

The evolution of the sediment bed is controlled by erosion and/or deposition phenomena. All sediment transport mathematical models are expected to give expressions for entrainment rate E_b and deposition rate D_b which requires to understand further how sediments are actually lifted from the bed.

Basically, a particle will be moved or entrained if the applied forces overcome the resistance. Those forces consist of the hydrodynamic drag, the hydrodynamic lift, and the submerged weight. Authors have been interested in defining threshold condition for sediment movement in the past years, but random fluctuations in the presence of turbulent flow prevent the clear definition of a critical condition for motion.

Shields (1936) is often considered as a pioneer in this motion threshold definition. He used the critical shear stress $(\tau_b)_{cr}$ concept and linked the point of incipient motion for a given sediment size to the shear Reynolds number $Re_* = u_* d / \nu$:

$$\frac{(\tau_b)_{cr}}{(\gamma_s - \gamma) d} = \mathfrak{F} \left(\frac{u_* d}{\nu} \right) \quad (3.54)$$

where γ_s is the specific grain weight ($= \rho_s g$) and γ is the water specific weight. Fig. 3.5 proposes a Shields diagram by Vanoni (1964) where the functional relationship defined in (3.54) is plotted.

Shields' work has been checked by many experimentalists and is by now widely accepted. However, the presence of the shear velocity in the ordinate and abscissa makes it quite uneasy to use. Among others, Van Rijn (1984a & 1984b) proposed to express the motion threshold (Fig. 3.6) as a function of the particle parameter d_* defined as:

$$d_* = d_{50} \left[\frac{(s - 1) g}{\nu^2} \right]^{1/3} \quad (3.55)$$

with $s = \rho_s / \rho$ the relative density of sand and this time, the $(\tau_b)_{cr} = \rho u_{*,cr}^2$ that can be readily computed when the properties of the water and the sediment are given. He

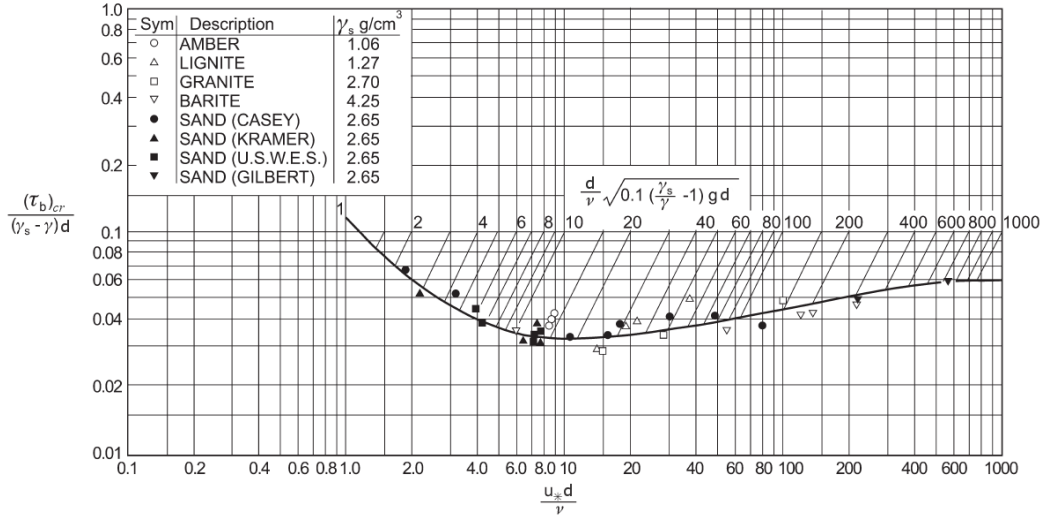


Figure 3.5: Shields diagram for initiation of motion (following Vanoni, 1964)

defined the critical Shields parameter as:

$$\theta_{cr} = \frac{u_{*,cr}^2}{(s-1)gd_{50}} \quad (3.56)$$

and he prescribed for small-sized sands:

$$\theta_{cr} = \frac{0.24}{(d_*)} \text{ for } d_* \leq 4 \quad (3.57)$$

The same figure also presents the flow conditions at which initiation of suspension will occur. Bagnold (1966) stated that the critical bed-shear velocity ($u_{*,crs}$) for initiation of suspension was equal to the particle fall velocity (w_s), counting on the fact that the vertical turbulent intensity destined to exceed the particle fall velocity was of the same order as the bed-shear velocity. Engelund (1965) derived:

$$\frac{u_{*,crs}}{w_s} = 0.25 \quad (3.58)$$

but Van Rijn (1984b) defined a zone of initiation of suspension based on experimental research at the Delft Hydraulics Laboratory and:

$$\frac{u_{*,crs}}{w_s} = \frac{4}{d_*}, \text{ for } 1 < d_* \leq 10 \quad (3.59)$$

$$\frac{u_{*,crs}}{w_s} = 0.4, \text{ for } d_* > 10 \quad (3.60)$$

Those definitions usually differ from one author to another and it is more a probabilistic approach to adress threshold conditions for initiation of motion and entrainment into

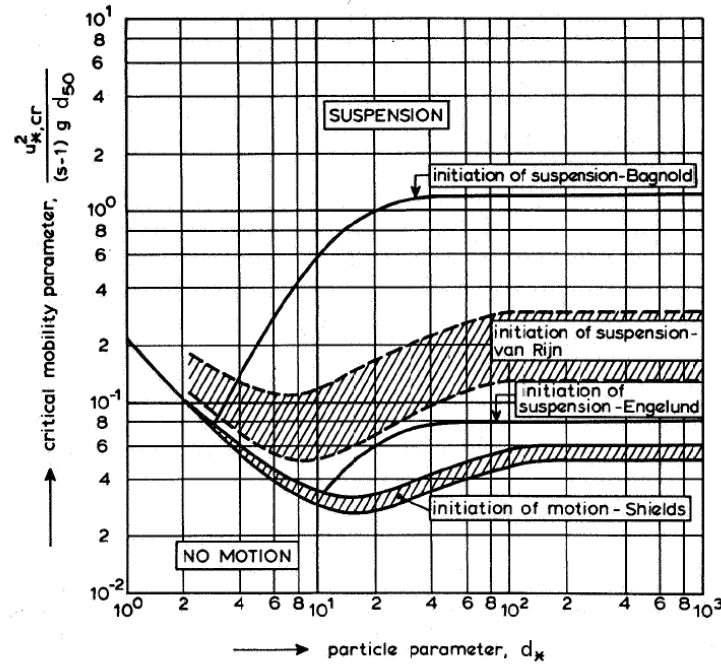


Figure 3.6: *Initiation of motion and suspension (Van Rijn, 1984b)*

suspension (see e.g. Lopez & Garcia, 2001). Actually, the Shields curve represents a critical stage at which only a minor part (say 1-10%) of the bed surface is moving (sliding, rolling, and colliding). Taylor & Vanoni (1972) explain further that as the size of sediment grain decreases, the dimensionless critical shear stress increases more slowly than one would infer by extrapolating the Shields curve. Even Van Rijn (2007) proposed a more accurate Shields parameter for fine sand beds, based on data of Miller *et al.* (1977):

$$\theta_{cr} = 0.115 (d_*)^{-0.5} \quad \text{for } d_* < 4 \quad (3.61)$$

which gives smaller shear stress than for his earlier definition in Eq. (3.57).

It is now easy to understand that E_s , the dimensionless sediment entrainment rate, is expected to be a function of boundary shear stress τ_b and sediment related parameters. Actually, prescribing E_s or alternatively another appropriate boundary condition such as the value of c_a to compute D_b in Eq. (3.53) is the main problem in modelling suspended sediment transport for non-equilibrium situations. Celik & Rodi (1998) stated that the sediment net vertical flux is expressed as the difference between deposition to and entrainment from the bed and, while the deposition rate is known from the local concentration at

the interface and settling velocity, the entrainment is assumed to occur at the same rate as it does under equilibrium conditions, provided sufficient sediment material is available:

$$(E - D)_{z=a} = w_s (c_a^* - c_a) \quad (3.62)$$

and c_a^* is defined as the equilibrium near-bed concentration. A number of relations are available in the literature for estimating the entrainment rate of sediment into suspension (and thus the reference concentration c_a^* for the equilibrium case), with formulations by Einstein (1950), Engelund & Fredsoe (1976), van Rijn (1984b), but also Zyserman & Fredsoe (1994), who proposed this empirical relation, which is used in the *Telemac* software:

$$c_a^* = \frac{0.331 (\theta_s - \theta_{cr})^{1.75}}{1 + 0.72 (\theta_s - \theta_{cr})^{1.75}} \quad (3.63)$$

In this formulation, the near-bed equilibrium concentration c_a^* is assigned to a reference level $a = 2d_{50}$. The θ_s is a mobility parameter given by $\theta_s = u_*^2 / (s - 1)gd_{50}$ while θ_{cr} is given by Eq. (3.56).

On the other side, the deposition flux $D_b = w_s c_a$ is usually determined by relating c_a to the depth-averaged suspended load concentration C through $c_a = \alpha_c C$ in which α_c is the adaptation or recovery coefficient (Wu, 2008). This coefficient is little known under nonequilibrium transport states and is often approximately evaluated using the Rouse distribution of suspended load concentration introduced in Eq. (3.50), established under equilibrium conditions. Minh Duc (1998) suggested hereafter:

$$\alpha_c = (h - a) / \int_a^h \left(\frac{h - z}{z} \frac{a}{h - a} \right)^{z_*} dz \quad (3.64)$$

with Lin (1984) who proposed:

$$\alpha_c = 3.25 + 0.55 \ln(z_*) \quad (3.65)$$

Garcia (2008) also suggests using the nonequilibrium depth-averaged suspended load concentration to express the reference concentration c_a . The streamwise suspended load q_s was already given in Eq. (3.39) and

$$q_s = \int_a^h u(z)c(z)dz \cong UC \quad (3.66)$$

with U the depth averaged streamwise velocity. Reducing with the aid of (3.50) and considering a logarithmic vertical velocity profile defined in Eq. (3.48) in which $z_0 = k_c/30$

for hydraulically rough flow and k_c is a general roughness coefficient, the c_a concentration is given finally by:

$$c_a = \frac{UC}{u_*} \frac{\kappa}{I} \quad (3.67)$$

in which $I = J_1 \ln \left(30 \frac{z}{k_c} \right) + J_2$. J_1 and J_2 are defined as follows (Garcia, 2008):

$$J_1 = \frac{\delta_a}{0.216} I_1 = \int_{\delta_a}^1 \left[\frac{(1-\delta)/\delta}{(1-\delta_a)/\delta_a} \right]^{z_*} d\delta \quad (3.68a)$$

$$J_2 = \frac{\delta_a}{0.216} I_2 = \int_{\delta_a}^1 \left[\frac{(1-\delta)/\delta}{(1-\delta_a)/\delta_a} \right]^{z_*} \ln(\delta) d\delta \quad (3.68b)$$

in which I_1 and I_2 are the so-called *Einstein integrals* and $\delta = z/h$ with $\delta_a = a/h$.

3.5.3 Sediment transport capacity laws

Formulae for predicting sediment discharge can be found in abundance. Such formulae have been developed for noncohesive sediment in steady uniform flow. Because of the complexity of the sediment transport processes, prediction of the transport rate has not been accomplished following purely theoretical pursuit. All existing sediment transport formulae have been established relying on calibration using flume and field data supposedly under the steady uniform flow condition. As expressed earlier, very few, but well validated, formulae are presented in this section. More recent developments exist, but are not detailed here, as only well-established formulae were used.

Sediment loads are combined to provide the bed material load or *total load*:

$$q_t = q_s + q_b \quad (3.69)$$

where q_t is the sediment discharge per unit width. q_b is the aforementioned bed load transport rate for which a great number of empirical predicting laws are developed (see e.g. Chang, 1988). Those laws are mostly based on the shear stress concept but contain many uncertainties, looking at the disparities in the predictions from one formula to another. The suspended load q_s was found to be calculated through the resolution of a transport equation in the concerned flow layer and using boundary conditions which use the near-bed equilibrium concentration concept. Focus will now be brought on so-called *power-approach* formulas, which are used to compute the bed material load with all its components.

Bagnold's approach

Bagnold (1966) proposed to express sediment transport capacity by relation with the available power ω as supply of energy. He introduces the general power equation by equating the rate of doing work with the available power times the efficiency. A derived form of this equation will be presented in later chapters when analyzing experimental sediment transport and deposit results. The available power, per unit length and unit width, is given by

$$\omega = \frac{\gamma QS}{B} = \gamma hSU \quad (3.70)$$

in which B is the width and U the mean cross sectional velocity.

Engelund-Hansen formula

Engelund & Hansen (1967) applied Bagnold's stream-power concept and the similarity principle to obtain the following relation:

$$C_f q_t^* = 0.05 (\tau_*)^{5/2} \quad (3.71)$$

with

$$q_t^* = \frac{q_t}{\gamma_s \sqrt{(s-1)gd}}, \quad \tau_* = \frac{\tau_b}{(\gamma_s - \gamma)d} \quad (3.72)$$

$$C_f = \frac{gRS}{U^2} \quad (3.73)$$

where C_f is the general friction coefficient defined earlier in Eq. (3.24) which accounts for the effect of skin friction and form drag altogether, q_t^* is the dimensionless sediment discharge, and τ_* is the dimensionless shear stress (*Shields stress* or mobility parameter θ_s). Substituting (3.72) and (3.73) in (3.71) yields:

$$C_t = 0.05 \left(\frac{s}{s-1} \right) \frac{US}{[(s-1)gd]^{1/2}} \sqrt{\frac{RS}{(s-1)d}} \quad (3.74)$$

with $C_t = (Q_t/Q)$ the sediment concentration by weight. Engelund-Hansen relation is one of the simplest to use for sediment transport, and one of the most accurate. It is applied to streams with bed forms in accordance with the similarity principle. The following sections will present the same authors' method to determine how bedforms act on flow resistance.

Ackers-White formula

Based on Bagnold's stream-power concept, Ackers & White (1973) proposed a sediment

load equation in terms of dimensionless parameters:

$$C_t = cs \frac{d}{R} \left(\frac{U}{u_*} \right)^n \left(\frac{F_{gr}}{A_{aw}} - 1 \right)^m \quad (3.75)$$

where the so-called Ackers-White mobility number is given by:

$$F_{gr} = \frac{u_*^n u_*'^{1-n}}{\sqrt{Rgd}} \quad (3.76a)$$

$$u_*' = \frac{U}{\sqrt{32} \log \left(10 \frac{R}{d} \right)} \quad (3.76b)$$

The parameters n, m, c and A_{aw} were determined with best-fits of laboratory data, as functions of the dimensionless grain size d_* defined in Eq. (3.55):

Coefficient	$d_* > 60$	$60 \geq d_* > 1$
c	0.025	$\log(c) = 2.86 \log(d_*) - [\log(d_*)]^2 - 3.53$
n	0.0	$1 - 0.56 \log(d_*)$
A_{aw}	0.17	$\frac{0.23}{\sqrt{d_*}} + 0.14$
m	1.5	$\frac{9.66}{d_*} + 1.34$

Other relations for Total Bed-Material Load

Several other total sand-discharge formulae can be found in the literature, including those of Shen & Hung (1972), Kikkawa & Ishikawa (1978), Ranga-Raju *et al.* (1981), van Rijn (1984), Pacheco-Ceballos (1992), and Molinas & Wu (2001) among others. A more complete review and evaluation and ranking of various relations can be found in Yang (2005) or Garcia (2008).

3.5.4 Bedforms and their effect on the flow

Flow resistance is usually attributed to grain roughness and form roughness. The former is due to shear force on sand elements and the latter is attributed to the pressure difference in the presence of larger elements such as bedforms. Any such roughness may be expressed in terms of the resistance coefficients described earlier in section 3.4.3 (see e.g. Chang, 1988). In the case of a sand bed, the equivalent roughness k_s of a plane bed is usually only related to the largest particles of the bed material. van Rijn (1982) summarizes the contributions from different authors:

Ackers-White (1973): $k_s = 1.25d_{35}$

Engelund-Hansen (1967): $k_s = 2d_{65}$

van Rijn (1982): $k_s = 3d_{90}$

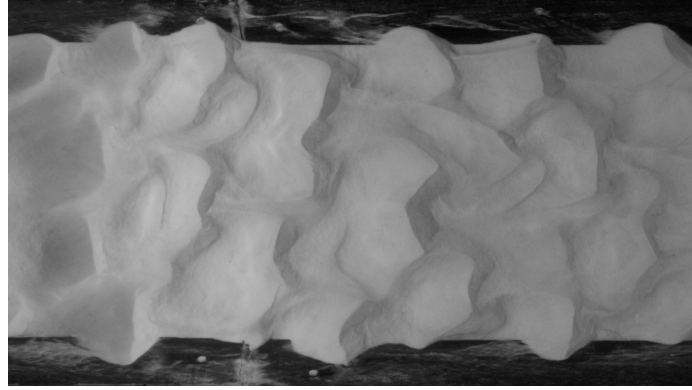


Figure 3.7: *Dunes in experimental flume*

The presence of bedforms molded from the bed material such as presented on Fig. 3.7 requires particular care in river hydraulics, because they not only affect the flow resistance, but they also evolve together with the flow regime and provide characteristic variation of bedform roughness which has important effects on the stage-discharge relationship of a river. Thus, the hydraulic resistance in the presence of bedforms is a dynamic parameter that depends strongly on flow conditions as well as on the bed sediment properties.

The ripples, dunes, and antidunes illustrated in Fig. 3.8 are the classic bedforms of erodible bed, open channel flow. They are undular features with wavelength λ and wave height Δ that scale with the flow depth. The stationary flat bed, ripples, and dunes are usually called the lower flow regime, while the moving plane bed, anti-dunes, and chutes/pools are called the upper flow regime. The flow Froude number is often used to qualify regime conditions and is given by:

$$Fr = \frac{U}{\sqrt{gh}} \quad (3.77)$$

Division of grain and form resistances

Consider the flow in a channel bed in the presence of bedforms, the bed shear stress which satisfies $\tau_b = \gamma RS$, may be divided into components due to skin friction τ_{bs} and due to

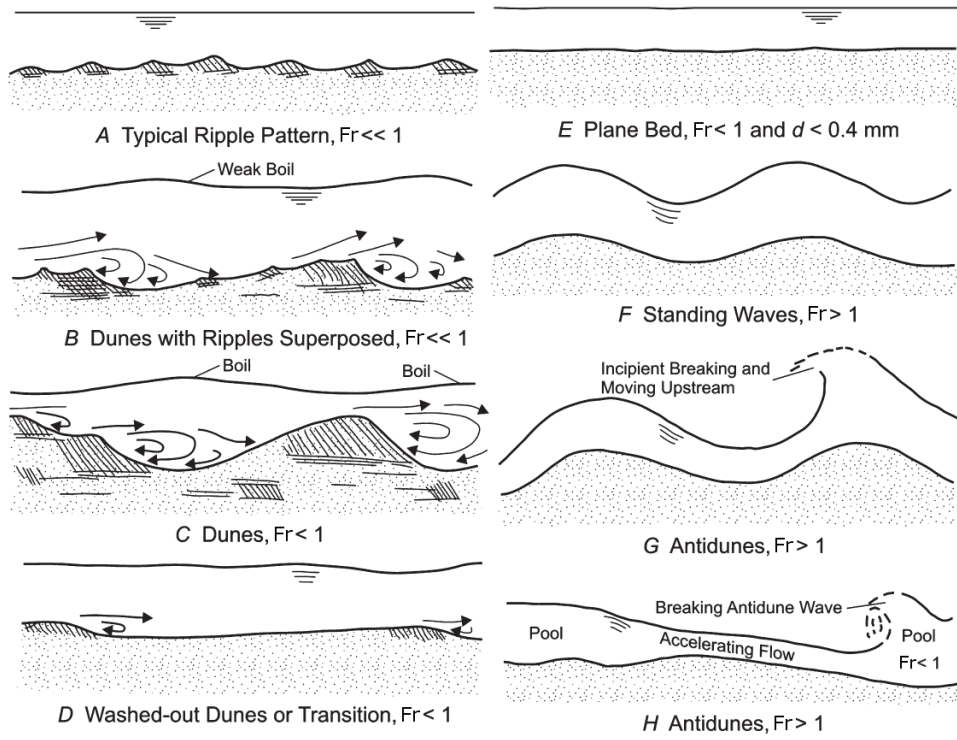


Figure 3.8: Idealized bedforms in alluvial channels (after Garcia, 2008)

form drag τ_{bf} , as long as the separate components do not exert a mutual influence on one another:

$$\tau_b = \tau_{bs} + \tau_{bf} \quad (3.78)$$

Einstein (1942) suggested the division of the hydraulic radius R into two parts R_s and R_f , corresponding to the grain and form roughnesses, and:

$$\tau_b = \gamma S (R_s + R_f) \quad (3.79)$$

The assumption of equal velocity U , together with dividing Eq. (3.78) by ρU^2 , where U is the average flow velocity, provides:

$$C_f = C_{fs} + C_{ff} \quad (3.80)$$

with resistance coefficients associated with skin friction and form drag. Resistance coefficients can also be expressed in terms of Manning coefficients with $U = R^{2/3} S^{1/2} / n$, $U = R_s^{2/3} S^{1/2} / n_s$ and $U = R_f^{2/3} S^{1/2} / n_f$, with n the Manning roughness coefficient of the channel bed, and n_s and n_f are the Manning coefficients corresponding to the grain and form roughnesses, respectively. Further derivations lead to $R_s = R (n_s / n)^{3/2}$ and

$R_f = R(n_f/n)^{3/2}$, which, knowing that $R = R_s + R_f$ from (3.79) gives:

$$n^{3/2} = (n_s)^{3/2} + (n_f)^{3/2} \quad (3.81)$$

An important thing to realize is that form drag has no influence on sediment transport. It was already stated that form drag results from the net pressure distribution over an entire bed form. This pressure acts normal to the body which makes this form drag directly effective neither in moving sediment as bed load nor in entraining sediment into suspension. The part of the effective (total) shear stress that governs sediment transport is thus seen to be the skin friction.

Einstein & Barbarossa (1952), Engelund & Hansen (1967), and Alam & Kennedy (1969) proposed empirical methods for separately calculating the grain and form resistances to flow. Engelund-Hansen's method will be detailed in the next section. van Rijn (1984c) and Karim (1995) established empirical relations to predict the height of bed forms and then the roughness coefficient on a movable bed. The van Rijn formula will be introduced later as example.

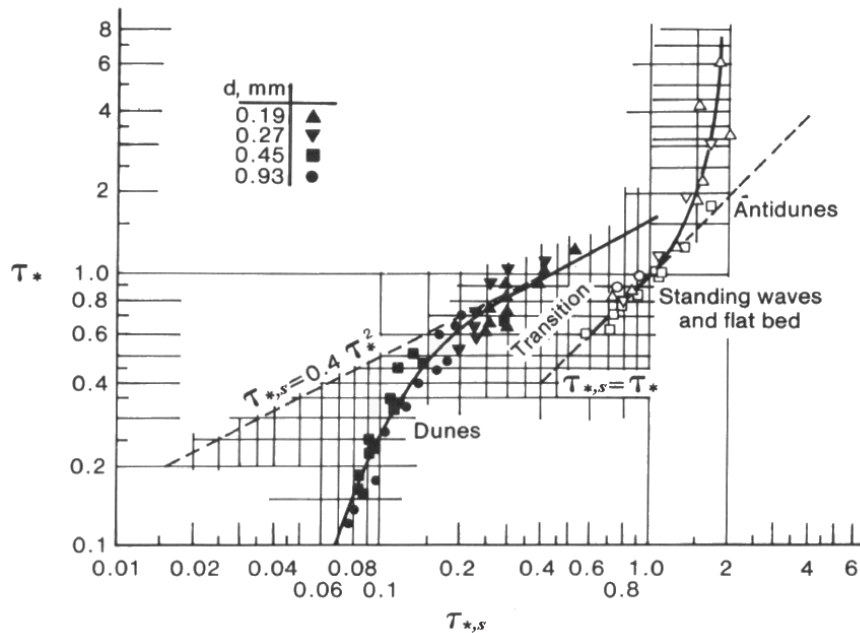


Figure 3.9: Engelund's universal relationship between dimensionless grain shear stress $\tau_{*,s}$ and total shear stress τ_* (following Chang, 1988).

Engelund-Hansen method for stage discharge relations on a movable bed

Empirical techniques, that work on sand-bed streams with dune resistance, have been developed to calculate the total effective boundary shear stress τ_b (as well as the contributions from skin friction and form drag), and thus depth h , for a flow of, say, given slope S and discharge per unit width q . Engelund & Hansen (1967) proposed to use the shear stress partition presented above. Roughness height is estimated as $k_s = 2.5d_{50}$.

The method is based on the empirical relation:

$$\tau_{*,s} = f(\tau_*) \quad (3.82)$$

which gives the form drag through (3.78) since τ_* and $\tau_{*,s}$ are the dimensionless total shear stress and shear stress due to grain roughness, respectively:

$$\tau_* = \frac{\tau_b}{(\gamma_s - \gamma)d}, \quad \tau_{*,s} = \frac{\tau_{bs}}{(\gamma_s - \gamma)d} \quad (3.83)$$

Relation (3.82) is shown graphically on Fig. 3.9. Lower-regime and upper-regime flows are separated by a transition zone which separate the two branches, suggesting the possibility of double-valued predictions. The different branches are given by:

$$\text{lower regime} \quad \tau_{*,s} = 0.06 + 0.4(\tau_*)^2 \quad (3.84a)$$

$$\text{transition} \quad \tau_{*,s} = 0.55 \quad (3.84b)$$

$$\text{upper regime} \quad \tau_{*,s} = \begin{cases} \tau_* & \text{if } \tau_{*,s} < 1 \\ [0.298 + 0.702 - (\tau_*)^{-1.8}]^{-(1/1.8)} & \text{if } \tau_{*,s} > 1 \end{cases} \quad (3.84c)$$

Practically, the procedure suggests to select values of R_s (the hydraulic radius due to skin roughness only), ranging from a low value to near bank-full. The cross-sectional mean velocity U is then determined using Yalin's (1992) formulae for vertical distribution of longitudinal velocity:

$$\frac{u}{u_*} = \frac{1}{\kappa} \ln \left(\frac{z}{k_s} \right) + B_s \quad (3.85)$$

with B_s a function of $Re_* = u_* k_s / \nu$ which is estimated through:

$$B_s = \underbrace{8.5}_{\frac{1}{\kappa} \ln A} + \underbrace{[2.5 \ln(Re_*) - 3] e^{-0.121[\ln(Re_*)]^{2.42}}}_{\frac{1}{\kappa} \ln \chi} \quad (3.86)$$

This form represents all possible hydraulic regimes and U is computed through vertical integration of (3.85) and considering that this depth-averaged velocity applies to the whole cross section:

$$\frac{U}{u_*} = \frac{1}{\kappa} \ln \left(Ae^{-1} \chi \frac{R_s}{k_s} \right) \quad (3.87)$$

The calculation then proceeds as follows:

$$\begin{aligned} R_s &\rightarrow \tau_{bs} \rightarrow \tau_{*,s} && \text{through } \tau_{bs} = \gamma S R_s \text{ and Eq. (3.83)} \\ \tau_{*,s} &\rightarrow \tau_* && \text{using Eqs. (3.84a) - (3.84b)} \\ \tau_* &\rightarrow \tau_b \rightarrow R && \text{using Eq. (3.83) and } \tau_b = \gamma S R \end{aligned}$$

which finally gives the discharge Q , given the cross-sectional dimensions. The value of $\tau_{*,s}$ can be used to calculate bed load transport using an appropriate empirical formula.

The Engelund-Hansen method to predict stage-discharge relationship over a mobile bed with bedforms will be used in section 4.3.2, when imposing uniform flow in the laboratory flume where morphological experiments took place.

Equivalent Roughness of bed forms

The equivalent roughness of alluvial beds in the presence of ripples and dunes was addressed with the Nikuradse hydraulic roughness k_s approach by e.g. van Rijn (1984c). In van Rijn's method, the grain roughness is $3d_{90}$ and the total (grain plus form resistance) effective roughness is given by:

$$k_s = 3d_{90} + 1.1\Delta \left(1 - e^{-25\Delta/\lambda} \right) \quad (3.88)$$

where d_{90} is the grain size for which 90% of the bed material is finer ; Δ and λ are the bedform height and length, respectively. This roughness subdivision should be considered as a model with its underlying limitations and hypotheses. In this case, the hypotheses are somewhat in conflict with the model defined in Eq. (3.78).

The same author (van Rijn, 1982) also computed the equivalent roughness of a bed from the resistance equation for hydraulic rough flow, which finally gives a link between the flow conditions and the bedform dimensions:

$$k_s = \frac{R}{\exp \left(\frac{\sqrt{U/u_*} - 6.23}{2.5} \right)} \quad (3.89)$$

where U is the mean flow velocity (m/s), R the hydraulic radius and u_* the shear velocity.

Chapter 4

Laboratory setup

Two different methodologies for studying solute and sand transport are followed in this thesis: laboratory experiments and numerical modelling. This chapter deals with the experimental approach. A number of experiments on flow modelling in compound channels have already been performed in the UCL laboratory (Bousmar, 2002). New experiments were set up in which the dispersive capacity of a compound channel flow, as well as sediment transport and floodplain deposition were observed, using novel measurement techniques and bringing extensive modifications to the pre-existent experimental setup. The objective of the experiments is twofold. The experimental results will be used, on the one hand, for understanding the mechanisms that govern mass transfer between the subsections and, on the other hand, for validation of numerical modelling.

This chapter describes the experimental setup, different data analysis with some selected results which mainly concern establishment of the regime flow, which is the condition chosen for all performed experiments. The extent of the modifications brought to the original flume will be briefly explained, as much work was requested to introduce morphodynamic experiments in the UCL compound channel flume. The tracer injection study will be presented in Chapter 5, along with the sediment transport and deposition analysis experimental campaign.

4.1 The UCL compound channel flume

Experiments were conducted in a compound channel flow facility with a total length of 14 m and a width of 1.2 m (Figure 4.1). This length comprises a 10 m long measurement section, with adjustable coated plywood bottom ; a 2 m long inlet tank ; and a 2 m outlet zone. This flow facility in the UCL laboratory has already been used for experiments



Figure 4.1: *UCL compound channel flume. (Left) Compound channel with mobile bed experiment featuring bedforms, floodplain deposition and sand traps. (Upper right) General view with upstream discharge separation. (Down right) Upstream subsection separation and elliptical profiles*

as described by Bousmar (2002), mainly to analyse geometrical and turbulent transfers between the subsections. The bed slope is variable in the interval $S_0 = 0 \dots 0.03$, even if the facility was mainly designed for experiments with a turbulent and sub-critical flow, as is the case in lowland rivers and the present experiments. The Reynolds numbers $Re \equiv Uh/\nu$ are sufficiently large ($Re > 4000$) and the Froude numbers $Fr \equiv U/\sqrt{gh}$ are smaller than 1 to meet these requirements.

The inlet section of the facility is divided into three parts, each with a separate water supply and discharge measurements is operated by electromagnetic flowmeters. This upstream discharge differentiation ensures equilibrated flow distribution from the inlet

section (Bousmar *et al.*, 2005). Injected water is stillled through perforated PVC tubes, wrapped around the supply-pipes. A vertical contraction connects the inlet section to the horizontal bottom of the flume. Those elliptical profiles are artificially roughened, to obtain fully developed turbulent flow as upstream as possible. Foam boards are mounted, floating just downstream of the injection to suppress free surface waves. The downstream level is controlled through an adjustable weir.

The discharge can be obtained through two distinct sources. The first system supplies water through an upstream reservoir with constant level (see Fig. C.1), in such a way that an accurate discharge control is possible, independently from the pump working. Flow rate in the interval $Q = 0 \dots 30 \text{ l/s}$ is available. This system was used by Bousmar (2002) to achieve his experiments. Following the need to perform morphodynamic experiments, a second supply system was designed. This time, a duo of vortex pumps provide the requested discharge. Those pumps are destined to feed the different subsections (see Fig. 4.4) and may produce $Q_{mc} = 0 \dots 25 \text{ l/s}$ in the main channel and up to $Q_{fp} = 18 \text{ l/s}$ over the floodplains. A sufficient number of valves is installed on the pipe network so that all of the discharge may be redirected in a single subsection.

Accurate and repeatable measurements are eased through a three-directional measurement trolley fixed above the flume. An automatic positioning device allows systematic measurements all over each cross section y - z . This device is presented on Fig. C.2 and may receive a wide set of different measurement probes.

4.2 Evolution towards sediment transport study

The sediment transport and floodplain deposition experimental campaign involved major modifications to the experimental flow facility installed in the UCL laboratory.

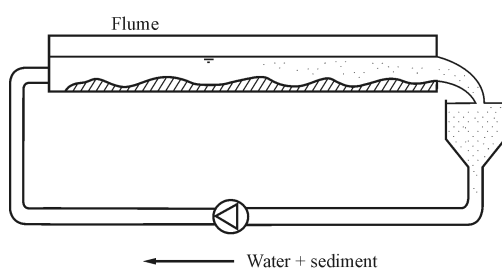


Figure 4.2: *Recirculation system*

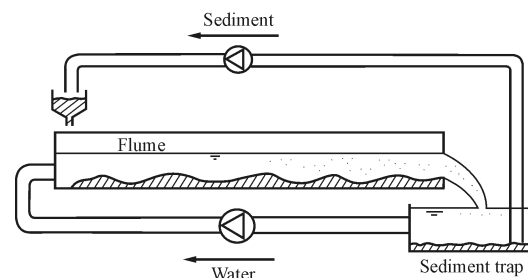


Figure 4.3: *Sand-feed system*

Usually, two types of flumes are being used for experiments with movable beds: (1) a sand-recirculation system and (2) a sand-feed system (van Rijn & Klaassen, 1981). Small

differences in the final equilibrium achieved are expected between both system in case of uniform sediment (Guy *et al.*, 1966) but each mode has particular operational advantages, with the specified flow and sediment constraints which differ (Parker & Wilcock, 1993). Table 4.1 gives a comparison between each operational mode.

Table 4.1: *Types of sediment flumes (van Rijn and Klaassen, 1981)*

<u>Sand-recirculation system</u>	<u>Sand-feed system</u>
The sand particles are returned continuously (Fig.4.2)	The sand particles are separated from the flow and returned by means of some mechanical method (Fig.4.3)
<u>advantages</u>	<u>advantages</u>
★ the system tends to act like an infinite flow (natural channels)	★ possibility to impose a specific rate of sediment transport
★ applicable for fine sediments	★ no limitations on flume dimensions
★ relatively easy to operate	★ accurate determination of sediment transport rate
★ equilibrium conditions are established rapidly, if pre-set slope is correct	
<u>imposed variables</u>	<u>imposed variables</u>
★ discharge	★ discharge
★ slope (tilting flume)	★ sediment transport
<u>adjusted variables</u>	<u>adjusted variables</u>
★ flow depth	★ flow depth
★ sediment transport	★ slope

The chosen final design presented on Fig. 4.4 appeared multipurpose, as both recirculating and sediment-feed experiments were envisaged. The first series programmed a mobile main channel bed and recirculating sediment-laden water from this subsection. Sediment feeding is then deactivated and a sediment bed is installed in the main channel, the pump 1 discharges the sediment-laden main channel flow and the pump 2 injects fresh water on the floodplains. The second series featured compound channel flow over a rigid bed initially sediment-free and upstream sediment feed (Fig. C.3): the valve 1 is closed and pump 1 also injects fresh water in the main channel (V2 opened).

The design phase included the choice of the pipe diameters, each pump's effective power,

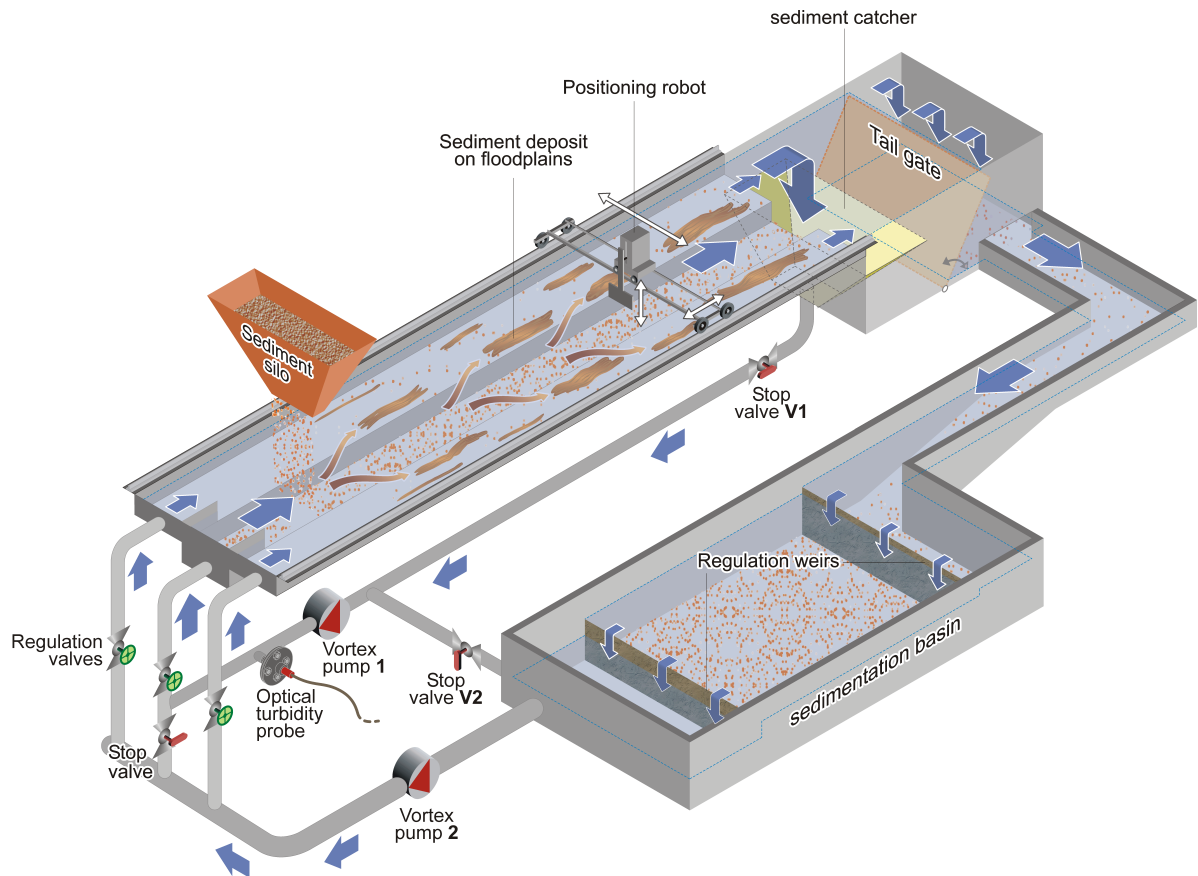


Figure 4.4: *UCL compound channel flume for sediment experiments. Experimental setup for (a) mobile bed with recirculation experiments: the sediment feeding is not active and the sediment bed is installed in the main channel. The pump 1 discharges the sediment-laden main channel flow and the pump 2 injects fresh water on the floodplains; (b) upstream sediment-feed over rigid bed experiments: the valve 1 is closed and pump 1 also injects fresh water in the main channel (V2 opened).*

the sedimentation basin's dimensions (6.3 m x 2.4 m) but also asked to settle on the sediment diameter to be used in morphodynamic experiments. All detailed design calculations are found in the work by Dupont & Tiquet (2008).

The chosen sand presents a $d_{50} = 91\mu\text{m}$ and is well graded. This sediment shows good mobility in the main channel and low mobility over the floodplains, according to van Rijn (1984b), to favour transfer and deposition on the floodplains. The mobility parameter $\theta_s = u_*^2 / (s - 1) g d_{50}$ was calculated in each subsection and compared using Fig. 3.6. Following the principle of maximizing sediment transfer from the main channel towards the floodplain and optimizing the deposition conditions, Rouse's (1937) relative concentrations profiles (Eq. (3.50) and Fig. 3.4) were plotted for different floodplain water depths in the compound geometry of Fig. 5.10b. Fig. 4.5 presents such curves for the chosen sand and with 7.5 mm or 20 mm water depth over the floodplains. $91\mu\text{m}$ sand offers relatively high concentrations in the upper region of the main channel which is likely to transfer towards the floodplains. At the same time, it provides over the floodplains a great proportion of relative concentrations near the bottom, which maximizes the deposition.

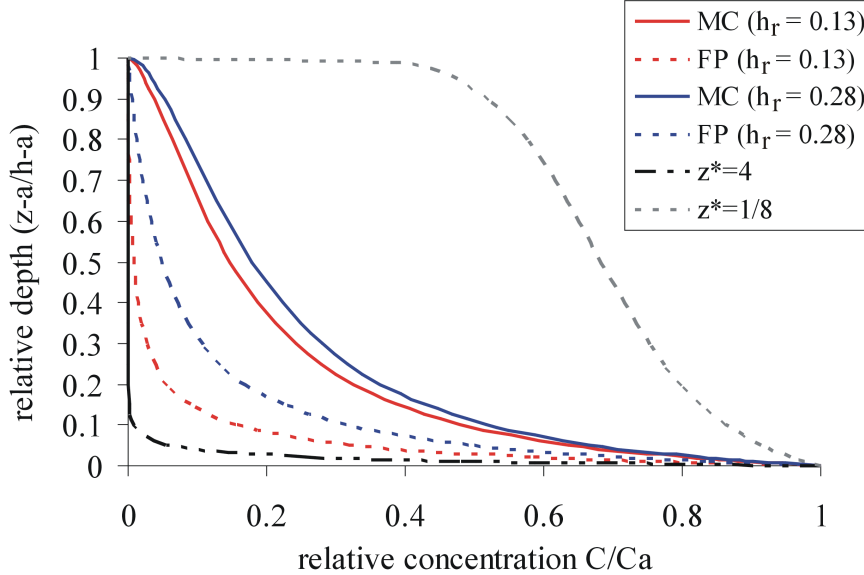


Figure 4.5: Vertical distribution of relative sand ($d_{50} = 91\mu\text{m}$) concentrations in main channel (MC) and over the floodplains (FP) following Rouse (1937) for different floodplain water depths in compound geometry where morphological experiments were performed (Fig. 5.10b with $S_0 = 1.9/1000$) . z_* is given by Eq. (3.51)

4.3 Measuring devices and techniques

4.3.1 Stage-discharge curves and velocity profiles

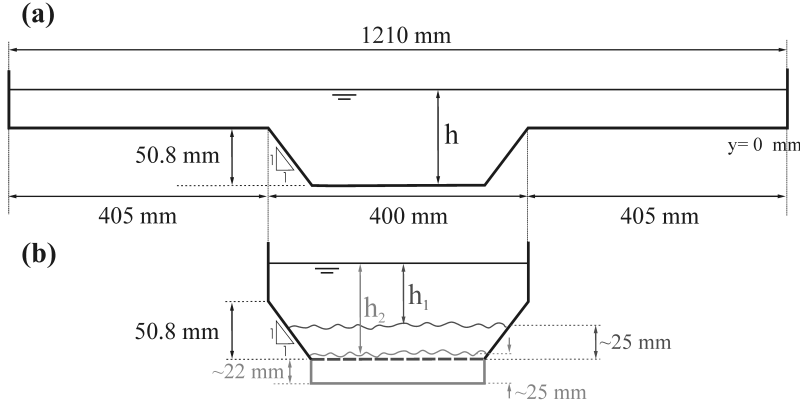


Figure 4.6: *Flume cross sections for (a) symmetric compound channel with $S_0 = 1.9/1000$ (b) Isolated Main Channel with $S_0 = 1/1000$ and h_1 or $S_0 = 1.9/1000$ and h_2*

For all experiments, uniform-flow conditions are expected. The stage-discharge curve is obtained from water-depth measurements along the flume, using an ultrasonic probe (*Keyence ud-020*) mounted on the automatic positioning device for water surface detection. The uniform flow for a given discharge is achieved by adjusting the downstream water level, until the water profile is parallel to the channel bed. Such a stage-discharge curve for a symmetric compound-channel cross-section (Fig. 4.6a or later on Fig. 5.10b) with $S_0 = 1.9/1000$ is plotted on Fig. 4.7. For comparison, the stage-discharge curve is also computed using the Exchange Discharge Method by Bousmar & Zech (1999), the Single Channel Method or the Divided Channel Method, all developed in Chapter 2. A Manning roughness coefficient equal to $n_{mc,fp} = 0.0091 \text{ s/m}^{1/3}$ is selected for both main channel and floodplain in EDM computations, which apparently describes relatively accurately the friction on the plywood bottom. A Manning coefficient of $n_w = 0.011 \text{ s/m}^{1/3}$ is requested to describe the rougher 45° lateral slope which is made of different wood material. For confirmation, inbank flow data on Fig. 4.7 is compared to general computations using Eq. (3.22) and a global manning coefficient which is a composite roughness coefficient as defined by Einstein (1942):

$$n = \left(\frac{P_{mc} n_{mc}^{3/2} + P_w n_w^{3/2}}{P_{mc} + P_w} \right)^{2/3} \quad (4.1)$$

with P_{mc} the wetted perimeter of the bottom of the main channel and P_w the wetted perimeter of both the 45° lateral walls.

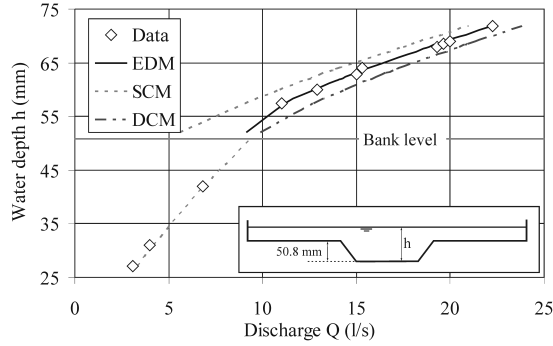


Figure 4.7: Stage-discharge curve, symmetric compound channel with $S_0 = 1.9/1000$: measured and computed values.

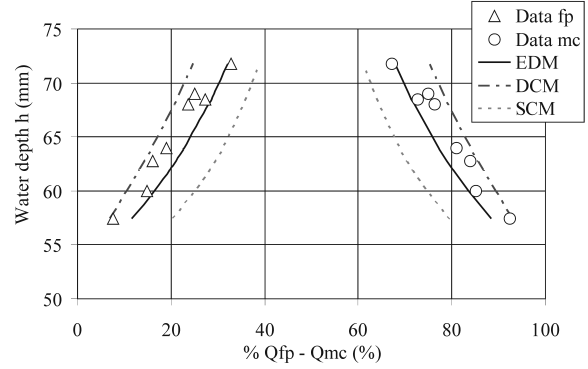


Figure 4.8: Percentage of total flow in the main channel and on the floodplain : measured and computed values.

Parallel to uniform flow conditions, equilibrated discharge from inlet section are imposed. Systematic velocity measurements are performed using a Pitot tube (diameter = 4 mm). The depth-averaged velocities U are estimated from the local measurements and, if integrated in the different subsections and taken sufficiently far from the inlet (e.g. 7 m), provide a measure of the equilibrium discharge in each subsection. This equilibrium flow distribution is imposed directly upstream with the subsection's separate water supply. Recalling the experimental conditions which produced the stage-discharge curve from Fig. 4.7, the corresponding flow division data is shown on Fig. 4.8.

4.3.2 Sediment bottom measurements

Compound channel experiments with a mobile bed in the main channel (Chapter 5) show some bedforms development conditions, which influences the flow resistance, as expressed in section 3.5.4. Stage-discharge curves are thus trickier to predict, as resistance of an alluvial channel is expected to evolve with the water discharge. Experiments in the laboratory flume intended to illustrate this key aspect of alluvial channel behavior, isolating the main channel from the floodplains using coated plywood sidewalls. Different devices were used to measure the sediment level under flow conditions and under dry (post-experimental) conditions. On the one hand, the dry sediment bed was very precisely detected using a CDD laser displacement sensor (*Keyence LK-G 500*). On the other hand, a Bed profiler (*Delft Hydraulics PV-07*) was used to track the bed level and measure bedform amplitudes

during experiments but mainly helped to check uniform flow conditions when combined to the water surface detection probe (Fig. C.2).

Isolated main channel experiments (IMC)

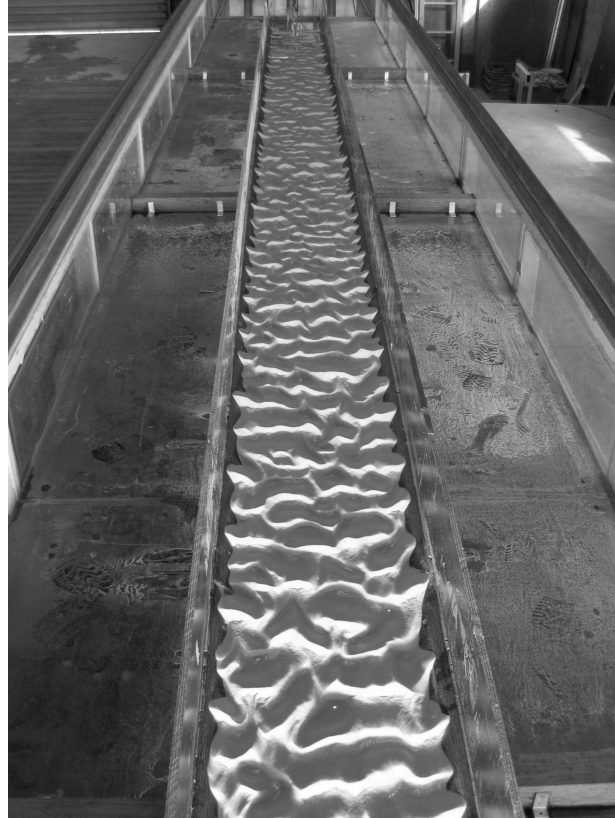


Figure 4.9: *Bedforms in isolated main channel configuration*

Fig. 4.9 illustrates how the main channel was isolated with vertical coated plywood sidewalls to provide an extended trapezoidal cross-section with mobile bed and how bedforms developed. Stage-discharge measurements were performed in such geometry with $S_0 = 1/1000$ and a 25 mm sand layer covering the bottom of the main channel section depicted in Fig. 4.6b. The experimental data are plotted in Fig. 4.10 together with Engelund-Hansen's prediction for stage discharge relation on the movable bed (h_1), following the method explained in section 3.5.4. The hydraulic radius for the sediment bed R_b is also presented, as well as its decomposition due to bed forms R_f and skin friction R_s . The discharge in the isolated main channel was such that the flow was kept in the lower regime described by Eq. (3.84a). The shear stress hardly attained the transition

regime of Fig. 3.9, and optimal conditions were found for the development of dunes.

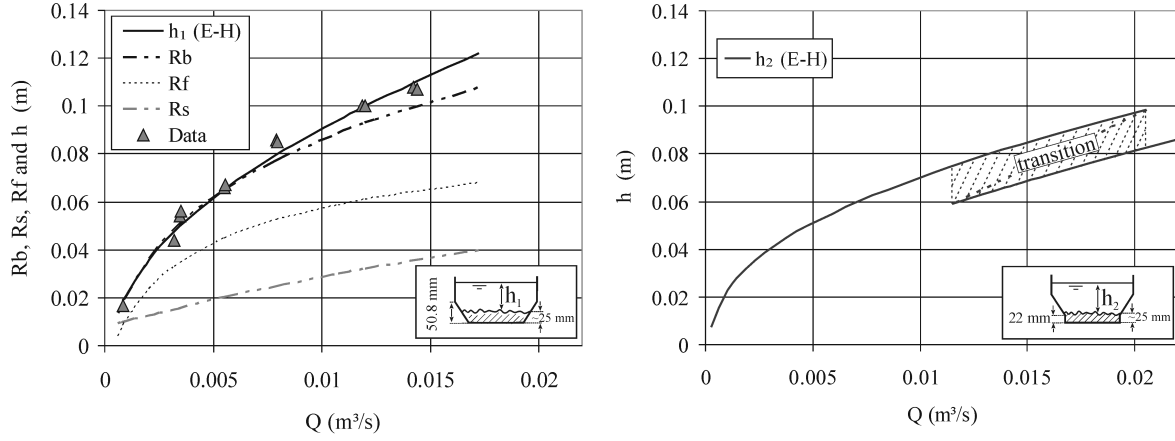


Figure 4.10: (Left) *E-H predictions of Stage discharge curve for Isolated Main Channel section with $S_0 = 1/1000$ (h_1 from Fig. 4.6b).* (Right) *Previsions for $S_0 = 1.9/1000$ (h_2 on the right)*

The Manning roughness coefficient n_b for the mobile bed can be estimated from those measurements, using a wall correction, as described in Bousmar *et al.* (2006). Fig. 4.11 (left) shows the roughness coefficient as a function of the discharge. The experimental values are compared to Engelund-Hansen predictions and the roughness coefficient of a bed without bedforms. In contrast with the experiments by Atabay (2001) and Ayyoubzadeh (1997), which used a larger sediment size ($d_{50} = 0.88$ mm), bedforms appear very fast with increasing discharge, while their drag seems to reduce slowly when reaching higher flow regimes.

The importance of form drag on the flow is made clearer expressing the roughness coefficient as the equivalent roughness of Nikuradse k_s . The equivalent roughness can be decomposed as

$$k_s = k_{s,grain} + k_{s,form} \quad (4.2)$$

and $k_{s,grain} = 3d_{90}$ = sand roughness; and $k_{s,form}$ = form roughness, due to dune development (van Rijn, 1982). Fig. 4.11 (right) shows k_s compared to the $k_{s,grain}$ for a $d_{90} = 91\mu\text{m}$ where obviously form drag is mostly responsible for flow resistance.

The empirical predictions of Engelund-Hansen for h_1 on Fig. 4.10 give certain credit to the theory compared to experimental data. However, in the following Chapter, a slope

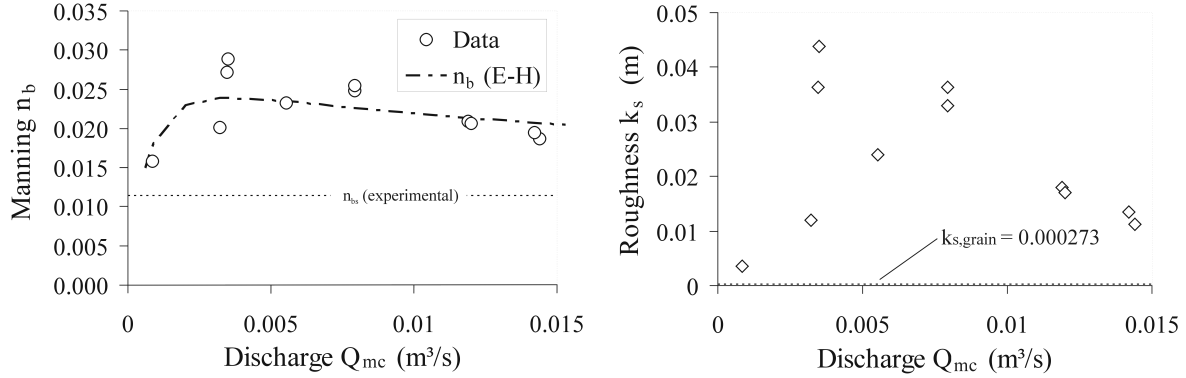


Figure 4.11: Roughness coefficients for IMC series and $S_0 = 1/1000$ (h_1 from Fig. 4.6b). (Left) Manning coefficients predicted by Engelund and Hansen compared to experimental data and measured manning due to grain roughness only $n_{bs} = 0.0114$ ($s/m^{1/3}$). (Right) Nikuradse k_s calculated by van Rijn's formula (3.89).

of $S_0 = 1.9/1000$ will be used in a geometry with the water depth defined as h_2 in Fig. 4.6b. This justifies the interest towards Engelund-Hansen's previsions for h_2 . The highest flow regimes will enter a so-called transition zone for bedforms and higher scattering in roughness coefficients are expected.

Bedforms characterization

The main river channel was monitored and thorough data stands available for future research on bedform geometry, resistance, and dune migration rate. More details on bedform developments and analysis for a d_{50} of 0.88 mm sand are given by Tang and Knight (2006). They investigated the effect on bed forms of the floodplain roughness during overbank flow using statistical methods.

The bed slope with bedforms was observed for uniform flow checking using a Bed profiler which detects electrical conduction gradients which characterizes the sediment-water interface. A laser displacement sensor provides very accurate post-experimental bedforms measurements. The dune's wavelength λ (0.05-0.15 m) and height Δ (5-30 mm) showed important variability throughout this study and a longitudinal profile is presented on Fig. 4.12 for different overbank flow experiments presented in the following Chapter.

Laser measurements provides very precise information on the bed topography, and were also used to record the importance of deposition on the floodplains. Since bedform

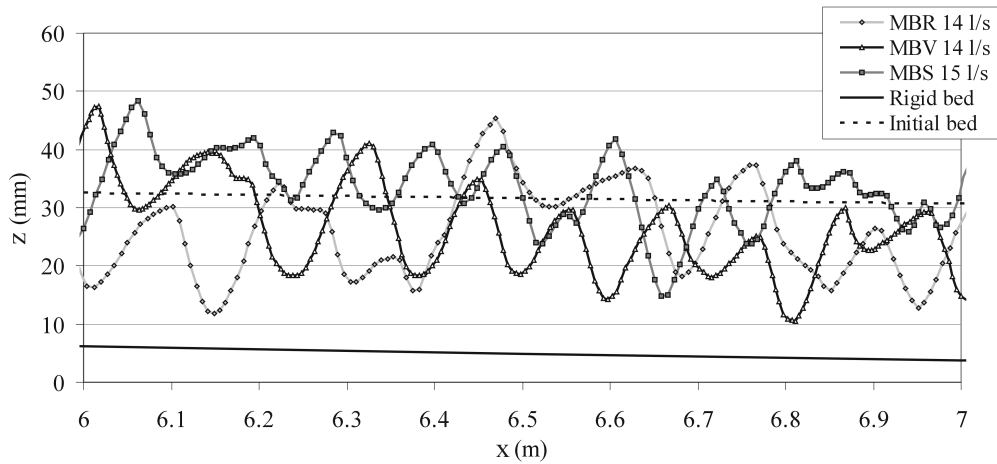


Figure 4.12: *Bedforms for overbank flow and mobile main channel bed (see Table 5.4 for legend explanation). The measurements are taken in the main channel ($y = 0.605$ m)*

regularity shows also important influence on the flow resistance, bed visualization using digital imaging, explained in the next section, appears as a powerful tool for bedform characterization. Figure 4.13 illustrates the reliability of bedform visualization compared to laser measurements.

4.3.3 Digital imaging

Flow visualization using digital imaging filled different needs. The first objective was to produce instantaneous general views of the mobile bed, along with the deposition formation on the floodplains throughout the experiments described in the next chapter. Further, the presence of large scale turbulent structures at the main channel and floodplain interface requested to record images of seeded flow, as performed by Sellin (1964) on Fig. 1.2 or Bousmar (2002). In all cases, the chosen setup looked like the one presented on Figure 4.14, with a lateral lighting, to avoid direct reflections on the water surface.

As bedforms and sediment deposits show a relatively small length scale, a high resolution (4672×3104 pixels) reflex camera Pentax K20D was used to observe the flume when investigating sediment transfers and deposition on the floodplains. Successive views along the flow, taken as sketched on Fig. 4.15a, re-constructed after post-treatment the overall instantaneous situations. Each individual frame covered the whole width and 1.77 m longitudinal distance along the channel. This method gives a qualitative time evolution

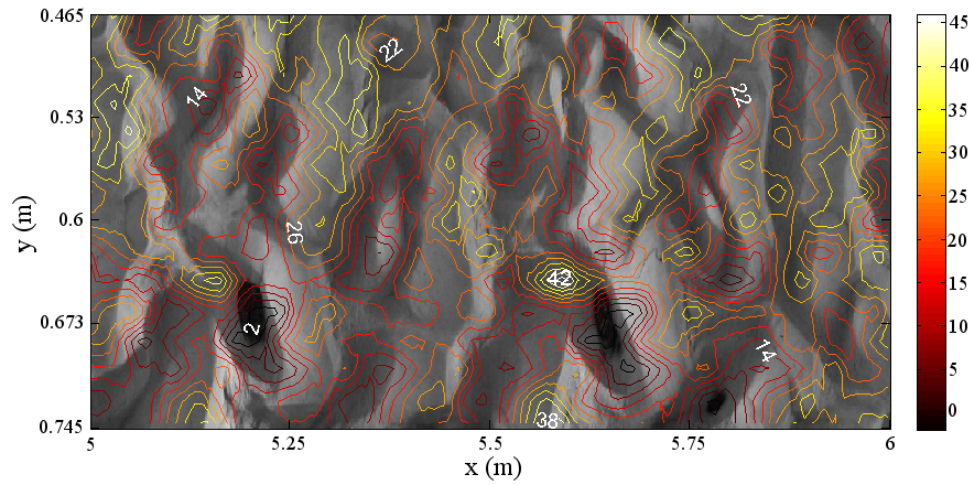


Figure 4.13: *Tentative superposition of dune heights contours z (mm) over a digital view of main channel bedforms. The bottom topography is recorded with a Keyence laser displacement sensor fixed on the flume's measurement trolley.*

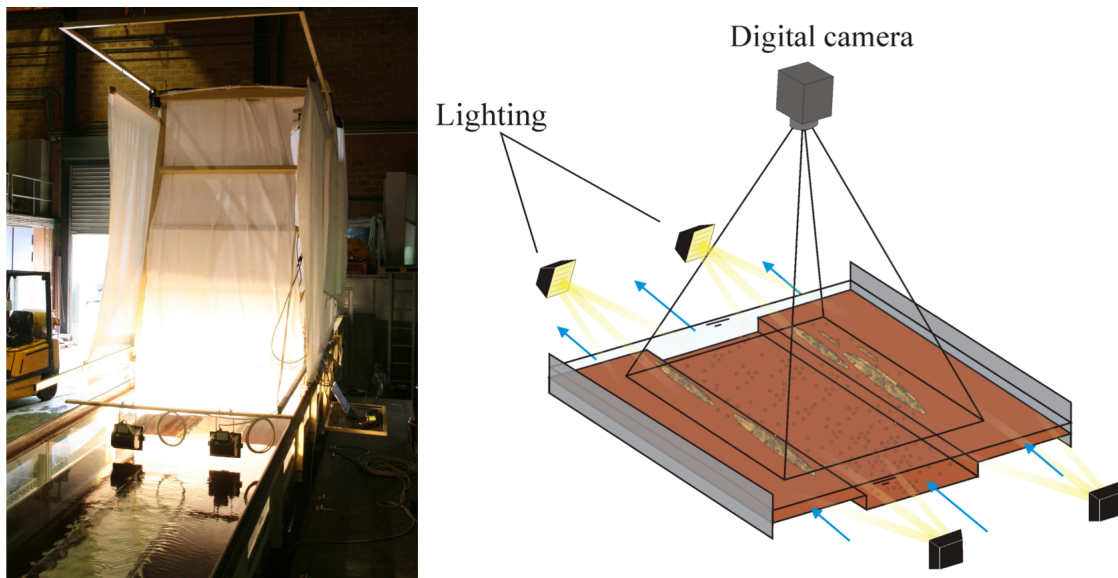


Figure 4.14: *Experimental setup for digital imaging. The steel frame on the left permits an aerial view of the zone of interest, isolated from parasitic light source.*

of sand deposits on the floodplains and was used throughout the study to discuss general deposition patterns.

Likewise, the flow field was determined by distributing homogeneously floating tracers (expanded-polystyrene or EPS pearls with a diameter 4-6 mm) upstream and recording their positions with a digital camera. The camera (MARLIN AVT F-131 firewire) has a resolution of 1280×960 pixels with 256 gray levels and a maximum frame rate of 15 Hz. Time series of images are stored directly on the hard disk of a PC. Measurements were performed for different areas, depending on the experiments, covering a 2.68 m longitudinal distance (Fig. 4.15b). Such measurements are presented in Chapter 6 when dealing with large scale interfacial turbulent structures.

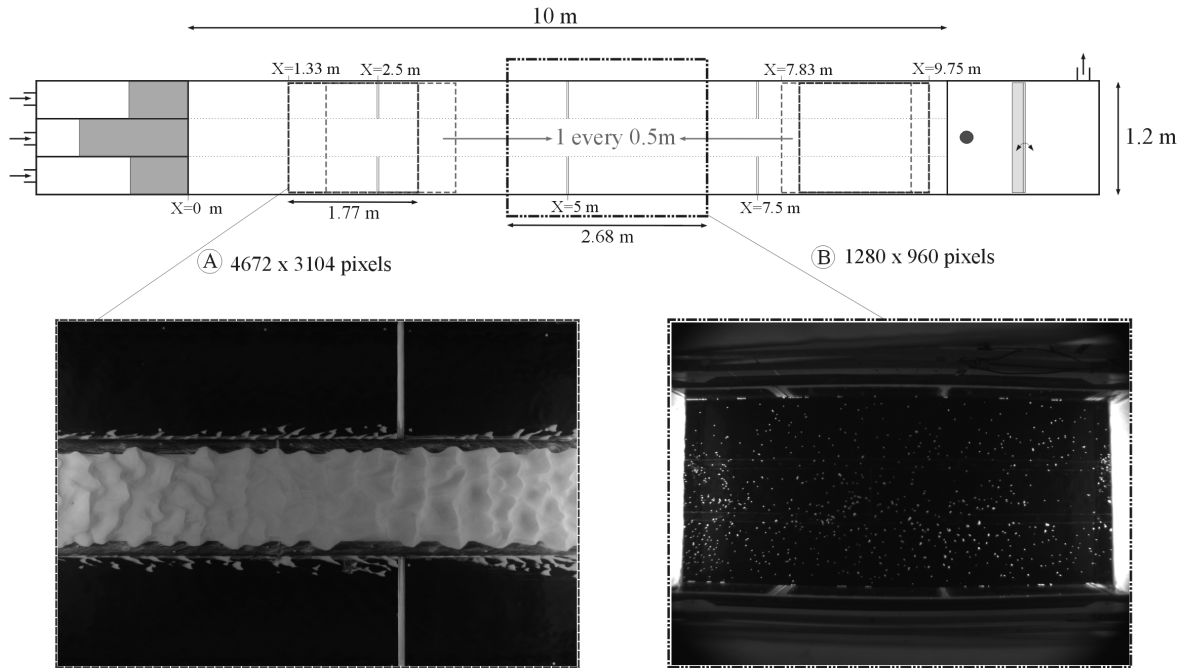


Figure 4.15: Top view of the compound channel flow facility. The High resolution recording areas are indicated with the dashed squares (A) while the time series of lower resolution images are found with the dashed-dot square (B). Strong distorsion is expected for the latter, and appropriate correction is brought during analysis.

Chapter 5

Laboratory experiments

The understanding and prediction of floodplain deposit pattern rely on mixing processes in compound channels, including convection, diffusion and dispersion. Indeed, fine sediment of which we intend to study the deposition over the floodplains are mainly transported by suspension. Predicting the way sediments will settle on the floodplains requires analysis of the diffusion/dispersion and convection phenomena occurring during overbank flow. Diffusion processes are thus first investigated in compound channel involving dye/solute concentration measurements instead of sediments.

This chapter describes the different experimental campaigns which took place in the UCL compound channel flume. The tracer injection study as described in Section 5.1 is published in Fraselle *et al.* (2007) and Fraselle *et al.* (2008). In Section 5.2, the sediment transport and deposition analysis experimental campaign is found as reported in Fraselle *et al.* (2010b).

5.1 Solute tracing

This section reports results from an experimental study of diffusion phenomena through solution injection and tracing of the plume in a prismatic compound channel flume (Université catholique de Louvain, Belgium). The concentration of the saline solution (NaCl) used is measured by means of a conductivity meter specially designed to avoid wall and free-surface effects. Results from different injection configurations are compared to concentrations obtained through three-dimensional numerical integration of a simplified solute transport equation together with experimental velocities. The model assumes steady source, unidirectional flow and Fickian law for Reynolds fluxes. Anisotropic mixing coefficients will be chosen to fit the numerical results on experimental profiles and they will

be compared to results collated from published field studies (Rutherford, 1994).

Before presenting the experimental approach together with the data, a comprehensive background on past studies on diffusion/dispersion study is given, to feed the discussion on the results.

5.1.1 Background

Researchers investigated diffusion processes in compound channel involving dye/solute concentration measurements instead of sediments. Arnold *et al.* (1989), Wood & Liang (1989) and Spence *et al.* (1997) have all measured concentrations and diffusivities using different setups: going from digital imaging techniques to Laser Induced Fluorescence (LIF) measurements, passing by conductivity measurements. More recently, Shiono & Feng (2003) used Laser Doppler Anemometer (LDA) and LIF to measure simultaneously the three components of velocity and turbulent fluctuations of dye concentration. They investigated the effect of secondary flow on passive contaminant transport processes. Djordjevic (1993) developed a 2-D mathematical model for transport and used it for interpretation of dynamic measurements of tracer with photoelectronic colorimeters. Lin & Shiono (1995) as well as Simoes & Wang (1997) both checked their entire three-dimensional hydrodynamic model to study solute transport in compound channels with Wood & Liang's data (1989). More recently, Shiono *et al.* (1997, 2003) used turbulence models which enabled three-dimensional simulations with and without explicit secondary flows computation.

Most of the authors consulted who have performed solute tracing experiments in compound channels agree on several points. Wood & Liang (1989) observed how effluent released in the deeper (main) channel near the vertical step leads to the maximum concentrations moving towards the step. They explain that solute accumulates in this zone due to partially reflecting surface. Less initial dilution is also observed for floodplain tracer injection than for main channel injection (e.g. Djordjevic *et al.*, 1989), pointing out a lower general mixing intensity in the former.

Releasing effluent in the shallow channel near the step leads to the maximum concentration moving away from the step due to solute accumulation in slower flowing zones. Besides, concentration distribution are skewed, indicating that the solute experiences a higher rate of mixing as the floodplain approaches the main channel (Figure 5.1 on the right). Guymer *et al.* (1998) further assumes that a localized increased mixing over the

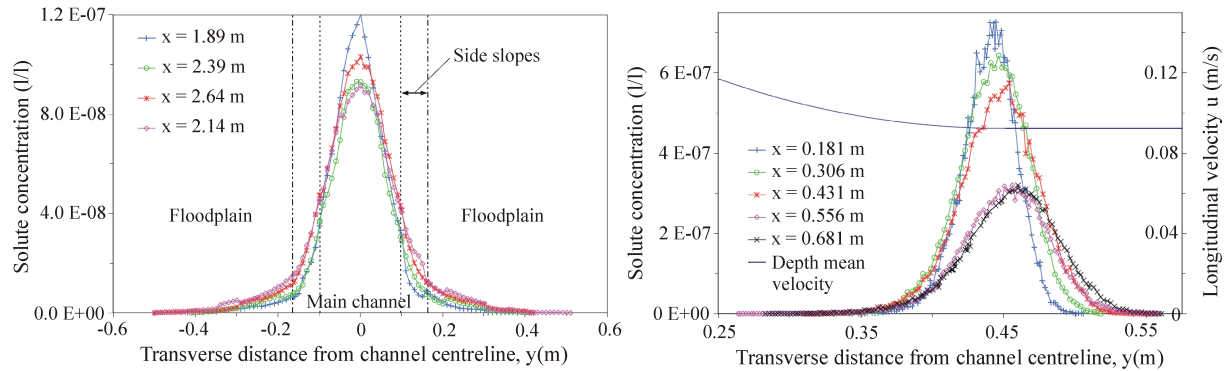


Figure 5.1: (Left) Transverse variation of depth averaged solute concentrations; channel centreline bed injection at $x=0$ m, overbank discharge = 17.2 l/s, $h_{fp} = 32$ mm and $h_r = 0.49$ (Spence & Guymmer, unpublished). (Right) Transverse variation of solute concentrations 5 mm below the water surface on a floodplain; floodplain bed injection at $x = 0$ m, $y = 0.446$ m, overbank discharge = 10.8 l/s, $h_{fp} = 15.7$ mm and $h_r = 0.24$

floodplains is expected. As seen in Chapter 2, stronger turbulent mixing is noticed in this shear layer region, and we see the effects on depth-averaged concentrations on Figure 5.1 (left).

The tracer plume behaviour depends mainly on the relative depth h_r which determines the interaction intensity in the shear layer and thus dominant turbulence phenomena which act on tracer mixing. *There is a strong effect of the transverse shear layer on the dispersion of pollutants* (Prinos, 1992). The injection position has also great impact since helical secondary circulation completely influence tracer mixing (solute transport rate) and heterogeneous mixing in the compound section is expected (its direction can oppose the turbulence diffusion of dye). This is illustrated by Figure 5.2 where concentration values obtained by Shiono & Feng (2003) are sketched.

Authors have also been discussing on theoretical expression of solute mixing. As discussed in Chapter 3, the Fickian law is often used to express mathematically the Reynolds fluxes $\overline{c'v'} = -D_y \frac{\partial C}{\partial y}$ (Taylor, 1954 ; Csanady, 1973). However, dye injection experiments proves that the Fickian law is not valid everywhere, mostly in the interaction region (Shiono & Feng, 2003).

Lateral mixing will receive the greatest attention throughout the study, conditioning lateral transfers from the main channel towards the floodplains. To summarize, river transverse or lateral mixing is due to the following causes (Rutherford, 1994): (1) turbulence

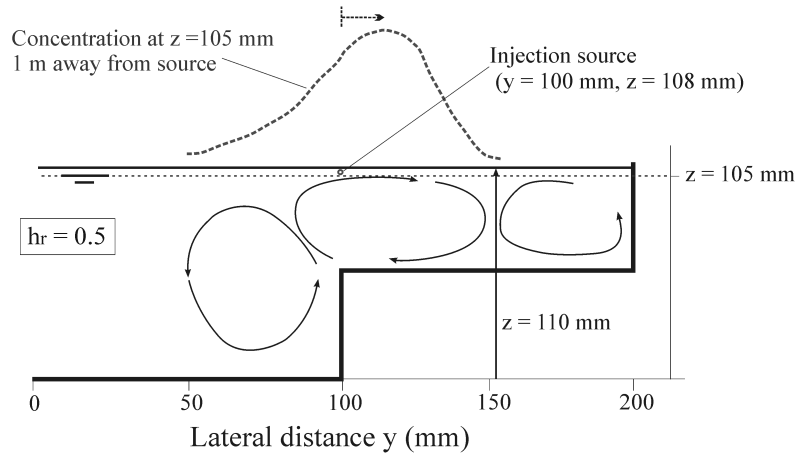


Figure 5.2: *Effect of helical secondary currents on concentration distribution in compound channels, following Shiono & Feng (2003). Concentrations and secondary currents are represented schematically in a $h_r = 0.5$ compound channel.*

generated by the channel, expressed through many eddies, which transfer mass and momentum, resulting in a solute mixing far exceeding that occurring at the molecular scale. In a river, lateral mixing is due to the horizontal eddies which rotate about a vertical axis. (2) vertical variations in the transverse velocity (velocity shear), which are significant in the vicinity of channel banks and further contribute to transverse spreading of contaminants ; (3) secondary currents, which causes solute to move in opposite directions at different depths increasing the rate of mixing.

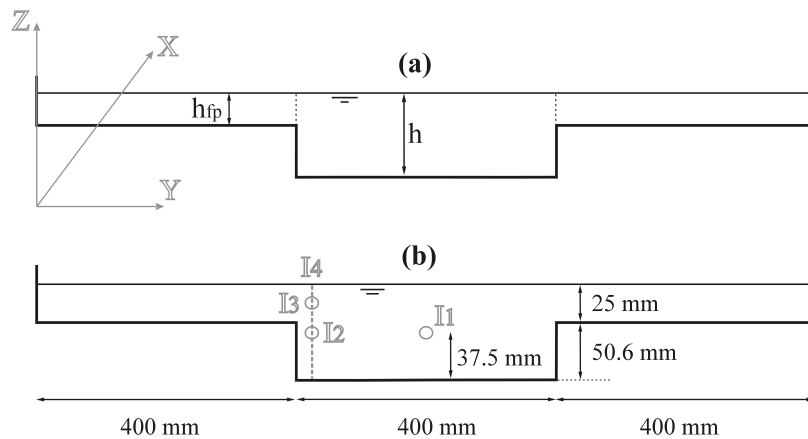


Figure 5.3: *Compound channel cross section with (a) variables definition and (b) locations of I1, I2, I3 (point injections) and I4 (line injection).*

5.1.2 Experimental technique

The final purpose of the experimental work presented in this thesis is to investigate full transport and deposition processes of fine sediments in prismatic compound channel. In a first approach, tests focus on diffusion/dispersion processes at the interface between main channel and floodplains, using solute tracers. Experiments reported in this chapter involved steady point and vertical line sources of tracer discharging into steady, uniform flow in prismatic compound section. The tracer used was diluted common table salt (NaCl), injected through a 13 mm diameter copper tube at a constant rate, supplied from a reservoir by a peristaltic pump. As advised by Lau & Krishnappan (1977), methanol was added to the solution in order to decrease the solution density and prevent undesirable gravity effects. Solute concentrations were chosen in order to optimize the conductivity measurements and adjust the injection speed to the flow velocity. The tracer plume was recorded by measuring concentration distributions in selected cross-sections downstream of the source.

Physical model

Experiments were performed in the UCL compound channel flume presented in section 4.1 with a bed slope $S_0 = 1/1000$. Tests were carried out at a single water depth (75.6 mm) and a discharge equal to 18.55 l/s. For this flow regime, a detailed set of velocity measurements was recorded in order to check uniform flow conditions. Strong interaction between the subsections is expected for the selected relative depth ($h_r = h_{fp}/h = 0.33$) as noted by Knight & Shiono (1996). The flow mean velocity is $U = Q/A = 0.369$ m/s and the main channel velocity is $U_{mc} = Q_{mc}/A_{mc} = 0.452$ m/s. The tracer was injected at a flow rate $m = 1$ g/s in 4 successive positions in the main channel, hereafter referred to as case I1, I2, I3 and I4 (see Table 5.1 and Figure 5.3). Concentration distributions were measured in cross-sections 1, 2, 4 and 7 m downstream from the injection point.

Measuring device

The choice of sodium chloride as a tracer is justified as follows: (1) it is easily available at low cost; (2) it has no toxicity for human beings; and (3) its concentration can be correlated to electrical conductivity. The measured conductivity ($\mu S/cm$) is linearly dependant on the tracer concentration C (g/l), this relation being dependent on temperature T° :

$$C(t) = f(\chi, T^\circ) = [\chi(t) - \chi_0(t)] \propto (T^\circ) \quad (5.1)$$

Table 5.1: *Tracer injection location*

Test case	Tracer injection location			Injection concentr.	Injection flow
	x(m)	y(mm)	z(mm)	C_0 (g/l)	rate (l/s)
I1	2.0	600	37.5	12.55	0.0797
I2	2.0	415	37.5	16.74	0.0597
I3	2.0	415	60.0	16.74	0.0597
I4	2.0	415	—	16.74	0.0597

where $\chi(t)$ is the conductivity value, $\chi_0(t)$ is the water baseline conductivity (due to solute already present in the reservoir water) and α depends on temperature.

Tracer concentration measurements are realized with specifically developed conductivity probes. Local conductivity is obtained by limiting the size of the control volume by a plastic tube (Figure 5.4). This configuration avoids perturbation of the measurement by wall or free-surface effect. The developed probes are sufficiently small to avoid excessive flow perturbation and sufficiently large to deliver reliable measurements. The conductivity is measured by observing electrical current variations between the two steel needles electrodes submitted to constant tension.

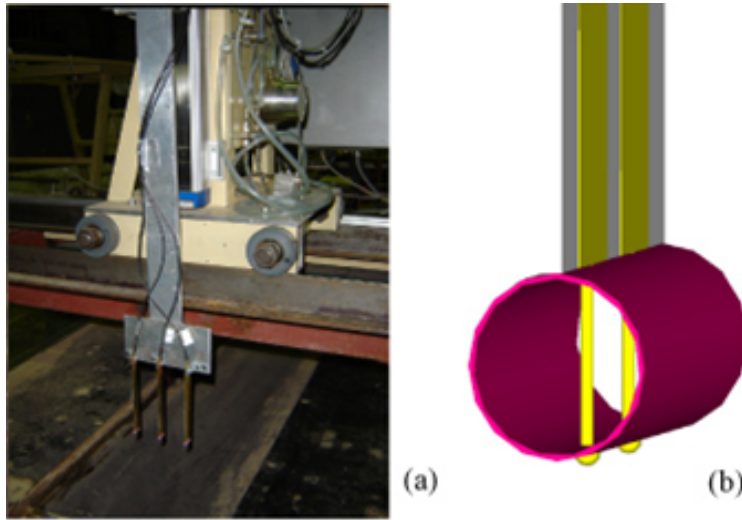


Figure 5.4: *Conductivity probes: mounted on automatic positioning device (a);(b) head with plastic tube and steel needles.*

Experimental setup

A series of samples in injection conditions were extracted to establish the relationship between the concentration of NaCl in water and the voltage output of conductivity probes (translation of current variations at the electrodes). The simultaneous use of two probes mounted on the automatic positioning device enables higher density of measuring points.

Since the water in the flume was recirculated, the base conductivity at the inlet of the channel was affected. This background conductivity $\chi_0(t)$ was measured continuously during each experiment in order to correct the measured value prior to data analysis. Figure 5.5 gives a general sketch of the experimental set-up.

Parasitic influences between the probes by ionization of the media reported in Fraselle *et al.* (2007) and electronic deviations were corrected in Fraselle *et al.* (2008) by improving the isolation. Reliability of the system was proven, since measured tracer mass flow rate (through integration of measured concentration profile together with velocities) showed a maximum of 3 % error with the injected tracer mass flow rate.

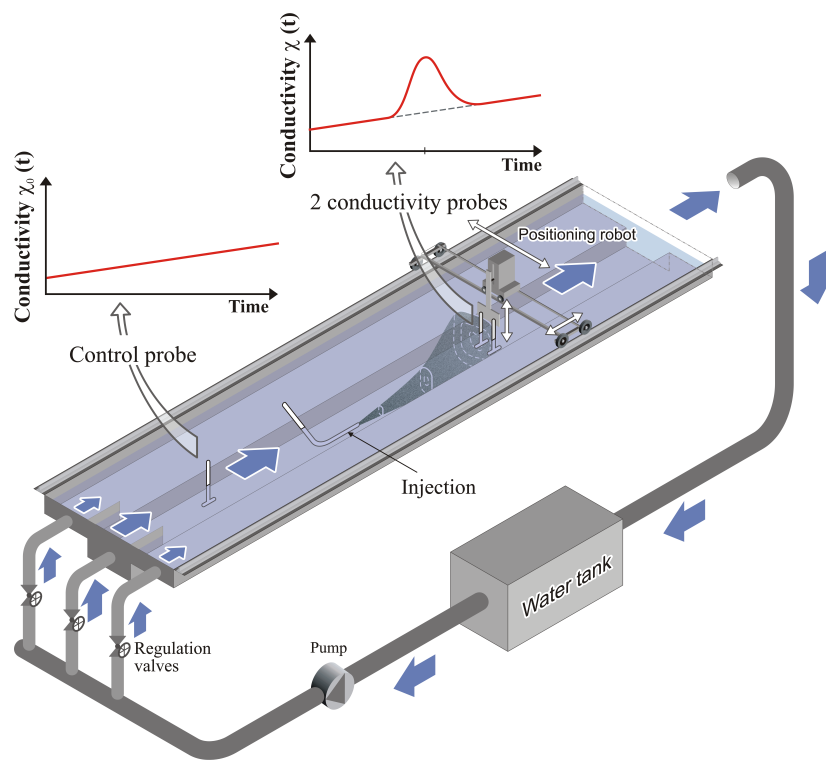


Figure 5.5: *Experimental set-up. The conductivity of recirculated water is measured upstream of the flume to calculate relative tracer concentrations.*

5.1.3 Analysis tool

A simplified form of the solute transport equation (3.11) developed in section 3.2.2 is solved using the finite difference approximation technique together with experimental longitudinal velocity profile. Comparing the simulations to experimental concentration measurements and choosing appropriate mixing coefficients should reveal the differentiation of transport intensities in the main channel and over the floodplains. This method was preferred to geostatistical methods used by some authors, such as generalised method of moments (Holley *et al.*, 1972; Boxall & Guymer, 2003), which is used to deduce the transverse mixing coefficient for instance, achieved by analysing the longitudinal variation of the transverse concentration variance. These methods require large amount of data as well as knowledge over transverse velocity and a much simpler method was used in this work.

Assuming steady conditions, uniform flow and encompassing convection due to secondary currents in the mixing coefficients, the equation (3.11) can be rewritten as:

$$u \frac{\partial C}{\partial x} = \frac{\partial}{\partial y} \left(D_y \frac{\partial C}{\partial y} \right) + \frac{\partial}{\partial z} \left(D_z \frac{\partial C}{\partial z} \right) \quad (5.2)$$

where C is the tracer concentration, x, y, z = longitudinal, lateral and vertical directions, u = longitudinal velocity component, D_y, D_z = the mixing coefficients in the lateral and vertical directions, which thus represent transport by turbulent fluctuations (eddy diffusivities) and by secondary currents.

As further simplification, prompted by Arnold *et al.* (1985); constant values of mixing coefficients are taken in each subsection, leaving us two partial differential equations to be solved: one in the main channel and the other over the floodplains. Such hypothesis could also be supported by the fact that for lateral spreading, it is customary to assume that the lateral mixing coefficient is proportional to the product of shear velocity u_* and average depth (h):

$$u_* \approx \sqrt{gRS_0} \approx \sqrt{ghS_0} \quad (5.3)$$

and those values stay almost constant in the quasi-separated part of the channel (Wood & Liang, 1989).

The boundary condition to be imposed for solute transport is that no material is lost through the boundaries of the flow and can be written as:

$$\frac{\partial C}{\partial \mathbf{n}} = 0 \quad (5.4)$$

where $\mathbf{n}$ is the outward unit direction normal to the boundary.

Flow is predominantly in the x direction and all second-order derivatives (turbulent diffusion term) in the longitudinal direction are thus neglected. Those partial equations are therefore assumed to be parabolic in the streamwise direction but elliptic in the cross-stream planes (Lin & Shiono, 1995). The solution was obtained by marching the 2-D cross section solutions along the streamwise direction, solving thus explicitly in longitudinal direction and implicitly in the other directions. An example of such procedure is found for a 2D-vertical problem in appendix B and described by Eq. (B.10).

5.1.4 Results and analysis

Validation experiment

The concentration profile 2 m downstream from the injection point for case I1 is shown on Figure 5.6. Full vertical mixing is not realised yet (but is effective at 4 m). Despite the use of methanol, light buoyancy effects persist. Further, secondary circulations are not measured in the flume. This makes a three-dimensional analysis difficult; since the numerical model does not account for any buoyancy effect. Relevant analysis of vertical diffusion is thus compromised. Therefore, only depth-averaged values, fully proper only in regions where vertical mixing is accomplished, enable us comparison with numerical results (Figure 5.7). Good agreement is reached with a lateral mixing coefficient estimated at $D_{y,mc} = 3.44 \times 10^{-4} \text{ m}^2/\text{s}$. Its non-dimensional value equals:

$$D_{y,mc}/h_{mc}u_{*,mc} = 0.17 \quad (5.5)$$

where $u_{*,mc}$ is the shear velocity in the main channel:

$$u_{*,mc} = \sqrt{gh_{mc}S_0} = 0.027 \text{ m/s} \quad (5.6)$$

The numerical tool is thus used to provide an estimation of the mixing coefficients. The analysis consists in finding two pairs of mixing coefficients, one for main channel and one for the floodplains such that the evolutions of the numerical and experimental tracer plumes correspond in space (due to density effects, this operation is realised on a depth-averaged manner, with vertical mixing dictated by literature). Those coefficients will be compared to results collated from published field studies.

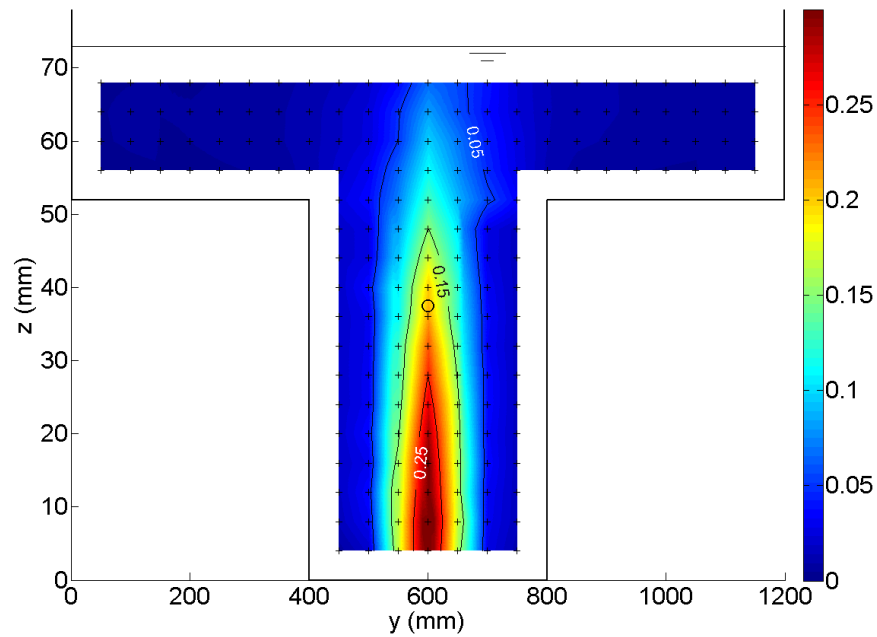


Figure 5.6: *Tracer concentration (g/l), I1 injection case, 2 m downstream from source.*

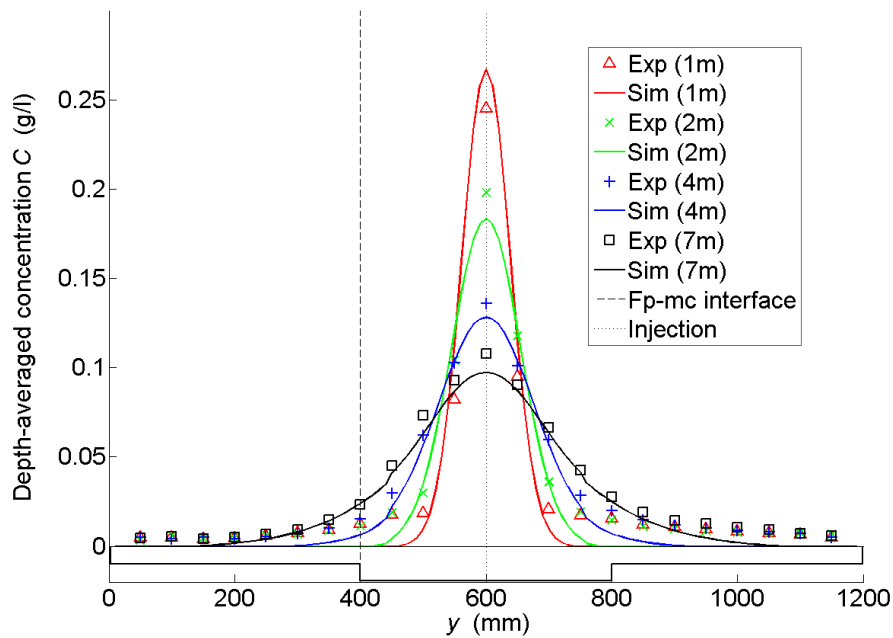


Figure 5.7: *Experimental and simulated depth averaged concentrations (g/l) at different distances from the I1 injection point (1 m, 2 m, 4 m and 7 m from the source).*

Steady point source injection in shear layer zone

The downstream spatial evolution of the tracer plume for case I2 is given on Figure 5.8. Figure 5.9 (left) shows the evolution of the depth-averaged concentrations for I3 case as Figure 5.9 (right) depicts them for I4 injection case.

The respective role of turbulence and secondary currents on transport rate is rather

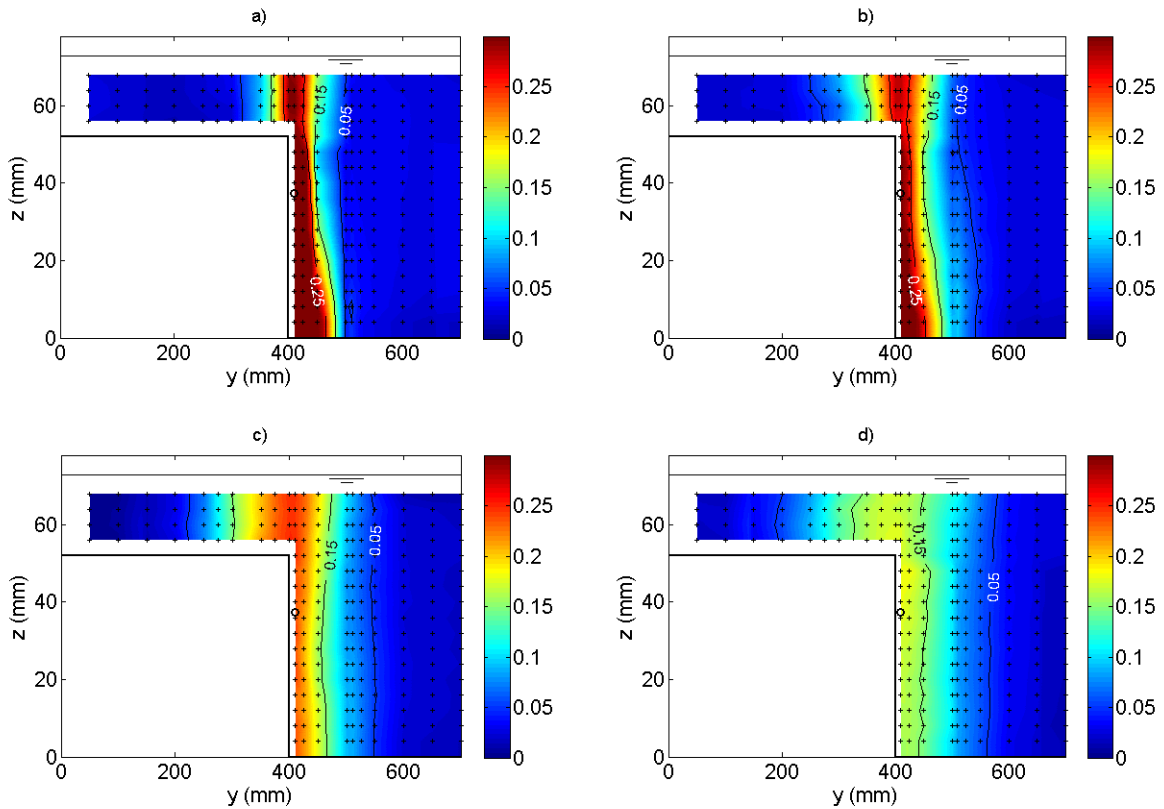


Figure 5.8: *Tracer concentration (g/l) with I2 injection case ($y = 415$ mm $z = 37.5$ mm); (a) 1 m distance from source; (b) 2 m; (c) 4 m and (d) 7 m*

complex. Shiono & Feng (2003) showed how the transport rate varies with the different mechanisms along the channel. Following the magnitude of secondary flow, it can become the main contributor. This occurs in the shear zone between the main channel and the floodplains, where the secondary flow magnitude reaches 4 ... 5 % of the main flow, and the tracer is transported laterally with large horizontal eddies that occur in shallow over-bank flow.

The observed asymmetrical expansion of solute on floodplain could be caused by vari-

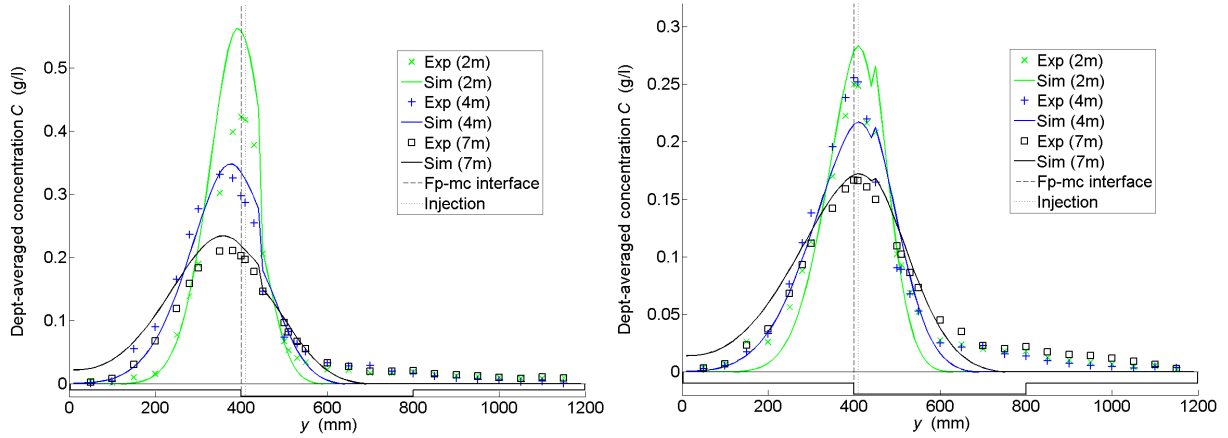


Figure 5.9: Experimental and simulated depth averaged concentrations (g/l) for (Left) I3 injection and (Right) I4 (vertical) injection case.

ations of secondary flow and eddy diffusivity due to hydrodynamic interaction between main channel and floodplain (Shiono *et al.*, 2003). For I2 and I4 injection cases, the maximum concentration is observed to move towards the bed discontinuity. This is not surprising since the step acts as a partially reflecting surface as discussed earlier in section 5.1.1. Further, the peak migration towards floodplain visible on Figure 5.9 (left) is due to injection in strong velocity shearing zone since the numerical concentrations follow the same trend. The velocity difference between the subsections ($U_{fp} < U_{mc}$) induces longer tracer stagnation over the floodplains and concentration increase.

The vertical injection (I4 case) was attempted to isolate the phenomenon of lateral diffusion, but vertical mixing due to strong turbulence for I2 case was so quick, that those two cases only show minor differences.

Discussions

Different combinations of mixing coefficients were tested for each injection case. The coefficients which gave the best matching between the numerical and experimental results showed very little variations for all the tested cases. Table 5.2 specifies the range of those coefficients in the main channel and over the floodplains (see equation (5.2)). Values for lateral mixing coefficients in the main channel lie within a range predicted by Rutherford (1994) for straight channels (Table 5.3). Deng *et al.* (2001) explain that variability of the mixing coefficients estimated from tracer experiments can be attributed to secondary flow, variations of water depth, velocity and diffusivity within the cross section.

Table 5.2: *Estimated mixing coefficients in the main channel and over the floodplains*

Coefficient	Estimated value
$D_{y,mc}$	$(0.16 \dots 0.24) h_{mc} u_{*,mc}$
$D_{z,mc}$	$(0.17 \dots 0.22) D_{y,mc}$
$D_{y,fp}$	$(1.7 \dots 2.0) h_{fp} u_{*,fp}$
$D_{z,fp}$	$(0.17 \dots 0.22) D_{y,fp}$

Over the floodplains, the estimated values for the lateral mixing coefficients are closest to what is expected for strongly curved channels where stronger mixing takes places due to intensive secondary circulations. Like Czernuskenko & Rylov (2005), this confirms the role of secondary currents in the mixing processes. As expected, the numerical model can only justify the strong migration towards the floodplains by a relatively high lateral diffusion coefficient associated with floodplains since secondary advection currents are not taken into consideration.

Regarding vertical mixing, Rutherford (1994) advocates the value of $D_z \cong 0.5D_y$ in straight channels but given that the analysis was made on depth-averaged manner, little care should be brought to interpretation of the estimated vertical mixing coefficients. They were however chosen with a constant value over the whole cross section and to reproduce approximately the global spatial behaviour of the tracer plume.

Table 5.3: *Empirical Formulae for evaluation of transverse mixing coefficient from Rutherford (1994)*

"Straight" channels	$0.15 \leq D_y/hu_* \leq 0.3$
"Meandering" channels	$0.3 \leq D_y/hu_* \leq 1.0$
"Strongly curved" channels	$1.0 \leq D_y/hu_* \leq 3.0$

5.1.5 Conclusions and recommendations

This section reported the results and analysis of the laboratory campaign initiated by Fraselle *et al.* (2007) and brought further by Fraselle *et al.* (2008) which aims at improving our understanding of these transport phenomena in compound channel. In a first stage, the study is based on an experimental simplification of the problem, where fine sediment diffusion is associated with the one of a solute (NaCl), much easier to handle.

Diffusion and dispersion processes in compound channels are thus investigated through tracer injections.

Fraselle *et al.* (2008) brought trustworthiness to the experimental technique, since the credibility of Fraselle *et al.*'s (2007) experiments were affected by measured tracer mass loss. Reliability of the system and designed probes was thus proven since only minor error persists between the measured and injected mass flow rate. Results from different injection positions were treated with a numerical simulation tool based on the development of simplified advection/diffusion equation in the same conditions as the physical model. Due to a lack of knowledge over secondary circulations, the model considers the migrations of solute introduced by the last as transport due to diffusion only. Despite the addition of methanol to the tracer solution, intending to make it neutrally buoyant, residual density effects prevented from an efficient analysis of vertical mixing and simulations were matched to experimental concentrations using a depth-averaged approach.

When injected on the flume symmetrical axis, the plume behaviour is correctly described with mixing coefficients in the main channel whose values lie within a range given in literature for straight channels. Furthermore, injections in the shear zone between the main channel and its floodplain show interesting trend of the tracer plume at the interface, with a skewed solute distribution on the floodplain. Larger lateral mixing coefficient on the floodplains was required to match the simulated concentrations with the experiments, confirming the enhanced action of complex turbulence mechanisms in the interaction zone on solute transport. The results seem thus to concur with observations reported in the literature and summarized in section 5.1.1. The mixing coefficients obtained could thus hereby become the starting point for diffusion calibration during fine sediment transport numerical analysis (chapter 6).

The method is promising and further efforts could however be brought in the future to improve the quality of the observations and of the analysis. Lower tracer concentrations could be used to avoid the downwards plunging of the plume due to density effects (Wood & Liang, 1989) and density driven secondary circulations (Ishigaki *et al.*, 2001) which could disrupt the flow field. Improvements could be brought to the analysis tool including three-dimensional velocity measurements (e.g. using PIV). Enhanced comprehension of which part of the transport is attributed to advection or turbulent diffusion would hereafter be expected. Further, the analytical solution which takes secondary flow into account developed by Shiono & Komatsu (2000) could enrich the analysis.

This study shows that it is possible to understand compound channel mixing with relatively low cost installations, electronic probes being totally home-made and tracer easily

available. Knight & Shiono (1996) insist on the strong dependence of the mixing mechanisms with relative depth and the need to investigate different geometrical configurations. Chronologically, the UCL compound channel flume has evolved towards fine sediment transport and deposit analysis after this work, counting on this tracer experimentation background to investigate diffusion and convection processes.

5.2 Sediment transport and deposition

5.2.1 Introduction

This section presents results from an experimental study of fine sediment transport and deposition in a prismatic symmetrical compound channel. The experimental campaign investigates fine sediment exchanges between the main channel and the floodplain. Following Garcia (2008) sediments are expected to be mainly transported by suspension, given its median diameter ($d_{50} = 91 \mu m$). Preliminary results are reported in Fraselle *et al.* (2010b).

More precisely, the lateral sediment transfer and the subsequent deposition patterns are studied. Two kinds of experiments were led. The first series were performed in a compound channel flume with mobile main channel bed and recirculating sediment-laden water from this subsection. The second series featured compound channel flow over a rigid bed initially sediment-free and sediment injection upstream of the main channel. Weighting of deposited sediment and digital imaging are used to evaluate the deposition formation. Results in terms of sediment deposition rate on the floodplains are presented. Lateral and longitudinal distribution are also analyzed for different water depth and floodplain roughness. Strong dependence upon those two parameters is observed. The percentage of mass transferred and deposited towards the floodplain is related to the floodplain's stream-power (Bagnold, 1966).

The experimental study of fine sediment transport and deposit analysis over the floodplains required heavy modification of the UCL compound channel flume, as explained in Chapter 4. The expected experiments were roughly modelled by different means in order to be able to make the best design choices (Dupont & Tiquet, 2008). Prior to results display, the experimental approach will be presented, developed together with De Monie & Van Obbergh (2009). Results are also enlightened by previous works on the subject, presented in the following section.

5.2.2 Previous works

For several reasons, our knowledge of sediment transport in rivers during overbank flow is limited. Indeed, most sediment transport experiments have been undertaken in simple channels with inbank flows. Further, compound channel flow with loose bed itself is poorly described, as turbulent flow structures for overbank flows have recently mainly been studied in rigid-bed channels (Bousmar *et al.* 2004; Knight & Demetriou, 1983; Knight & Shiono, 1996). Few authors studied stage-discharge in mobile bed compound channel (Atabay & Knight, 2006; Bousmar *et al.*, 2006) under some bedforms development conditions; together with the effect of floodplain roughness on bed form morphology (Tang & Knight, 2006).

Increased complexity is expected when suspension comes into play. Very few studies implying fine sediment transport and deposition on floodplains have been performed, and even less planning non-prismatic geometry. Chapter 2 already mentioned that laboratory experiments stayed exploratory until now (Benson *et al.*, 1997; Knight *et al.*, 1999; Bathurst *et al.*, 2002) and that deposit simulation models (James, 1985; Pizzuto, 1987) provide results that concurs previous laboratory and field observations: (1) an exponential decrease in the deposition rate with distance to the main channel; (2) a decrease in the sediment particle size deposited with the distance from the main channel; and (3) a depositional trend, which depends on the micro-topography of the floodplain.

If the action of turbulence on fine sediments may be assumed to be analogous to a diffusion-dispersion process (Graf, 1971), it does not account for all influences and specific empirical theories will often be used to describe sediment transport (Chapter 3), especially when bed load transport gains increasing importance (Garcia, 2008). Among those functions capable of predicting the (total) sediment transport rate in river reaches, we find the Engelund-Hansen sediment transport equation (3.74) (Engelund, 1966, Engelund & Hansen, 1967) and the Ackers-White formula (3.75) (Ackers & White, 1973) which are both to be used in this study, considering that their underlying hypothesis fit the present experimental conditions. Besides those transport laws extracted from experimental data, authors such as Bagnold (1966) adopted a more theoretical approach. Based on Bagnold's stream-power concepts, Verbanck (1995) suggested the following power balance equation (W/m^2 of riverbed area):

$$(\rho_s - \rho_l) g w_s C_{SL} R = \rho_l \eta_{SL} C_* u_* (u_*^2 - u_{*,c}^2) \quad (5.7)$$

where ρ_s is the sediment density, ρ_l the fluid density, w_s the effective settling velocity,

C_{SL} the suspended particle matter concentration, R is the hydraulic radius, C_* the non-dimensional Chezy coefficient ($C/g^{1/2}$), η_{SL} the suspension efficiency, u_* the shear velocity, and $u_{*,c}$ the shear velocity at the onset of motion for sediments forming the channel bed. Without η_{SL} , the right part of Eq. (5.7) is the stream-power, calculated as the product of one-dimensional velocity and shear stress in excess of that required to put the deposited particles into motion. The power balance simply expresses that the gravitational power of the transported material uses a certain fraction (η_{SL}) of the available stream-power to remain suspended in the water column. If the hydraulic radius R is replaced by $R = A/P$, and the wetted perimeter P is distributed in the right term, the left term $((\rho_s - \rho_l) g w_s C_{SL} A)$ even represents the submerged weight of sediments per longitudinal unit length and the right term is the resistance to friction. Eq. (5.7) clearly identifies the main parameters at stake when studying sediment transfer and deposition on the floodplains. Higher transfer of sediments from the main channel towards the floodplain could be characterized by an increased floodplain C_{SL} and ease of deposition is conditioned by η_{SL} , C_* and u_* .

5.2.3 Experimental approach

Physical model

Experiments were performed in the 10 m long compound channel flume with a bed slope $S_0 = 1.9/1000$ presented in Figure 4.4. This flume was used under 2 configurations, with respective cross-sections pictured in Figure 5.10. Either the experiments were performed over a mobile main channel bed ($h_s = 25$ mm), with no additional sediment feeding and recirculation of the main channel sediment-laden water (named "mobile bed" experiments). Or sediment was injected at the upstream end of the main channel section (Figure C.3) and transport was observed in fresh water over a rigid bed (named "rigid bed" experiments). For all experiments, the sediment median diameter was $d_{50} = 91\mu m$.

Experimental conditions

Different water discharges were used for both configurations, in order to modify the subsections transport capacity and water depth. The floodplain's roughness was also varied throughout the tests. Sediment deposits were observed over "smooth" floodplains, where the same material as the rest of the section was used, namely coated plywood. Further, the influence of "rough" floodplains with "trapping" abilities was tested by overlaying the

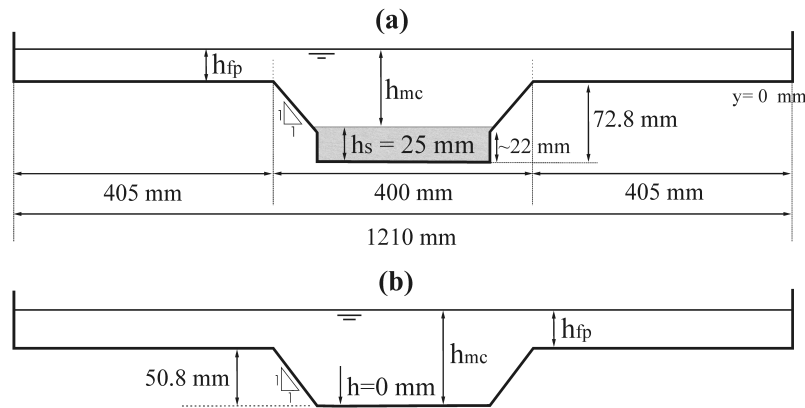


Figure 5.10: *Flume cross-sections for (a) mobile bed with recirculation experiments and (b) upstream sediment-feed over rigid bed experiments*

floodplains with steel sheeting (0.7 mm thickness) with squared 8 mm side holes (Figure 5.11). Finally, in order to reduce drastically the floodplain water velocity through increased drag, "vegetated" floodplains were tested by placing inverted V-shaped wire meshes with a V-angle of 30° every 50 cm in the longitudinal direction (Figure 5.11).

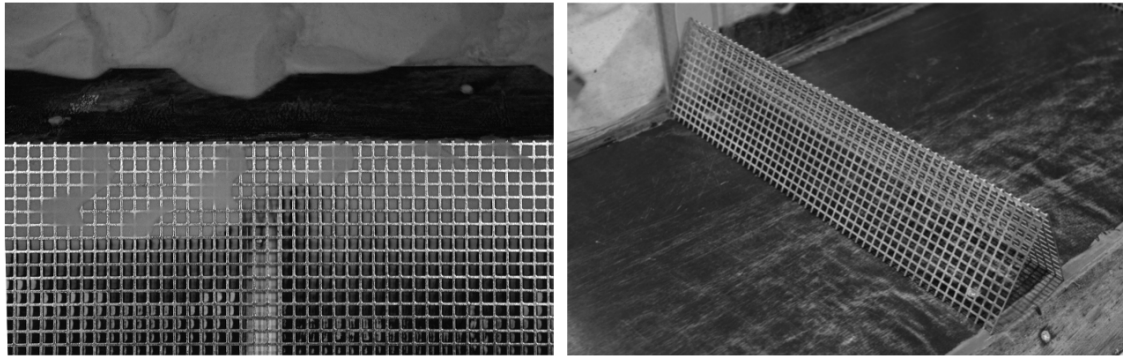


Figure 5.11: *(Left) Steel sheeting for rough floodplains. (Right) Inverted V-shaped wire mesh on the floodplains (1/50 cm) for "Vegetated" configuration*

Table 5.4 gives a summary of the experimental conditions for all different tests, with the whole section discharge Q (l/s), the main channel water depth h_{mc} (mm), the main channel discharge Q_{mc} (l/s), the main channel measured sediment concentration C (g/l) or for rigid bed experiments and the injected sediment load G_s (g/s).

Table 5.4: *Experimental conditions. MB stands for mobile bed experiments. RB stands for rigid bed experiments. S, R and V stand respectively for smooth, rough and vegetated floodplains*

		Q (l/s)	h_{mc} (mm)	Q_{mc} (l/s)	C (g/l)
Mobile Bed	Smooth FP MBS	6.6	56.22	5.33	0.24
		7.6	61	5.73	-
		9.1	62	6.07	0.4
		10	64.6	5.82	0.46
		15	70.5	7.66	0.75
		20	77.2	10.06	1
	Rough FP MBR	7.4	62.5	5.62	0.37
		9.09	65.3	6.43	0.44
		11	70.4	6.82	0.45
		14	74.6	8.54	0.38
		20	82.2	10.6	0.95
	Vegetated FP MBV	7.07	63.5	6.15	0.43
		9	69.7	7.35	0.5
		11	77.3	8.81	0.8
		14	93.2	10.7	0.68
Rigid Bed		Q (l/s)	h_{mc} (mm)	Q_{mc} (l/s)	G_s (g/s)
	Smooth FP	15	62	12.6	10.02
	RBS	20	69	14.98	13.78
	Rough FP	15	66	12.8	10.02
	RBR	20	72.7	15.5	13.78

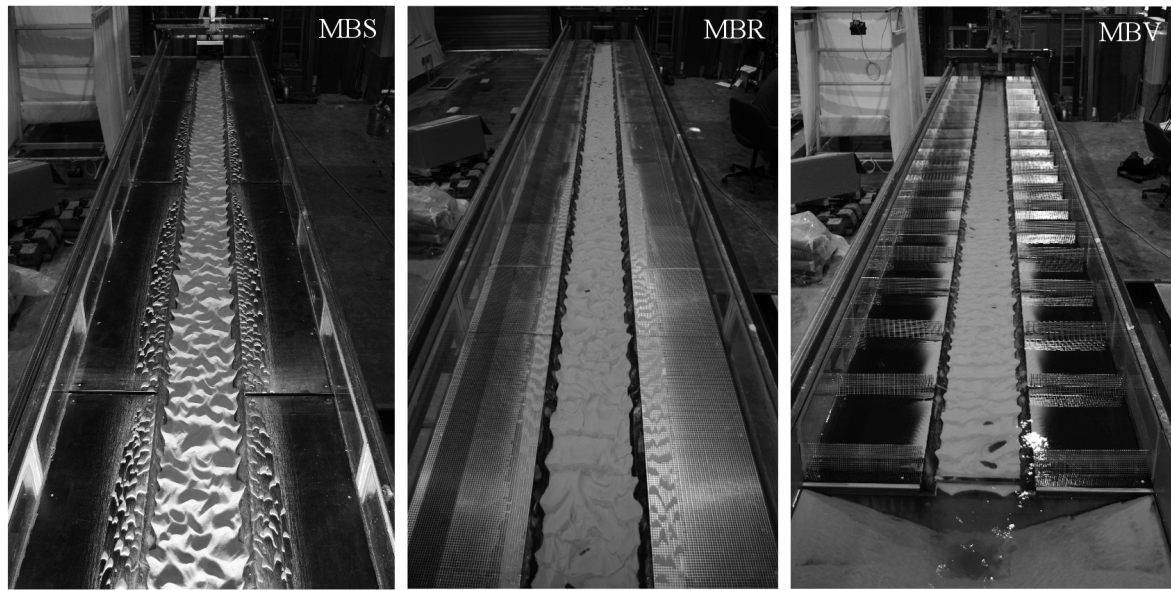


Figure 5.12: *Post-experimental flume view for mobile bed experiments. (Left) MBS with smooth floodplains. (Center) MBR with rough floodplains. (Right) MBV with "Vegetated" floodplains.*

Measuring techniques

The main channel sand bed was initially screed to the same slope as the fixed floodplains (1.9×10^{-3}), and then the water discharge set. As sediment transport and bedforms developed, the tailgates were adjusted until the water surface slope, S_w , was the same as the bed slope, S_b . Finding the appropriate tailgate settings was time consuming, as any slight change in tailgate setting altered the bed resistance and the flow depth, and produced non-uniform flow. Attaining a quasi-equilibrium condition was thus a trial and error process. The morphological equilibrium was verified using ultrasonic probes for free surface detection and bed profiler PV-07 (Delft Hydraulics) for sediment bed tracking (Section 4.3). For every flow regime, a detailed set of streamwise velocity measurements (Pitot tube) was recorded in order to check uniform flow conditions.

The dynamic evolution of sediment deposition was monitored by two different means. Sediment traps (Figure 5.13) were installed on the floodplains every 2.5 meters ($x = 2.5$ m, 5 m and 7.5 m) to capture mobile sediment deposits (slowly drifting downstream) and to give thus a measure of the sediment transferred to the floodplains, deposited and not resuspended. During and after every experiment, the global sedimentation pattern was

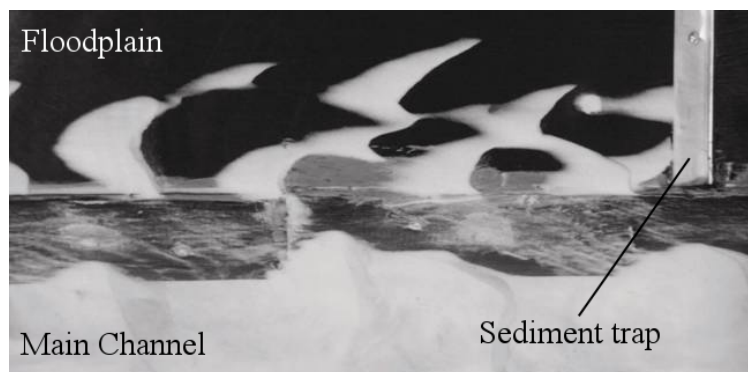


Figure 5.13: *Sediment captured in traps displayed every 2.5m*

pictured using digital imaging techniques (Section 4.3.3). An example of such instantaneous global view is depicted in Figure 5.19(a).

The main channel sediment transport capacity was deduced from numerous samplings made all along the flume. Sand-water mixtures were extracted from the flow using a siphon tube and dried to provide local concentration. The measured concentrations were confronted to sediment transport prediction laws and we see in Figure 5.14 that the Engelund-Hansen and Ackers-White calculated sediment load give a relatively correct approximation. When reaching higher flow regimes, where equilibrium is more difficult to maintain and to measure, concentration data become more scattered, somehow due to bedform instability observed when reaching morphological transition regime (Figure 4.10). The scattering could also come from the presence of bedforms which makes it difficult to measure the targeted sand bed slope S_b . Post-experimental manipulations included weighting of sediments in sand traps but also sediment mass deposited over the floodplains, with distinction of various longitudinal and lateral zones. Moreover, very precise topography measurements were realized all over the flume by using a laser distance sensor (Keyence LK-G).

Hydraulic conditions

Figure 4.7 shows how the Manning roughness for the rigid bed was deduced. The discharge increases when the floodplain section area contributes to the conveyance with a characteristic curve slope change when reaching bankfull level. This figure also suggests that the EDM is most appropriate to predict stage-discharge relationships in compound channels, by its ability to account for momentum exchange between the subsections.

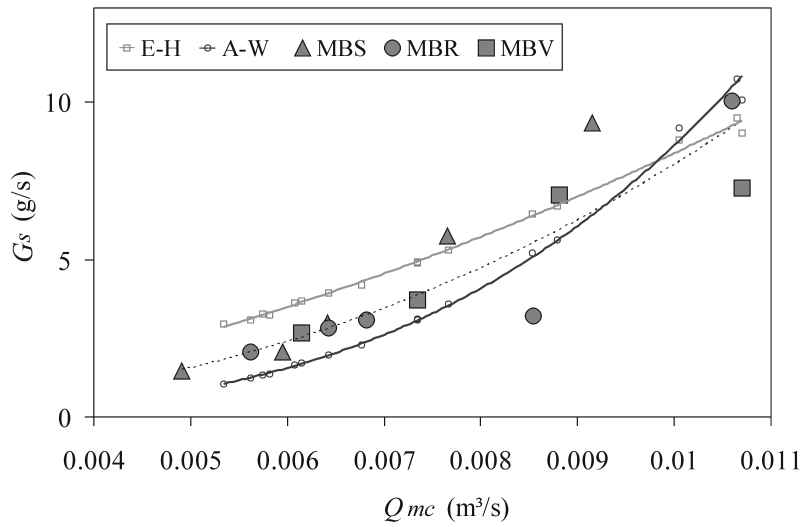


Figure 5.14: Main channel sediment transport capacity G_s (g/s) function of the main channel flow regime Q_{mc} (m^3/s) through (a) Engelund-Hansen (b) and Ackers-White relationships using experimental hydraulic parameters and (c) experimental concentration measurements.

The stage-discharge curves for all test cases are plotted in Figure 5.15, together with EDM theoretical predictions, using appropriate Manning coefficients to model the different surface roughnesses. The uniform water depth on the floodplains h_{fp} is computed as the uniform water depth in the main channel h_{mc} (for which water surface and bed level measurements are taken) minus the floodplain level. With the presence of dunes for the mobile bed cases, this h_{mc} is more a mean water depth. This is even more obvious for MBV cases where the presence of V-shaped profiles create local depth variations between two profiles. Table 5.5 presents the roughness coefficients used for stage-discharge predictions. n_{mc} is computed as a composite value for the main channel which comprises lateral 45° slopes together with the main channel plane bottom. *Italic* values indicate that part of it that is based on speculations, while non-slanted values indicate that experimental proof has been brought to the roughness coefficient. A larger discharge is obtained for the rigid bed cases than for the mobile bed ones. The bedforms observed in the latter case induce a larger flow resistance.

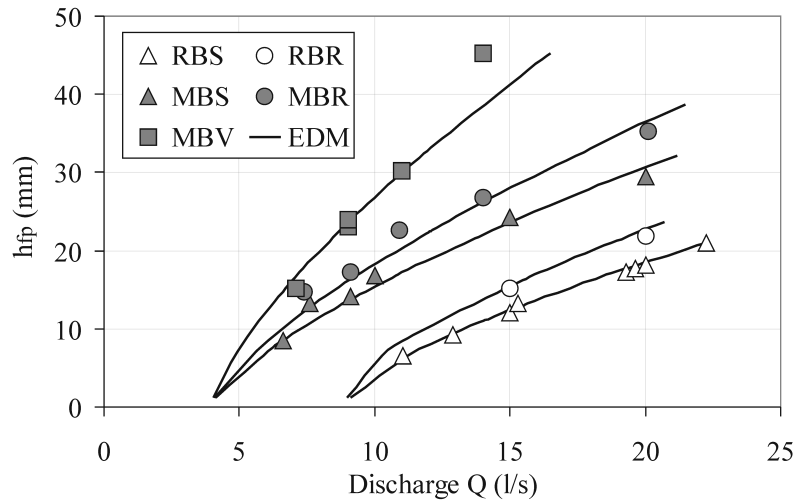


Figure 5.15: Stage-discharge curves for overbank flow and EDM predictions

Table 5.5: Roughness coefficients in each subsections

	n_b ($m/s^{1/3}$)	n_w ($m/s^{1/3}$)	n_{mc} ($m/s^{1/3}$)	n_{fp} ($m/s^{1/3}$)
RBS	0.0091	0.011	0.0097	0.0091
RBR	0.0091	0.011	0.0097	0.013 ¹
MBS	0.025 ²	0.013 ³	0.0215	0.0091
MBR	0.025	0.013	0.0215	0.013
MBV	0.025	0.013	0.0215	0.035 ⁴

¹ Determined experimentally in a rectangular flume

² Speculation on average roughness due to grains and bedforms

³ Speculation on rougher 45° slope due to sand and bedforms

⁴ In a rectangular flume, Manning roughness varied within 0.03-0.09 following the water depth

A Manning roughness of $n_{fp} = 0.013$ ($s/m^{1/3}$) was found for surfaces covered with steel sheeting. Even if an evolution of the loose bed roughness n_b with flow regime is proven (Figure 4.11), a mean value was preferred to express increased roughness due to bedforms, for which large scattering is expected when reaching the main channel morphological transition regime (Figure 4.10). This choice probably explains partially the difference between data and EDM predictions for MBS and MBR in Figure 5.15 where the water depth h_{fp} is under-evaluated at first, and over-estimated afterwards, due to

dune progressive washing out. V-shaped profiles are trickier to characterize: in a rectangular flume, the manning roughness varied within 0.03-0.09 following the water depth until submergence. Once again a fixed value of $n_{fp} = 0.035$ was chosen, and flow resistance for MBV is clearly under-estimated for higher flow regimes.

Figure 5.16 shows the depth-averaged velocity distribution for all experiments. For rigid bed cases, a classical compound channel flow behaviour is revealed. Larger main channel velocities generate an interfacial shear layer. When the discharge increases, the floodplain velocity increases faster than the main channel one, and the velocity gradient decreases.

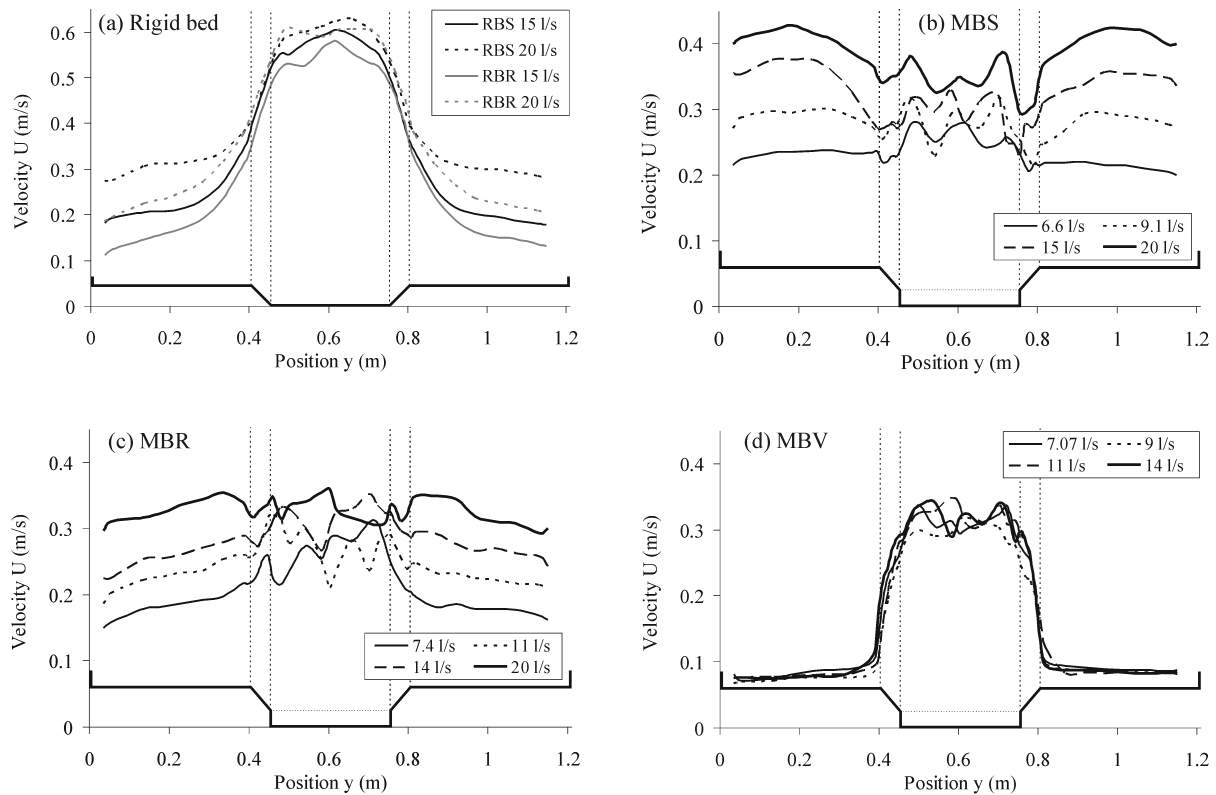


Figure 5.16: Depth-averaged velocity distribution: (a) Rigid bed series, (b) MBS, (c) MBR, and (d) MBV

For mobile bed with smooth floodplain experiments (MBS), the floodplain velocity increases rapidly and become even larger than in the main channel. Compound channel experiments with a loose bed in the main channel show that, with the development of bedforms which affects the flow resistance, the main channel velocity decreases and can become lower than the floodplain velocity (Bousmar *et al.*, 2006). This results in a re-

versal of the momentum transfer between the main channel and the floodplains, which obviously affects the channel conveyance.

This main channel flow resistance affects similarly the discharge distribution between main channel and floodplain (Figure 5.18a) where the floodplain discharge becomes larger than the main channel one.

Once again, the profiles for MBV cases are truncated parts of the reality, as the velocity measurements were taken at mid-distance from V-shaped profiles, and are probably not representative of the uniform flow mean velocities.

Stage-discharge curves and discharge distribution for mobile bed experiments are shown on Figure 5.17 and Figure 5.18, respectively. EDM computations using the roughness coefficients from Table 5.5 are also presented, together with SCM and DCM calculations. Predictions do not differ so much except for the MBV case, where roughness on the floodplain is bigger than in the main channel. In contrast with MBS experiments, the flow distribution for MBR and MBV experiments is similar to rigid bed cases (Figure 4.8), where the portion of the total discharge flowing on the floodplain increases with flow depth, but remains always lower than in the main channel.

5.2.4 Results and discussion

Careful attention towards the influence of the hydraulic parameters on the sediment deposition rate over the floodplain is requested. The excess floodplain stream-power, the right hand side of Eq. (5.7) without η_{SL} , seems to characterize correctly the available driving force available for suspended transport (Verbanck *et al.*, 2007). The floodplain stream-power was calculated for every experiment using the measured water depth and velocities (Figure 5.20). Those curves already show the effect of the floodplain configuration, with for a given water depth more stream-power available for smooth than for roughened or vegetated floodplains. The stream-power in excess was computed by using van Rijn's definition of initiation of motion defined in Eq. (3.57). All experiments show hydraulic conditions which exceed this theoretical threshold for sediment movement.

Qualitative description

Figures 5.12 and 5.19 presents deposits for different series of experiments. Different experimental procedures were used in each case. After installation of uniform flow, small

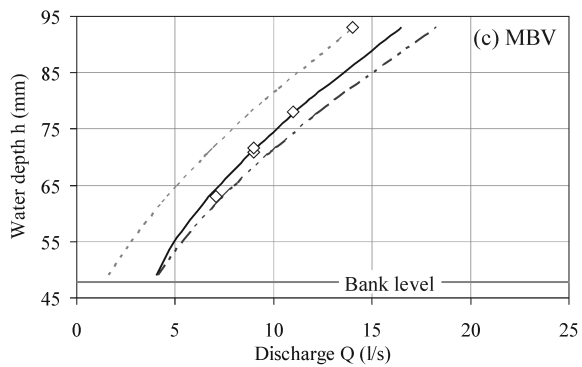
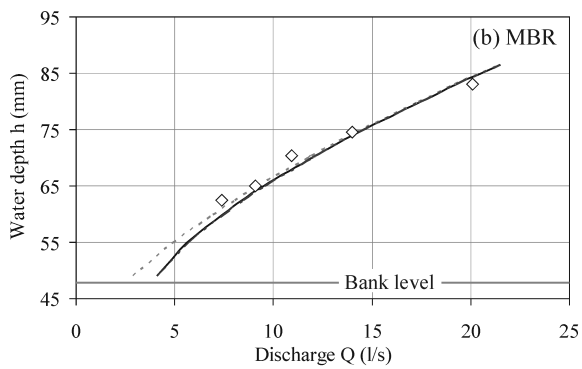
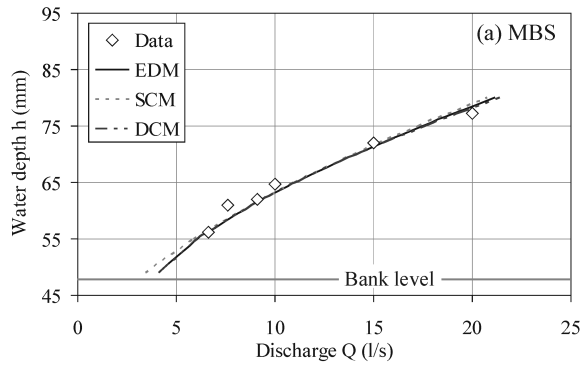


Figure 5.17: Stage-discharge. Experimental data and computation (a)MBS (b)MBR and (c)MBV

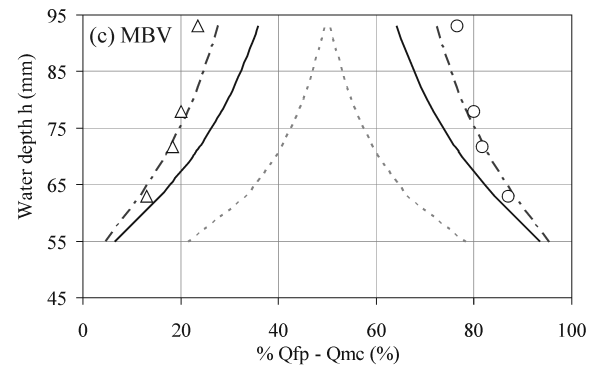
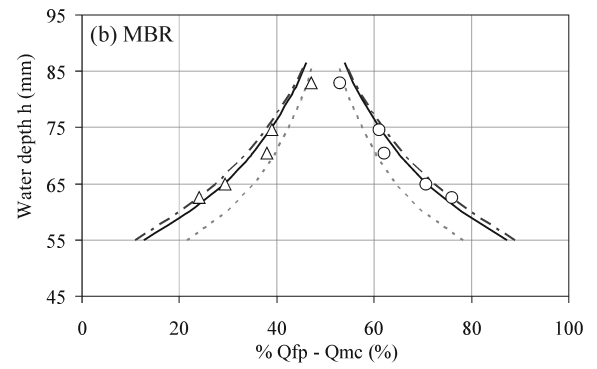
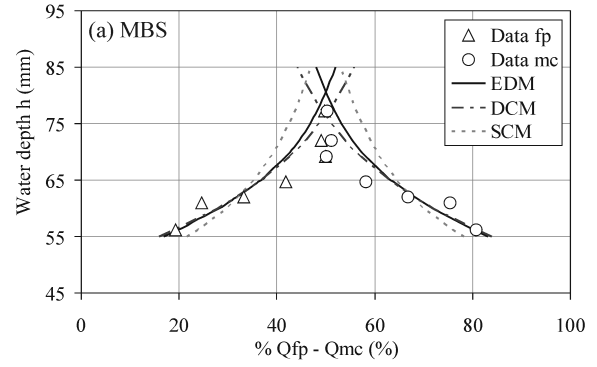


Figure 5.18: Discharge distribution. Experimental data and computation (a)MBS (b)MBR and (c)MBV

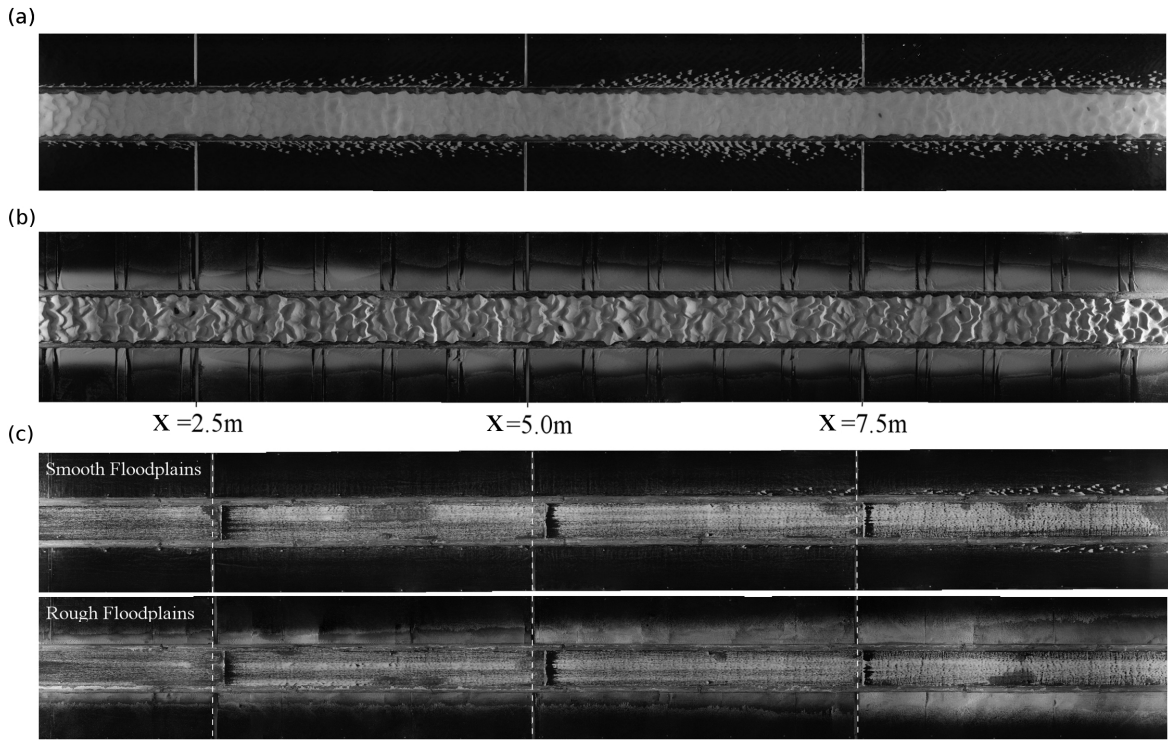


Figure 5.19: (a) Instantaneous deposition patterns for MBS 15 l/s experiment. (b) Final deposition patterns for MBV 11 l/s experiment. (c) Final deposition patterns for smooth and rough floodplains and rigid bed at 20 l/s (RBS/RBR)

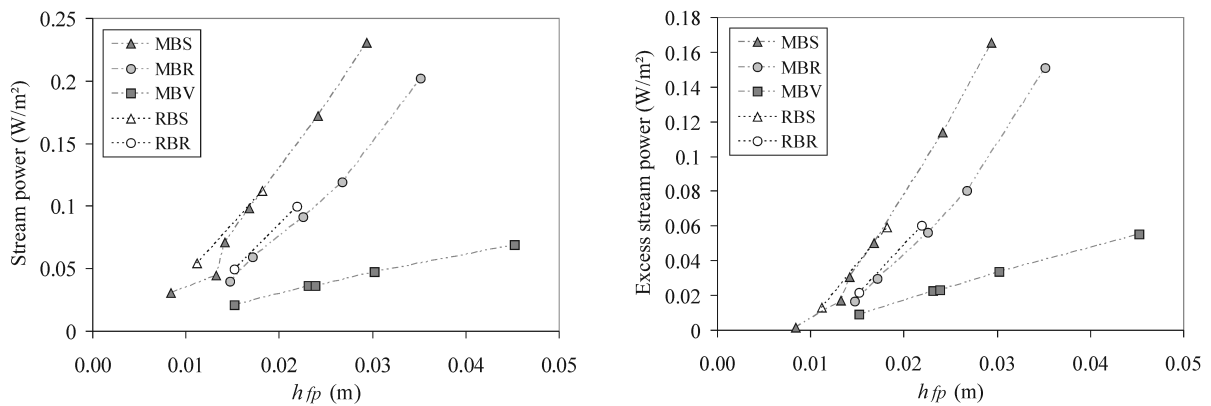


Figure 5.20: (Left) Floodplain stream power (W/m^2) function of subsection depth h_{fp} (m) for mobile and rigid bed experiments. (Right) Excess floodplain stream power (W/m^2)

sediment stacks appeared for mobile bed with smooth floodplains (MBS) experiments. Due to the smooth surface, those deposits were slowly drifting downstream, with a velocity depending on the distance with the main channel's banks. After some time of sand accumulation, those deposits tended to stabilize and some kind of "regime" deposits could be defined. By convention, regime deposition were formed one hour and a half after the beginning of the experiment. Digital imaging and deposits weighting were used to check, for a given discharge, that stable regime deposition occurred from one experiment to another. Smooth floodplains allowed sand to deposit, stay mobile, and even to be lifted into suspension once again. Further, deposit patterns were very sensible to local velocities.

When floodplains are overlayed with steel sheeting, the trapping ability of the surface hinders the resuspension phenomenon. No regime deposition was expected and floodplain sediments were wiped off at the beginning of the transient deposition analysis. Sand accumulated faster on the floodplains and motionless deposition was observed, the steel sheeting protecting the deposits from uplifting due to vertical turbulence and horizontal drifting. At some places where the deposition flux was most intense, sand deposition heights exceeded the 0.7 mm thickness of the steel sheeting and mobile deposition began to appear. To avoid sand deposit loss by resuspension, the experiment was stopped when such phenomenon occurred.

Vegetated floodplains showed very small velocities and when a sand grain entered the floodplain from the main channel, it was almost immediately deposited. Once again, transient deposition was analyzed, as sand could accumulate very easily on the floodplains. The V-shaped wire meshes created a very localized drag which influenced the global sand deposit patterns.

Rigid bed experiments were investigated under transient sediment deposition conditions, with deposits forming slowly and increasing height with the ongoing upstream sand injection. For both smooth and rough floodplains, sediment was transferred to the floodplains, due to turbulent diffusion and momentum exchange; suspended sediments were observed along the flume and on the floodplains. Deposits stayed mobile with smooth floodplains only and fine sediment bedload was responsible for sediment stacks migration.

Mobile bed with smooth floodplains

Lateral transfer and deposition of sediments from the mobile main channel towards the smooth floodplains (MBS experiments) was analyzed under regime conditions. Once the regime (mobile) deposits were formed over the floodplains with the associated main channel bedforms, the sediment traps were wiped out and the collected sediments from this time gives by mass conservation the lateral transfer rate of sediments towards the floodplains and per unit length ($g_{s,l}$) for the 2.5 m long zone preceding the sediment trap:

$$g_{s,l} = \frac{G_{s,trap}}{L} \quad (g/h \cdot m) \quad (5.8)$$

with $G_{s,trap}$ the collected sediment per unit time and L the length of the zone preceding the trap. This equation comes with the hypothesis that no sediment jumps towards the next 2.5 m longitudinal zone, which is justified by the triangular-shaped deposition pattern in each zone (Figure 5.19 (a)). Figure 5.21 presents $g_{s,l}$ as a function of the discharge Q (l/s) computed for the different longitudinal sections of the flume. $g_{s,l}$ generally increases with the flume discharge and with the longitudinal position of the zone, which seems to indicate an enhanced transfer downstream of the flume along with convection/diffusion of sediments on the floodplains. This enhanced transfer could be due to the downstream progressive development of vertical axis vortices at the interface (see Appendix F), which influence the lateral mass flux, as explained later. Figure 5.22 gives an example of lateral distribution of trapped sediment for MBS 15 l/s experiment, which illustrates once again the lateral diffusion of sediment on the floodplain.

An interesting trend of the regime deposition is shown in Figure 5.23, as if higher transport capacity of the main channel first favours the formation of deposit (more sediment transferred and C_{SL} from Eq. (5.7) increases) but is confronted then to a sudden decrease of those deposits when a certain threshold of floodplain stream-power is reached. This means that from this point, increase in resuspension of deposited sediments on the floodplain is more efficient than increase in lateral sediment transfer towards the floodplain.

Figure 5.21 expressed an enhanced lateral transfer with increasing discharge which is imputed to an increased main channel transport capacity (Figure 5.14). If this trend is proven for MBR experiments, MBS show a gentle diminution for extreme discharges, which could be due to a less effective transfer as a result of inversed momentum transfer (Figure 5.16b).

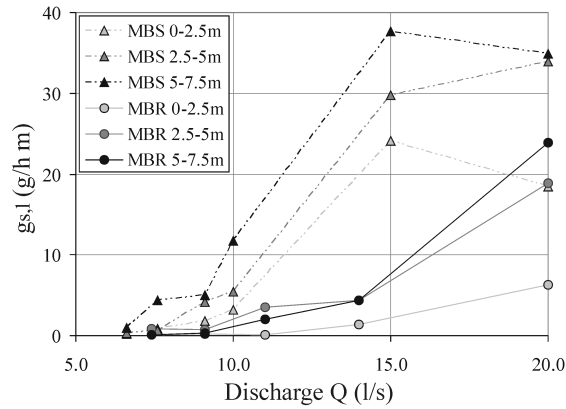


Figure 5.21: $g_{s,l}$ ($\text{g/h} \cdot \text{m}$) function of discharge Q (l/s) for MBS and MBR experiments. Sediment trapped at $x = 2.5$ m, $x = 5.0$ m and $x = 7.5$ m (Left floodplain)

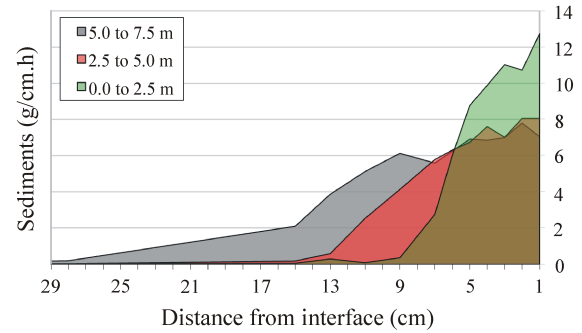


Figure 5.22: Lateral distribution of sediments ($\text{g/h} \cdot \text{cm}$) in different traps for MBS 15 l/s (Left floodplain). Abscissae are the distance from the main channel edge

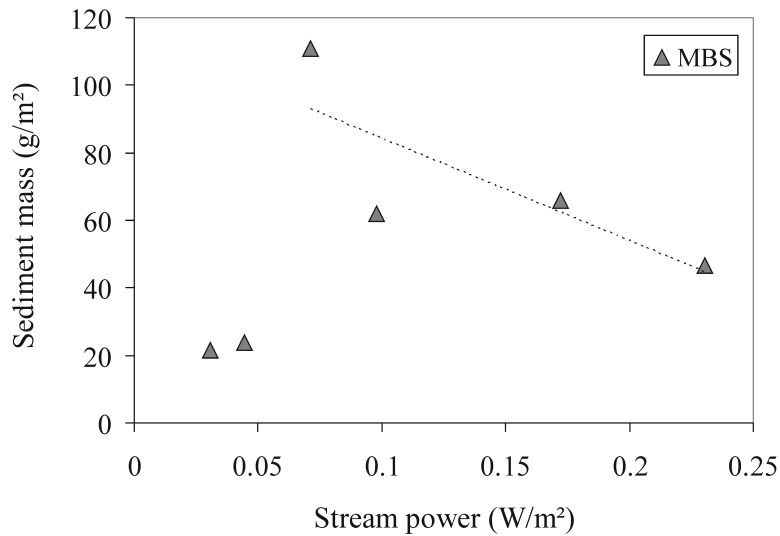


Figure 5.23: Equilibrium sediment deposits on floodplains (total mass (g/m^2)) function of floodplain stream-power (W/m^2) for mobile bed with smooth floodplains experiments (MBS)

Effect of floodplain roughness

We already observed in Figure 5.20 and Table 5.4 the effect on hydraulic parameters of changes in floodplain configuration. Floodplain roughening increases local water depth and decreases velocity but also changes sediment deposition patterns. Steel sheeting on the floodplains (MBR and RBR) enhances sediment fixation/absorption capacity of the floodplain and limits sediment resuspension. Convection/diffusion processes on the floodplain and local velocities condition the longitudinal distribution of sediment deposits (Figure 5.24). Meanwhile, vegetated floodplains create great additional drag in the flow and very low velocities force the transferred sediments to almost immediate deposition, as suggested by Figure 5.24, where relative deposition shows more uniform trend. This tendency is also illustrated on Figure 5.19 (b), where the final deposition pattern for MBV 11 l/s is presented.

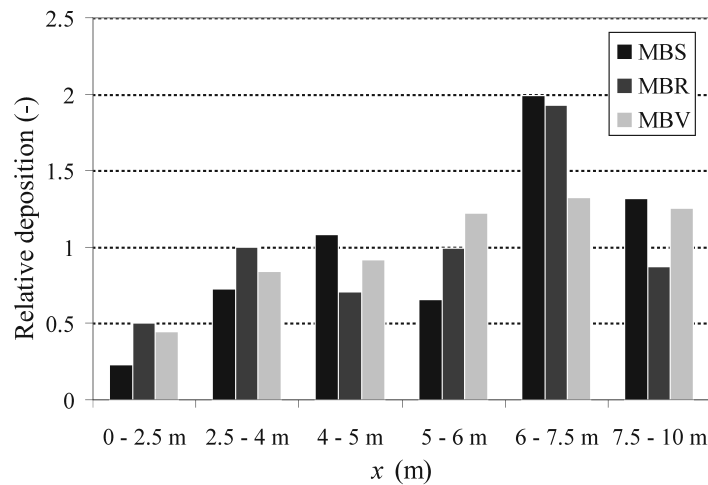


Figure 5.24: Floodplain relative deposition (-) (both sectional floodplain's deposits per longitudinal unit length/mean deposits per unit length) along the flume for MBS 15 l/s, MBR 14 l/s and MBV 14 l/s

Lateral deposition rate profiles suggest this ability of rough/absorbing floodplains to limit resuspension in interaction zone near the interface where higher velocities and bed shear take place (Figure 5.25). This phenomenon was also underlined by James (1985), where he used *smooth* surface defined numerically by a zero deposition probability $p = 0$ and *absorbing* surface showing $p = 1$.

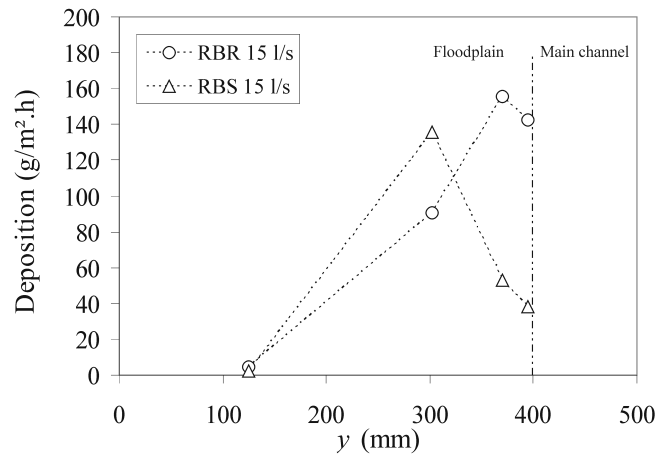


Figure 5.25: A comparison of deposition rate ($\text{g/m}^2 \cdot \text{h}$) over the (10 m long) left floodplain near the interface for experiments over rigid bed with smooth or rough floodplains (RBR 15 l/s and RBS 15 l/s)

Lateral sediment transfer over rigid bed

Injection conditions for rigid bed experiments are well-controlled, and a precise percentage of mass deposited against injected mass is presented in Table 5.6. By nature, RBR experiments compared to RBS experiments measure different deposition rates. Due to its absorbing characteristics, rough floodplain experiments will provide a percentage of sediments transferred towards the floodplains and deposited while smooth floodplains indicate a transfer/deposition rate of non-resuspended sediments.

Table 5.6: Percentage of sediment mass collected on the floodplains towards mass injected in the main channel

	$Q=15 \text{ l/s}$	$Q=20 \text{ l/s}$
RBS	1.8 %	0.82 %
RBR	6.02 %	8.48 %

Figure 5.19(c) shows the impact of resuspension on sediment deposits and triangular-shaped deposition show the intensity of convection/diffusion processes on the floodplains, which can be used for numerical models qualitative validation.

Floodplain deposition function of stream-power

As for MBS experiments, sediments collected on the floodplains (per unit time) are plotted against the floodplain stream power in Figure 5.26 for MBR, MBV, RBS and RBR experiments. Obviously, vegetated floodplains offer perfect conditions for lateral sediment transfer and deposition. Small velocities favour sediment deposition and high velocity gradient at interface enhances lateral sediment mixing and transfer (Prinos, 1992). This phenomenon is even more evident while focusing on MBR and RBR where for the same hydraulic conditions and trapping capacity of the floodplain, more sediment is collected for the latter. Figure 5.27 presents the depth-averaged velocity profiles in both cases. The high velocity gradient for the rigid bed case favours momentum transfer in the shear zone, together with sediment transfer, along with vertical axis eddies development which positively influences the interface sediment mass flux (Guymer *et al.*, 1998).

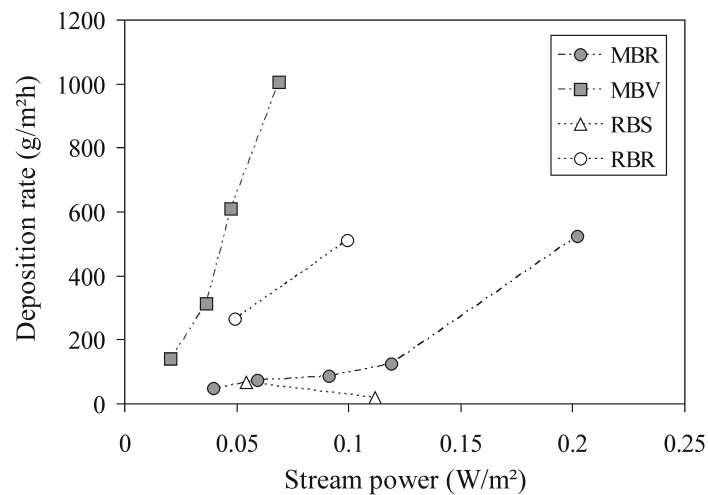


Figure 5.26: Sediment deposition rate on floodplains (total mass (g)/experiment time(h)/m²) as a function of the floodplain stream-power (W/m²) for MBR, MBV, RBS and RBR experiments

Parallel to MBS experiment, RBS show a decreasing sedimentation rate with floodplain stream-power, as if the present evolving hydraulic conditions were more likely to increase resuspension than lateral sediment transfer, in the absence of trapping capacity of the floodplain.

Analysis of Figure 5.26 shows in a certain way the parameters that should complete this work in a near future: (a) the lateral velocity gradient in the shear zone at the

interface between main channel and floodplain which acts on the transferred sediment and influences C_{SL} and the available sediment for deposition; and (b) the floodplain suspension efficiency η_{SL} which is presumed to show a link with the hydraulic conditions over the floodplain and characterizes the conditions for deposition.

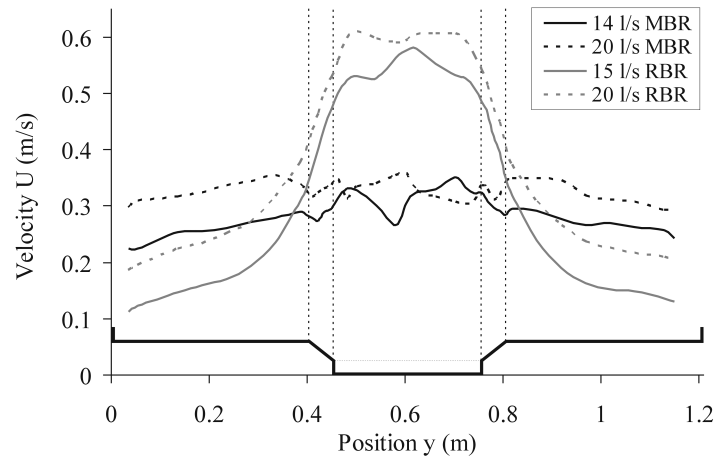


Figure 5.27: Cross section ($x = 8$ m) depth-averaged velocities U (m/s) for mobile bed and rigid bed experiments with rough floodplains

5.2.5 Summary and conclusions

Sediment transport and deposition during flood is a challenging problem for the engineer. The present laboratory campaign was initiated to improve our understanding of these phenomena. The study focuses on sediment exchanges between main channel and floodplains. More precisely, the lateral sediment transfer and the subsequent deposition patterns are studied. Two different experimental configurations were considered, with experiments either performed over a mobile main channel bed with recirculation of the main channel sediment-laden water, or with sediment feeding at the upstream end of the main channel section and transport observed in fresh water over a rigid bed. In both cases, the collected sediment on the floodplains gave a measure of the transfer rate of sediments towards the floodplains from the main channel, deposited and not re-suspended.

The study presents the influence of hydraulic parameters on sediment deposition rate, taken into account through the calculated stream-power over the floodplains. A quantitative and qualitative study of sediment deposition is also performed in correlation with various floodplain roughness or configurations.

Smooth floodplains basically showed a certain threshold in stream-power where increase in re-suspension of deposited sediments is more efficient than increase in lateral sediment transfer, due to higher main channel transport capacity. A thorough study on this re-suspension phenomenon could link those experiments to the roughened floodplain cases.

Steel sheeting playing the role of absorbing surface with higher roughness presented a constant evolution of the deposition with the stream-power over the floodplains, which comes along with an increased sediment concentration over the floodplains due to an enhanced lateral sediment transfer, especially when high velocity gradient is present in the interaction zone.

Vegetated floodplains reduce drastically the velocities over the floodplains and optimize the sediment transfer towards the floodplains, but also the conditions for deposition. This indicates that when rivers invade their floodplains, enormous quantities of sediments could be collected especially in these vegetated zones.

This study reports comprehensive experimental data which can be used to validate different numerical/theoretical models (Chapter 6). Ultimately, the determination of the influence of the transfer of longitudinal momentum on suspended particles should lead to the ability to model the sediment deposits laid on the river banks. This would request a thorough investigation of all turbulent features present in such geometry which condition the mixing mechanisms, but also sediment transport.

Chapter 6

Two-dimensional modelling with *Telemac*

6.1 Introduction

Flow in a compound channel displays a complex three-dimensional structure resulting from the interaction between the floodplain flow and the main channel flow (e.g. Shiono & Knight, 1991; Shiono & Muto, 1998). In recent years, two-dimensional (2-D) and three-dimensional (3-D) Computational Fluid Dynamic (CFD) models have increasingly been used for compound channel cases, in order to predict the flow fields and improve our understanding of turbulent flow behaviour (Rameshwaran *et al.*, 2008).

Fine sediment transport modelling, using global 3-D approaches are heavy to implement for large scale modelling and request non-isotropic turbulence models. They could however bring detailed informations on local phenomena that matter. They could help to judge which part of sediment transport is due to advection more than turbulent diffusion.

2-D depth-averaged CFD models with a hydrostatic pressure assumption are not capable to model transport due to secondary circulations (longitudinal vortices) explicitly. However, they are able to model to some extent horizontal shear layer turbulent structures which plays a great role in solute and fine sediment transport.

This chapter will present the two-dimensional numerical study of diffusion and dispersion processes, as well as sediment transport and deposition in compound channels with the hydrodynamic and morphological software *Telemac*, developed by EDF.

Conscious of the degrees of approximations proper to the use of 2-D models in com-

pound channels, tracer is injected in the numerical model under the same conditions as the experiments presented in chapter 5. A calibration of the 2-D dispersion coefficients as a function of a number of subsectional characteristics gives a *global* indication of the mixing intensities in the different regions of the compound cross section. Further, the particular role on solute transport of large scale turbulent structures at the interface of the main channel with floodplains is assessed. Vortices with vertical axis appear explicitly in simulations due to numerical instabilities, using a Sub-Depth-Scale approach first introduced by Nadaoka & Yagi (1998) and applied to compound channels by Bousmar (2002). The *global* turbulent diffusion is compared to those numerical simulations where turbulence modelling concerns much smaller scales and the role of these turbulent eddies in solute transport becomes more evident.

Telemac-2D is coupled to its sediment transport module *Sisyphe*. Focus is brought to sediment transport by suspension and lateral transfer towards the floodplains. Previous conclusions on lateral solute transport were helpful to control suspension transport. Numerical simulations aiming at reproducing the observations of the sediment transport and deposition experimental campaign reported in Chapter 5 identifies the parameters that condition deposition formation and deposits morphology.

In practice, the finite-element model *Telemac-2D* solves the depth-averaged continuity and Navier-Stokes equations already presented in this work. A two-dimensional area is described by a set of meshes (triangular and unstructured), whose nodes are associated with a ground level value. Different hydraulic parameters are then defined, including the friction coefficient, which can be defined area by area and then enables us to represent the different runoff zones in the computational domain. *Telemac-2D* solves the shallow water equations at each node of the model. The following section presents the set of equations solved by the software, which are particular forms of the equations presented in Chapter 3.

6.2 Model description

6.2.1 Hydrodynamic module

The finite-element model *Telemac-2D* solves the depth-averaged continuity equation (3.19) and momentum equations (3.20), or Shallow Water equations, with the term $\partial U_j / \partial x_i$ neglected in the diffusion term. This greatly simplifies the equations since no direct coupling

exists between the different velocity components. Experience has shown that this coupling can generally be neglected.

Conservative form

We eventually get the following set of equations (Hervouet, 2003):

Continuity

$$\frac{\partial h}{\partial t} + \text{div}(\mathbf{U}h) = 0 \quad (6.1)$$

Momentum

$$\frac{\partial U h}{\partial t} + \text{div}(\mathbf{U}U h) = -h g \frac{\partial z_w}{\partial x} + h F_x + \text{div}(h \nu_t \nabla U) \quad (6.2a)$$

$$\frac{\partial V h}{\partial t} + \text{div}(\mathbf{U}V h) = -h g \frac{\partial z_w}{\partial y} + h F_y + \text{div}(h \nu_t \nabla V) \quad (6.2b)$$

Here, ν_t is a general diffusion coefficient which takes into account molecular viscosity, dispersion and turbulence. F_x and F_y stand for depth-averaged source terms defined earlier in equation (3.20). The above equations are said to be in *conservative form*. Only a rigorous application of conservative equations will lead to a good grasp of the problem considering the discontinuities that can arise in the solution (e.g. hydraulic jumps). Numerical schemes will increase in complexity when these discontinuities are to be treated, and explains why a different formulation of these equations is adopted, called *non-conservative*.

Non-conservative form

By developing the derivatives in the conservative equations, those equations are obtained:

Continuity

$$\frac{\partial h}{\partial t} + \mathbf{U} \cdot \nabla h + h \text{div}(\mathbf{U}) = 0 \quad (6.3)$$

Momentum

$$\frac{\partial U}{\partial t} + \mathbf{U} \cdot \nabla U = -g \frac{\partial z_w}{\partial x} + F_x + \frac{1}{h} \text{div}(h \nu_t \nabla U) \quad (6.4a)$$

$$\frac{\partial V}{\partial t} + \mathbf{U} \cdot \nabla V = -g \frac{\partial z_w}{\partial y} + F_y + \frac{1}{h} \text{div}(h \nu_t \nabla V) \quad (6.4b)$$

Solute transport

Tracer transport is also modelled using *Telemac-2D* which solves the following equation, together with the previous one:

$$\frac{\partial C}{\partial t} + \mathbf{U} \cdot \nabla C = S_c + \frac{1}{h} \operatorname{div} (h \nu_T \nabla C) \quad (6.5)$$

with ν_T (m^2/s) the turbulent diffusion coefficient for the tracer, S_c ($g/l/s$) the depth-averaged source of tracer and C (g/l) the depth-averaged tracer concentration. This tracer transport equation is the depth-averaged version of equation (3.11) with isotropic diffusion and possibility of imposed tracer sources. The turbulent diffusivity ν_T is either computed using a transport model for turbulent quantities, or specified by the user, as it is done in this study for enhanced control. This mixing coefficient represents transport by turbulent fluctuations, by convective dispersion and by secondary currents (transverse and vertical velocity components).

Discretization of the system

A finite element discretization is employed in *Telemac-2D* to solve the shallow water equations. The fundamental operation in Finite Elements is the variational formulation (see e.g. Hirsch, 1988). The SUPG advection scheme (Brookes & Hughes, 1982) was used for continuity and momentum equations. The k - ϵ turbulence equations are solved by the method of characteristics coupled with a fractional step method (Hervouet, 1986). A detailed description of the solution is provided by Hervouet & Van Haren (1996).

6.2.2 Sediment transport module

The *Telemac* system contains a sediment transport module called *Sisyphe*. This module contains a morphodynamic model which computes bed load transport and/or suspension transport with the subsequent bed evolutions (3 unknowns). In this work, *Sisyphe* used the bi-dimensional hydrodynamic fields calculated with *Telemac-2D* as input in the transport equations. Those two modules may work in a coupled way, with updates in depth-averaged velocities U, V and water depths h following the bottom evolutions at every time steps. Optional non-erodible zones are well represented, which was mandatory to reproduce the experimental conditions presented earlier.

Calculation procedure

Sisyphé solves the bed evolution equation (3.53) after the computation of bed load and the concentration of suspended sediment. Bed load is calculated using empirical formulae. In case of non-equilibrium transport, suspended sediment concentration is deduced from the depth-averaged version of the transport equation (3.37). Erosion and deposition fluxes are computed using the empirical formulae developed in section 3.5.2.

Bed load and suspended load

The two-dimensional total load is defined as:

$$\mathbf{Q}_t = \mathbf{Q}_s + \mathbf{Q}_b \quad (6.6)$$

where $\mathbf{Q}_s \equiv (Q_{sx}, Q_{sy})$ is the 2-D suspended load vector and $\mathbf{Q}_b \equiv (Q_{bx}, Q_{by})$ is the 2-D bed load vector.

As explained in section 3.5.1, bed load occurs in a very thin high concentrated near-bed layer. The thickness $a = \max(2d_{50}, k_s)$ (in *Sisyphé*) of this layer depends on the size of the grains or the bottom roughness. Many empirical laws are left at the user's choice to compute this load, but they all suppose that local equilibrium conditions are considered to be generally valid. The formulas of Meyer-Peter (1948), Einstein-Brown (Einstein, 1942 and Brown, 1950) or van Rijn (1984a) are all available choices but concern different sediment size ranges and may provide very disparate previsions.

Suspended sediment transport is associated to a transport-diffusion phenomenon and concentration estimations are provided by solving a depth-averaged version of the transport equation (3.37), where a source term due to erosion and/or deposition is added:

$$\frac{\partial C}{\partial t} + U \frac{\partial C}{\partial x} + V \frac{\partial C}{\partial y} = \frac{1}{h} \left[\frac{\partial}{\partial x} \left(h \varepsilon_s \frac{\partial C}{\partial x} \right) + \frac{\partial}{\partial y} \left(h \varepsilon_s \frac{\partial C}{\partial y} \right) \right] + \frac{(E - D)_{z=a}}{h} \quad (6.7)$$

where C is the depth-averaged sediment concentration and ε_s is the sediment diffusivity coefficient. $(E - D)_{z=a}$ is the net erosion minus deposition flux of sediment, which is calculated at the interface between the suspended-load layer and the bed-load layer. z is the vertical direction and its local origin is situated on the bed.

Bed evolution

The bed evolution equation may be used in three different ways following the choice of modelling with suspended sediments (Sergent & Zhang, 2002):

1. *Bed load only*

In situation of equilibrium, the erosion equals the deposition and we have:

$$(1 - n) \frac{\partial z_b}{\partial t} + \text{div}(\mathbf{Q}_s) = 0 \quad (6.8)$$

with z_b the bottom level from from data and n the bed porosity. This equation assumes that the sand concentration has reached its equilibrium concentration and corresponds to saturation conditions. It is thus only valid for bed-load transport, but can be extended to *total load* assuming local equilibrium conditions. Engelund-Hansen's formula (3.71) is implemented in *Sisyphe* and can be used to compute the bed evolution using Eq. (6.8) but with $\mathbf{Q}_s = \mathbf{Q}_t$.

2. *Suspended load only*

The bottom level is then computed with:

$$(1 - n) \frac{\partial z_b}{\partial t} + (E - D)_{z=a} = 0 \quad (6.9)$$

the bed change is attributed to the net sediment flow at the lower boundary of the suspended-load zone (Wu, 2008).

3. *Combined bed load and suspended load*

In order to represent non-equilibrium conditions, the bed-evolution equation has to be coupled to a transport-diffusion equation for the suspended sand concentration and the source term $S = (E - D)_{z=a}$ is added to the equation:

$$(1 - n) \frac{\partial z_b}{\partial t} + \text{div}(\mathbf{Q}_s) + (E - D)_{z=a} = 0 \quad (6.10)$$

This separation of the water column into two zones is frequently used in morphodynamic models and, if bed load and suspended load behave very differently, appropriate interaction mechanisms exist between the two layers (Fig. 6.1).

$$\frac{\partial C}{\partial t} + U \frac{\partial C}{\partial x} + V \frac{\partial C}{\partial y} = \frac{1}{h} \left[\frac{\partial}{\partial x} \left(h \epsilon_s \frac{\partial C}{\partial x} \right) + \frac{\partial}{\partial y} \left(h \epsilon_s \frac{\partial C}{\partial y} \right) \right] + \frac{(E - D)_{z=a}}{h}$$

$$(1-n) \frac{\partial Z_b}{\partial t} + \text{div}(\mathbf{Q}_s) + (E - D)_{z=a} = 0$$

Figure 6.1: *Interaction between suspended load layer and bed load layer*

Erosion and deposition terms

The formulation from Celik & Rodi (1998) for erosion and deposition fluxes is used as defined in Eq. (3.62). The deposition rate is known from the local concentration at the interface and settling velocity while the entrainment is assumed to occur at the same rate as it does under equilibrium conditions. The equilibrium concentration c_a^* is given by the formula of Zyserman & Fredsoe (1994) of Eq. (3.63) while the reference concentration is given by (3.67).

Non-erodible zones

Sisyphe provides the possibility to model rigid beds where conceptually no erosion is possible. The effect of rigid beds is either to reduce the sand transport rate (deposition), or to let the incoming sand to pass over the bed, which becomes tricky numerically. Two different approaches are implemented in the software. The first possibility, looking at Eq. (6.8), is to reduce the sand transport rate when the rigid bed is attained. This method may however induce erosion underneath the rigid bed (see Hervouet, 2003 and Hervouet *et al.*, 2003). The other approach prescribes to find a limitation of the bed load transport which ensures a limitation of its divergence so that the rigid bed is not affected (Hervouet & Villaret, 2004).

Sisyphé's limitations

As for all numerical model, inherent accuracy limitations are present due to the model's hypothesis and its physical principles. A non-exhaustive list is given here (Villaret, 2004):

- the model is bi-dimensional, which induces a depth-averaged computation of sediment concentration with the transport-diffusion equation (6.7). Examining the possible vertical distributions of concentrations in Fig. 3.4, this can be a serious approximation.
- the accuracy of empirical sand transport formulas
- the scattered values of solid flow rates as obtained from the transport formulas
- the sensitivity of sand transport rate estimates to hydrodynamic variables including current velocity and friction coefficient
- the empiricism in the erosion-deposition fluxes calculations for the suspended load calculations.

6.3 Large scale turbulent structures acting on solute transport

The presence of strong turbulent features at the interface between a compound channel's subsections enhances mixing in this region and lateral solute transfer towards the floodplains by the same occasion, as explained by Guymer *et al.* (1998). The respective role of turbulence and secondary currents on transport rate is rather complex. The particular role of interfacial vertical axis eddies, observed experimentally, is investigated. Preliminary results of this numerical study are reported in Fraselle *et al.* (2010a).

A global indication of the mixing intensities in the different regions of the compound cross section is deduced from a calibration of the diffusion coefficient ν_T in *Telemac-2D* simulations, recalling the experimental injection data presented in Chapter 5. *Telemac-2D* is then used to investigate the effect on time-averaged tracer concentrations of vertical axis eddies. Injections of tracer in the whole main channel is done in a stable hydrodynamic field and in a field where numerical instabilities are allowed to grow and where those large-scale horizontal structures appear explicitly. To do so, a coherent structure capturing approach (Nadaoka & Yagi, 1998) is used, already applied to the shear layer in a compound channel by Bousmar (2002).

Forced stable flow conditions are modelled with a constant flow rate Q imposed at the inlet and water elevation h imposed at outlet. Periodic boundary conditions are also used where the outlet velocities are reimposed upstream, and upstream water levels reimposed downstream. The upstream boundary becomes an "image" of the downstream boundary, and vice-versa. Rounding errors in the computational procedure are then free to produce flow instabilities which create hydrodynamic oscillating conditions and model explicitly interfacial vertical axis eddies (Bousmar & Zech, 2003), as depicted later on Fig. 6.7. The question of the physical meaning of those numerical instabilities while solving the shallow water equations is not straight-forward, as depth-averaged RANS equations are expected to model mean flow behaviour. This issue urges the reader to great care during the analysis of the results.

6.3.1 Experimental reference data

Tracer injection data in the UCL compound channel flume presented in Chapter 5 are used again. The discharge was $Q = 18.55 \text{ l/s}$ and the bottom slope $S_0 = 1/1000$ (Fig. 6.2a). Results mainly show a very fast vertical mixing and an expansion of solute more developed towards the floodplains than towards the main channel when injected in the shear layer. Over the floodplains, the diffusion coefficient fitting can only reproduce the strong migration towards the floodplains by a relatively high lateral diffusion coefficient associated with floodplains since secondary advection currents are not taken into consideration. The values of those mixing coefficients are summarized in Table 6.1.

The UCL compound channel flume was also used with a slight geometry change (Fig. 6.2b) and a bed slope $S_0 = 1.9/1000$ to visualize large scale turbulent structures at the interface between the main channel and the floodplains, as done by Bousmar (2002). Fig. 6.3 presents a seeded flow in Lagrangian view (moving frame at the vortices celerity $= 0.314 \text{ m/s}$) which clearly identifies those vertical axis eddies and justifies the interest of this study.

6.3.2 Results and discussion

Hydrodynamics and numerically induced vortices

Telemac-2D hydrodynamic simulations were confronted to experimental data for each geometry tested. Fig. 6.4 presents the experimental velocities measured with a pitot

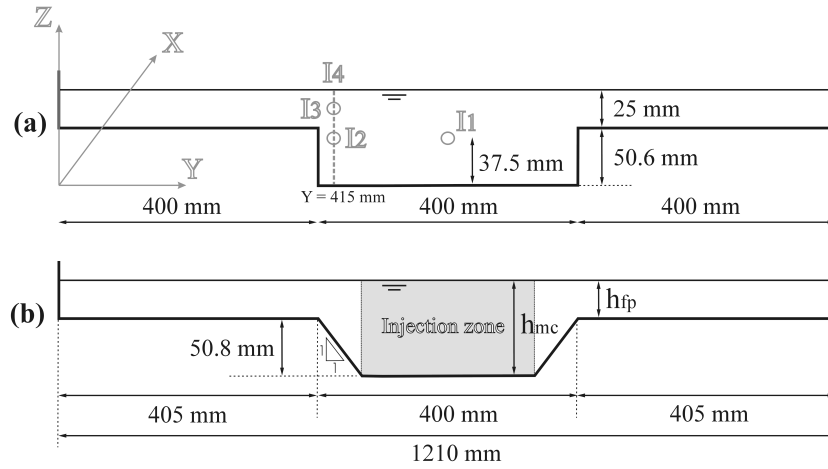


Figure 6.2: Compound channel cross sections with (a) geometry with $S_0 = 1/1000$ and locations of I1, I2, I3 (point injections) and I4 (line injection); (b) geometry with $S_0 = 1.9/1000$ and slight dimension modifications showing tracer injection zone for Telemac 2D simulations.

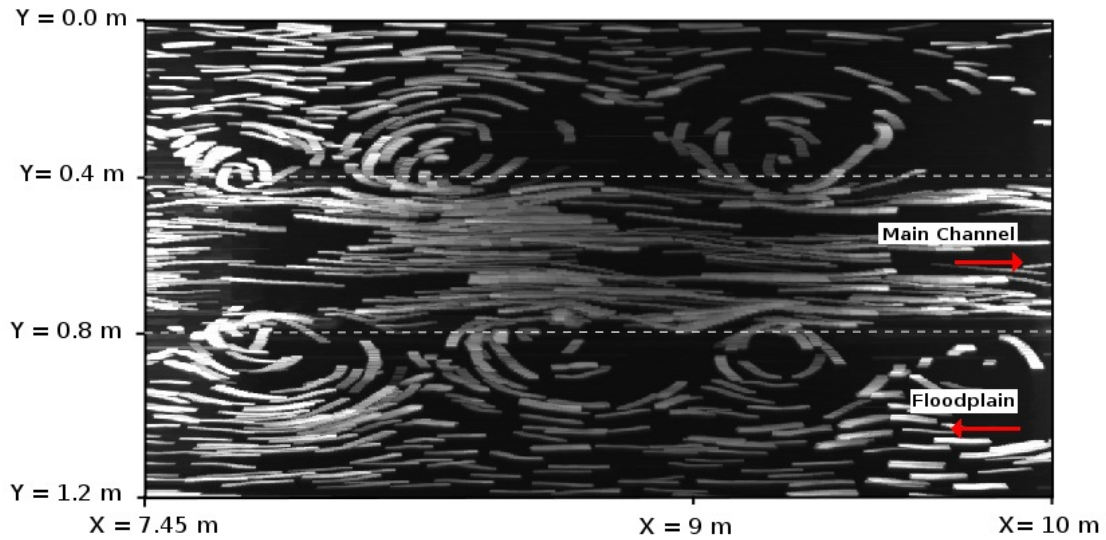


Figure 6.3: Vertical axis eddies appearing in downstream moving frame (0.314 m/s celerity) with $1.9/1000$ slope, 15 l/s (UCL compound channel flume, geometry B from Fig. 6.2b). Images are taken at 15 Hz frequency with MARLIN (F-131 B AVT $2/3''$ CMOS) cameras and floating EPS tracers. A 1 second sequence is taken to produce the figure. The oscillation period of those vortices is approximately 2-3 s.

tube in the geometry A (Fig. 6.2a), together with *Telemac-2D* numerical results using either a $k-\epsilon$ model to model turbulence or an Elder model (Eq. (3.29)). To match the model's limitation, the vertical step forming the main channel's bank is modelled as a 70° angle lateral slope. However, good correspondence is observed which suggests that the advective field applied to tracer transport is quite realistic.

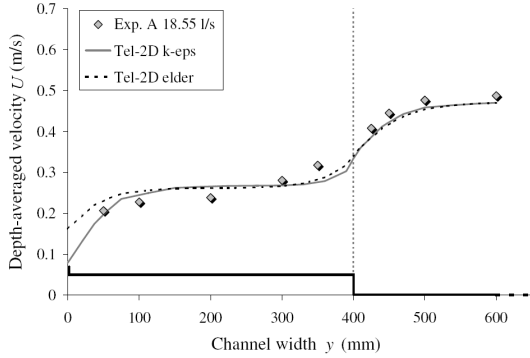


Figure 6.4: Longitudinal depth-averaged velocities U (m/s) for $Q = 18.55$ l/s in Geometry A (Fig. 6.2a) (a) experimental, (b) *Telemac-2D* with $k-\epsilon$ turbulence modelling and (c) *Telemac-2D* with Elder turbulence modelling.

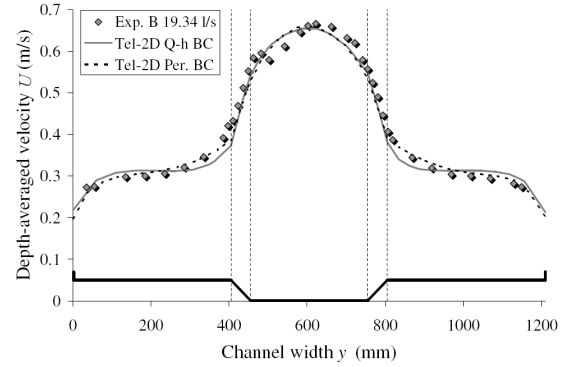


Figure 6.5: Longitudinal depth-averaged velocities U (m/s) for $Q = 19.34$ l/s in Geometry B (Fig. 6.2b) (a) experimental, (b) *Telemac-2D* with stable velocity field and (c) *Telemac-2D* with oscillating velocity field (time-averaged).

On the other side, flow in geometry B (Fig. 6.2b) was modelled on a 2-D numerical mesh (3-5 cm triangular elements), presented on Fig. 6.6, imposing $Q = 19,34$ l/s and either $Q-h$ boundary conditions or periodic boundary conditions. Time-averaged velocity profiles are presented on Fig. 6.5 using an Elder model. Periodic boundary conditions produce an instantaneous velocity field where oscillating conditions are recreated. Unlike the stable conditions forcing through $Q-h$ boundary conditions, numerical instabilities due to rounding errors are allowed to develop until stabilization with periodic boundary conditions. Large-scale turbulent structures, observed experimentally (Fig. 6.3) appear thus explicitly and are presented on Fig. 6.7, in a lagrangian velocity field ($U - 0.393$ m/s where the 0.393 m/s is the mean velocity at the interface). The oscillation period of the vortices ($\cong 4.17$ s) was extracted through a Fast Fourier Transform applied on the transverse velocities at the interface between the main channel and the floodplains (Fig. 6.8). The difference observed with the oscillation period of experimental vortices might indicate the lack of physical meaning of those numerical instabilities produced through an Unsteady depth-averaged RANS simulation (U-RANS), as performed in this study.

Hinterberger *et al.* (2007) explain how the complex situation with 2-D and 3-D turbulent structures is difficult to handle by solving the Reynolds-averaged Navier-Stokes (RANS) equations, which involve statistical models for the entire turbulence. Unlike DA-LES (Depth-Averaged Large-Eddy Simulation), which is in concept better suited than the RANS method, 2-D large-scale horizontal motions are not now called turbulence but are considered as unsteady mean flow. This difference is not essential since only a qualitative effect of those vortices on tracer transport is examined in this study.

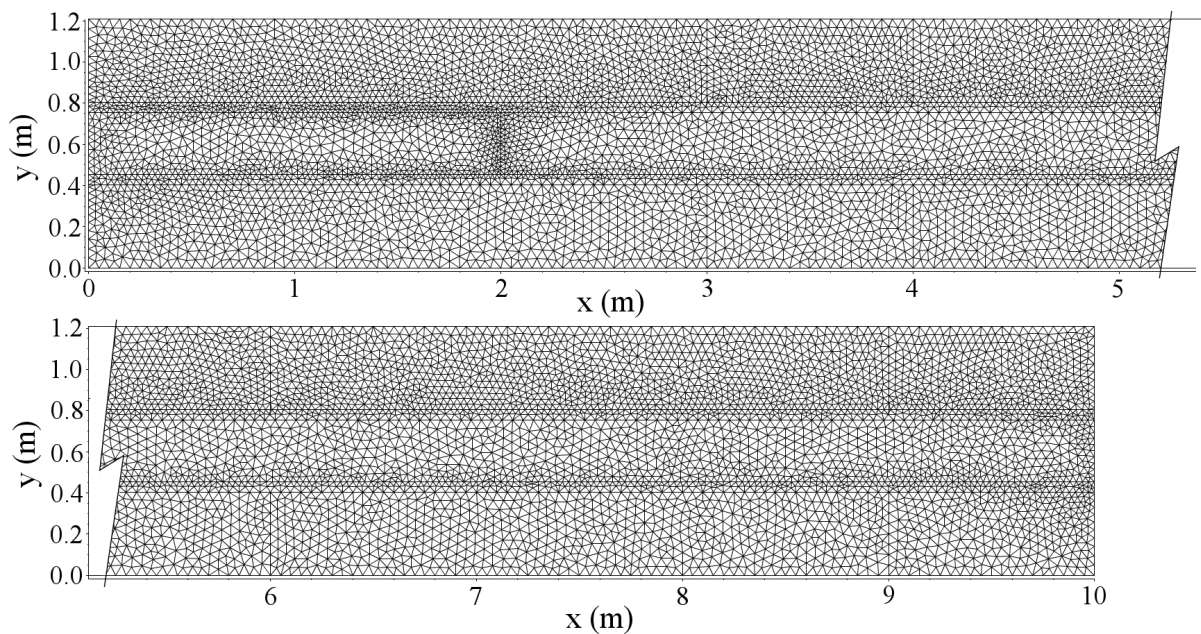


Figure 6.6: *Telemac 2D numerical mesh with triangular elements (3-5 cm) for geometry B. The mesh is densified at $x = 2$ m for tracer injection in the main channel.*

Transverse mixing coefficient calibration

Before studying the effect of interface vortices on tracer transport, correct calibration of the ν_T coefficient used in *Telemac-2D* simulations (Eq. (6.5)) is requested. This coefficient determines the numerical lateral mixing intensity and it was chosen to produce depth-averaged concentrations which fit the experimental data presented earlier in this work. Fig. 6.9 shows the depth-averaged concentrations for I1 injection in the centre of the main channel, where the Finite Difference model refers to the model developed in Chapter 5. A different value was chosen over the floodplains and Fig. 6.10 presents the depth-averaged concentrations for I4 injection in the shear layer while the corresponding lateral mixing

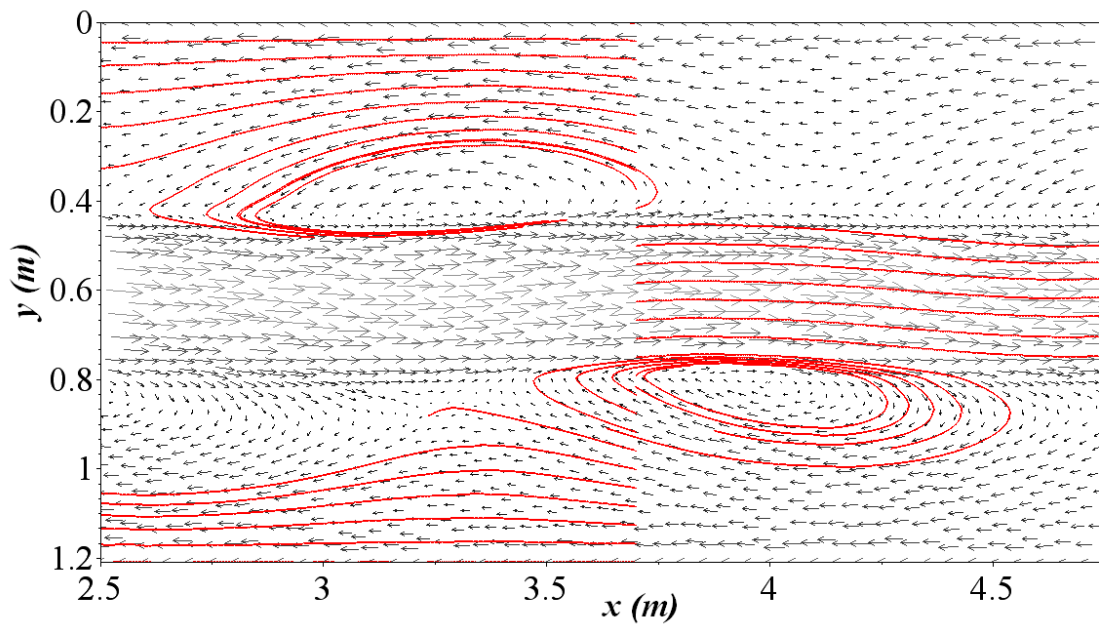


Figure 6.7: Stream lines in lagrangian velocity field ($U=0.393$ m/s) for initial segment at $x = 3.7$ m and $T = 5$ s with 20 s integration time. Telemac Simulation in geometry B.

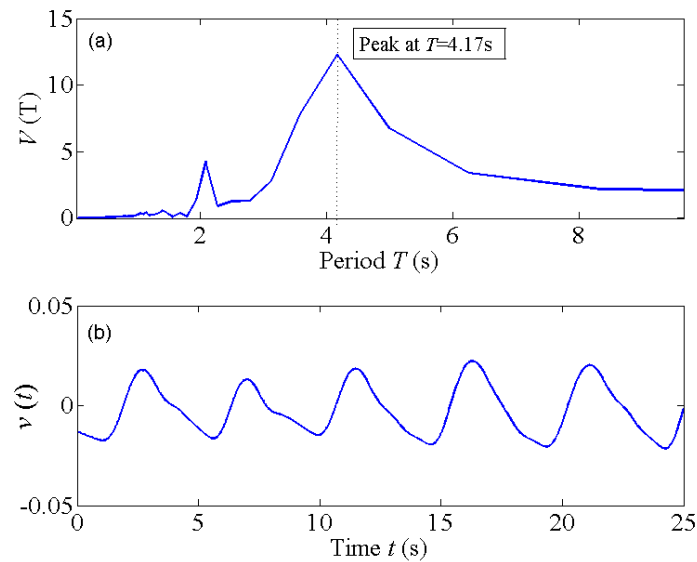


Figure 6.8: (b) Transverse velocity variations at main channel-floodplain interface ($x = 5$ m and $y = 0.805$ m) and (a) corresponding FFT of the time signal. Peak shows an oscillation period for the vortices of 4.17 s

parameters for the different numerical models used are presented in Table 6.1.

Table 6.1: *Estimated mixing coefficients in the main channel and over the floodplains*

Coefficient	$D_{y,mc} / \nu_{T,mc}$	$D_{y,fp} / \nu_{T,fp}$
Simplified 3D transport*	$\cong 0.18 h_{mc} u_{*,mc}$	$\cong 1.7 h_{fp} u_{*,fp}$
Tel. 2D with A geometry* ²	$0.135 (h_{mc} u_{*,mc})_A = 2.8 \cdot 10^{-4}$	$0.65 (h_{fp} u_{*,fp})_A = 2.6 \cdot 10^{-4}$
Tel. 2D with B geometry* ³	$0.135 (h_{mc} u_{*,mc})_B = 3.27 \cdot 10^{-4}$	$0.65 (h_{fp} u_{*,fp})_B = 2.13 \cdot 10^{-4}$

* see Chapter 5

*² see Fig. 6.9 and 6.10

*³ Scaling of coefficients calibrated in geometry A

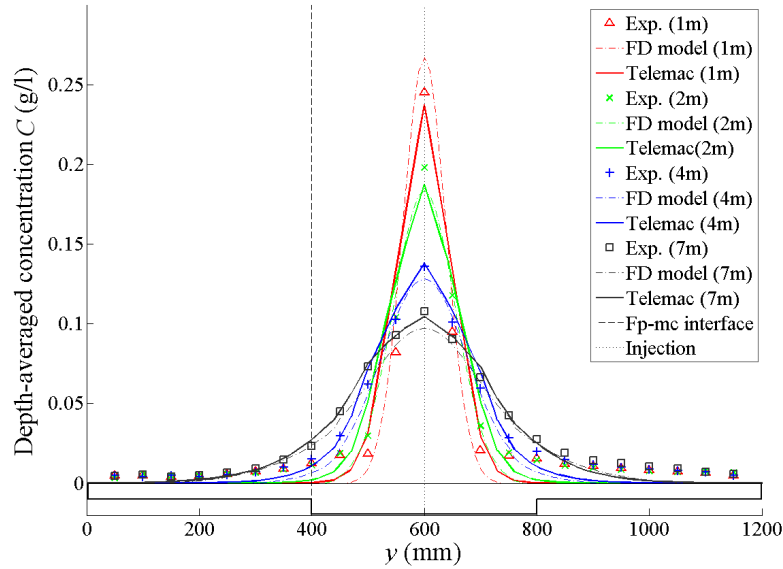


Figure 6.9: *Experimental and numerical depth-averaged concentrations C (g/l) when injection of tracer in the centre of the main channel (I1 injection case).*

The lateral mixing coefficients are taken proportional to the product of the shear velocity u_* and average depth h in each subsection. A duo of constant values is a valid simplification (Arnold *et al.*, 1985) but Guymer *et al.* (1998) suggested a more complex profile for this mixing coefficient where the shear layer presents a localized increase (see Fig. 6.14). Calibration of *Telemac-2D* mixing coefficients in geometry B consisted in a scaling of coefficients calibrated in geometry A. The difference ($> 100\%$) in the non-dimensional

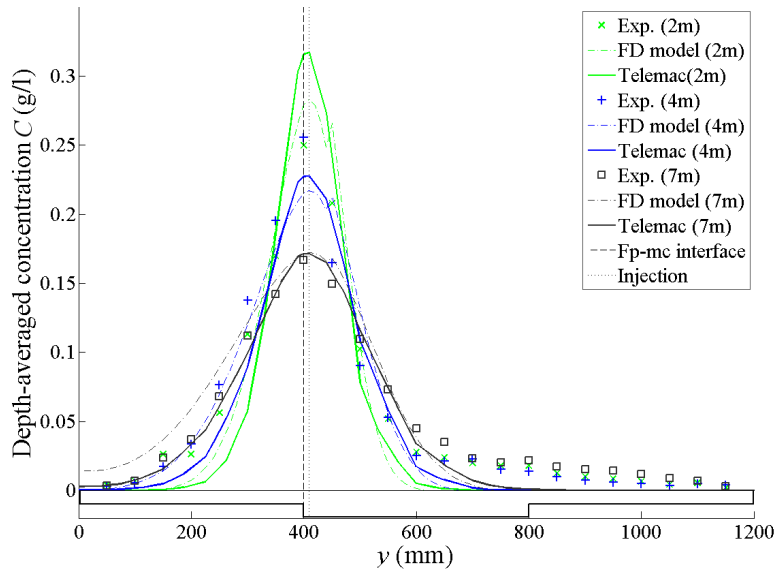


Figure 6.10: *Experimental and numerical depth-averaged concentrations C (g/l) when injection of tracer at the interface (I_4 injection case).*

mixing coefficients between the finite difference model and *Telemac-2D* finds many origins. It could probably come from an increased numerical diffusion in the latter due to a coarser mesh (3-5 cm mesh elements against 1 cm mesh elements). The different numerical schemes between the models could also influence the results. Further, the vertical edge at the interface could act as a concentration trap, which is not properly represented in the two-dimensional simulations. Indeed, vertical steps are modelled as relatively steep slopes. Anyway, this difficulty in estimating the lateral mixing coefficients is present in all related published studies, and will be discussed later. A visual calibration is requested for such complex process and correct estimation for near-field diffusion might lead to an over-estimation in the far-field, as for the floodplain concentrations at a 7 m distance from the I_4 source using the finite difference model (Fig. 6.10).

Vertical axis eddies and enhanced mixing

Tracer injections ($m = 1$ g/s) on the whole width of the main channel are performed with *Telemac-2D* in geometry B (Fig. 6.2b) with stable velocity field and oscillating velocity field. Fig. 6.11 gives the time evolution of tracer injected in an oscillating velocity field created through periodic boundary conditions. The concentration C is plotted together with

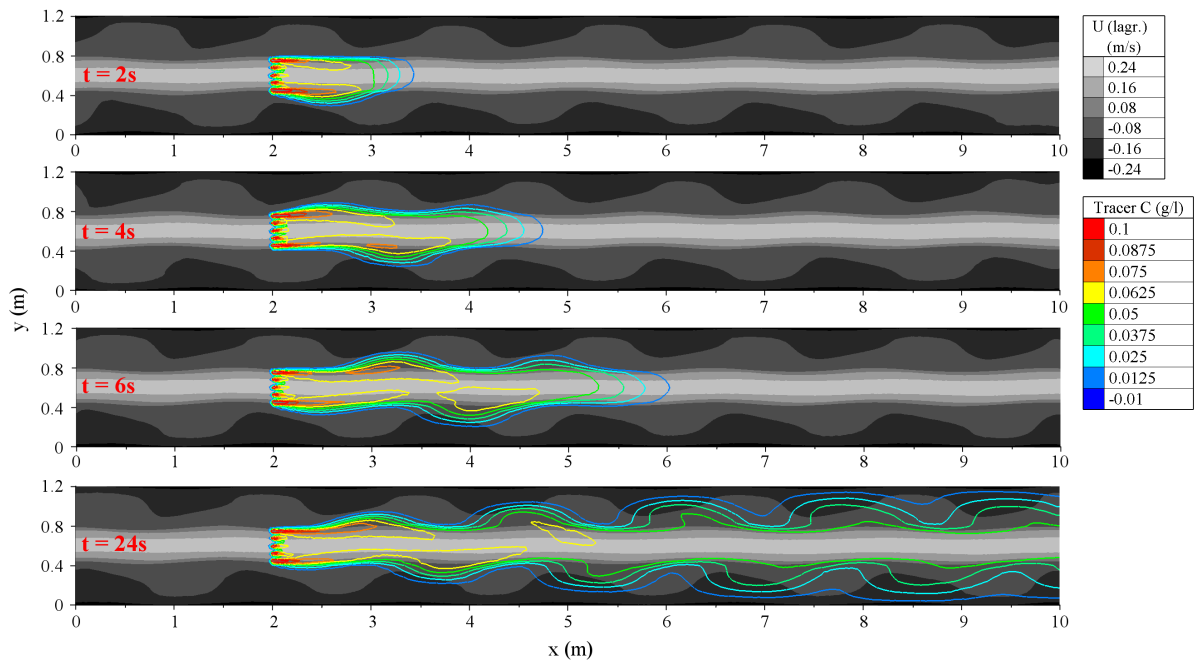


Figure 6.11: Time evolution of tracer concentration C (g/l) injected in oscillating conditions superposed to lagrangian velocity field (m/s). Telemac simulations with periodic boundary conditions and geometry B.

the instantaneous lagrangian velocity field with the reference velocity taken as the mean velocity at the interface between subsections. The presence of vortices clearly influences the tracer plume. Likewise, Fig. 6.12 shows the developed plume in an instantaneous lagrangian velocity vector field .

To make this influence even more obvious, time-averaged concentration profiles for injection in oscillating velocity field were compared to concentrations in case of injection in stable velocity field where no interface vortices are explicitly modelled. Fig. 6.13 shows those concentration profiles where concentrations for injection in oscillating field are time-averaged over 3 complete oscillation periods ($\approx 12s$) and proves the enhanced mixing due to the effect on the tracer plume of those interface vortices, especially in the shear zone near to the interface where strong vorticity is present. The transverse mixing coefficients were the one used in Table 6.1 for geometry B and Fig. 6.13 makes clear that a more complex combination of mixing coefficients would be required to model the effect of interface turbulence.

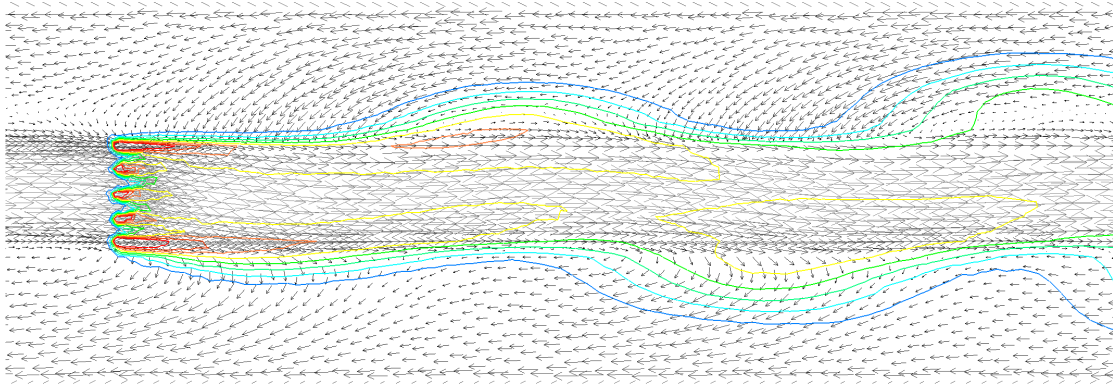


Figure 6.12: *Instantaneous tracer concentration C (g/l) contours injected in oscillating conditions superposed to lagrangian velocity vector field (m/s).*

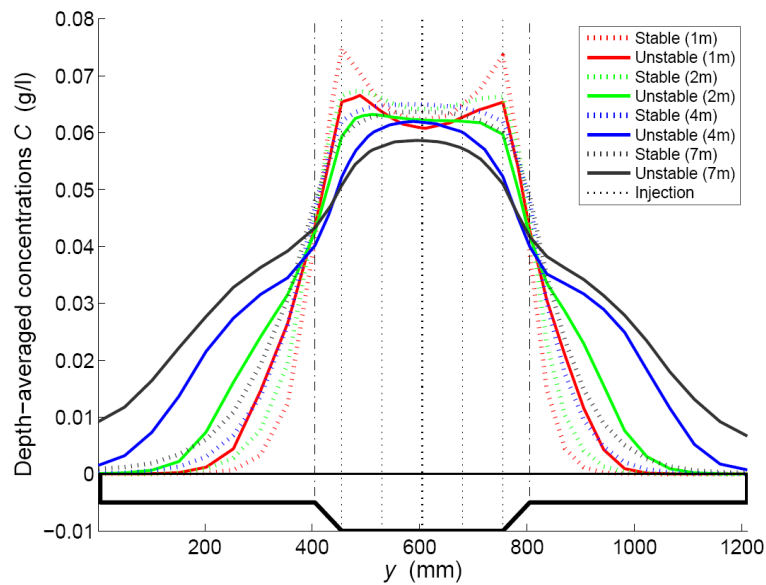


Figure 6.13: *Depth-averaged concentrations C (g/l) for tracer injection in B geometry at different distances from injection location (1, 2, 4 and 7 m from the source). Results are shown for injection in stable velocity field and injection in oscillating field were concentrations are time-averaged over 3 oscillation periods ($\approx 12s$).*

Discussion and perspectives

It is tempting to determine the precise effect of this region with high vorticity on the mixing properties of the flow. To give an order of magnitude of the localized increase in mixing at the interface, a lateral distribution of the ν_T coefficient is chosen following Guymer *et al.* (1998) and presented on Fig. 6.14. Tracer is injected in a stable velocity field with this spatially variable mixing distribution. Fig. 6.15 shows that a $1.5 \cdot 10^{-3}$ step is requested to produce a tracer plume in stable conditions which follows the same trend as the time-averaged tracer plume in simulations where large-scale structures are explicitly modelled.

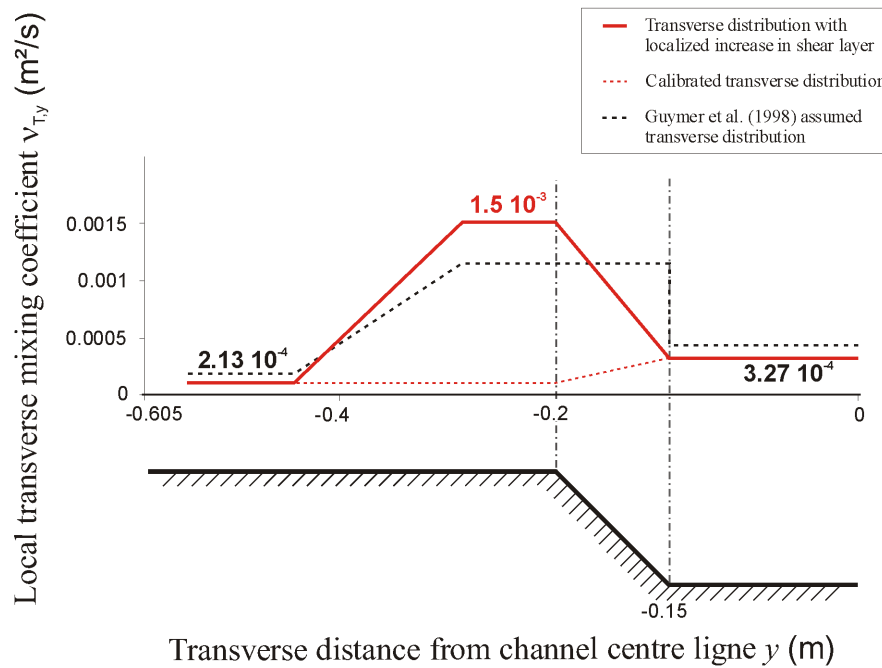


Figure 6.14: Assumed transverse distribution of lateral mixing coefficient $\nu_{T,y}$ (m^2/s).
 $Q = 19.34 \text{ l/s}$, $h_r = 0.265$ and $S_0 = 1.9/1000$.

If the influence of those vortices on mass transfer rate is now proven, together with an approximative quantification of their effect, much work is still requested to isolate their action from other turbulent phenomena. A numerical experiment was led in geometry A with $h_r = 0.33$ where the precise effect on tracer of the overall turbulent state is known (see Chapter 5). Fig. 6.17 indicates that large scale turbulent structures are much less developed than for B geometry (where $h_r = 0.265$). Fig. 6.16 shows how tracer injection in oscillating velocity field can't explain the experimentally observed diffusion and pushes

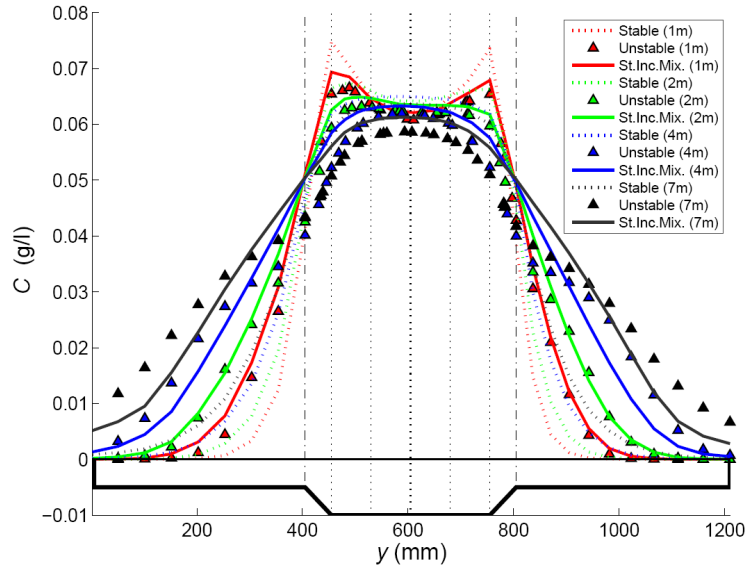


Figure 6.15: Depth-averaged concentrations C (g/l) in stable and oscillating field ($Q = 19.34$ l/s and $h_r = 0.265$). The last concentration profiles are obtained with injection in a stable velocity field with manually increased mixing in the shear layer.

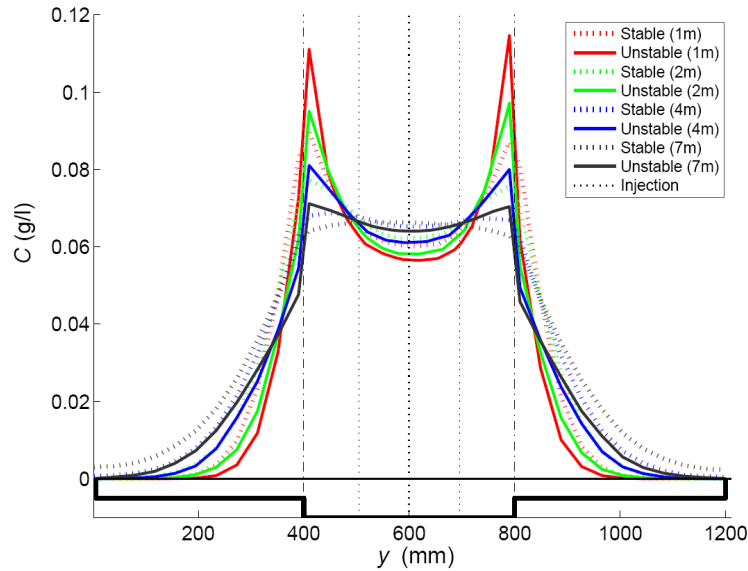


Figure 6.16: Depth-averaged concentrations C (g/l). Results are shown for injection in geometry A ($Q = 18.55$ l/s and $h_r = 0.33$) with stable velocity field and injection in oscillating field where concentrations are time-averaged and a 10^{-5} global diffusion coefficient.

to suspect the intervention of secondary circulation on mass transfer.

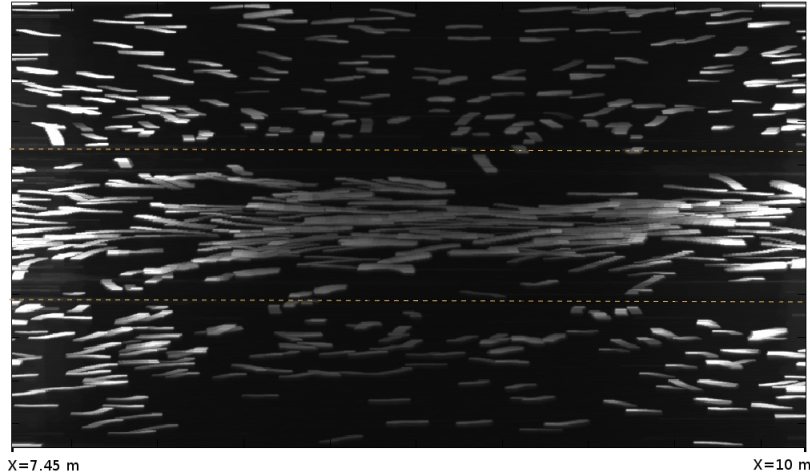


Figure 6.17: *Vertical axis eddies appearing in downstream moving frame (0.357 m/s celerity) with 1/1000 slope, 18.55 l/s (UCL compound channel flume, geometry A from Fig. 6.2a).*

The difference in relative depth between A geometry and B geometry explains partially the difference in turbulence intensity, especially concerning the two-dimensional large scale structures (see Fig. 6.3 and 6.17). Since mixing coefficients are calibrated according to a specific turbulent state, a simple scaling of those coefficients to model tracer transport in different geometries is probably not sufficient. Tracer injections in all possible study cases could of course provide empirical relations for those mixing coefficients as a function of the geometry ($h_r, S_0, \dots$) and the flow (Q). However, apart from the limitations linked to a two-dimensional analysis where many three-dimensional structures are not modelled explicitly, this study shows the path towards a numerical study of the influence of specific turbulent features on mass transfer.

As already noticed during the calibration of the transverse mixing coefficient, much uncertainties remain concerning this essential parameter. This disparity in the estimation of lateral mixing is also manifest in literature. Many authors agree on the proportionality with the local depth and shear velocity, applying the following relationship:

$$\nu_{T,y} = C_v \times u_* h \quad (6.11)$$

but they stay mitigated regarding the value to give to C_v . Wood & Liang (1989) found

$C_v = 0.135$ in a compound geometry with a relative depth $h_r = 0.5$. Djordjevic *et al.* (1989) suggested a $C_v = 0.2$ value in geometries where the relative depth was varied from 0.1 to 0.25 while Arnold *et al.* (1989) recommended C_v values in the range of 0.27 to 0.85. Authors recognized the necessity to vary this C_v coefficient over the floodplains, due to the presence of turbulent structures identified earlier. Prinos (1992) suggested an 0.35 value of the floodplains and an 0.25 in the main channel.

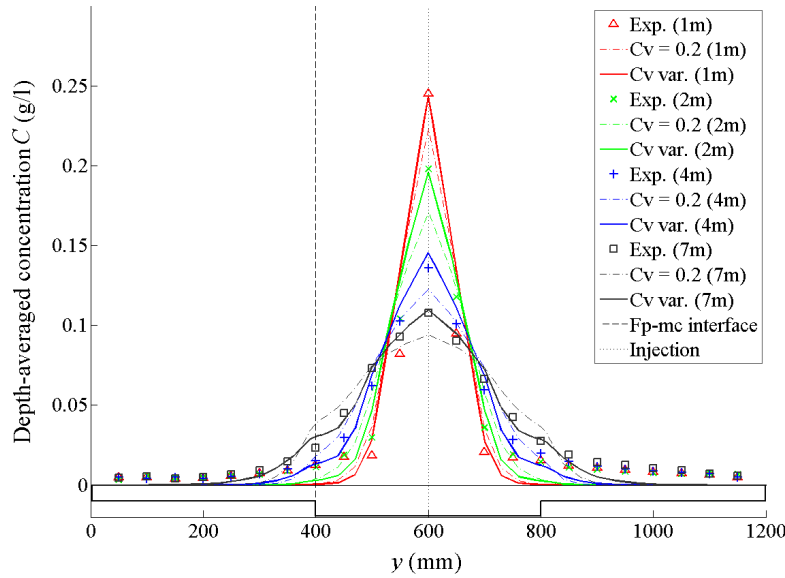


Figure 6.18: Experimental and numerical (Telemac) depth-averaged concentrations C (g/l) (I1 injection case). Mixing coefficients are chosen as $\nu_{T,y} = C_v \times u_* h$

The experimental data presented on Figures 6.9 and 6.10 was confronted to mixing coefficients using the (6.11) relationship. The 0.2 value suggested by Djordjevic *et al.* (1989) was used for C_v , together with a variable value over the cross section (Figure 6.20). Figure 6.20 presents a hypothetic lateral distribution of C_v which signifies a relative increase over the floodplains with a higher step in the main channel/floodplain interaction zone. $u_* h$ values are also presented for the A geometry. This distribution is a result of a trial and error process pursued to optimize the numerical predictions regarding the experimental concentrations. Figure 6.18 presents the depth-averaged concentrations for I1 injection and Figure 6.19, the concentrations for I4 injection. The constant value advocated by Djordjevic *et al.* (1989) fails to represent the effective mixing in this compound geometry while the distribution of Figure 6.20 provides much more reliable results.

The choice of a constant coefficient in each subsection has been made in this study, for

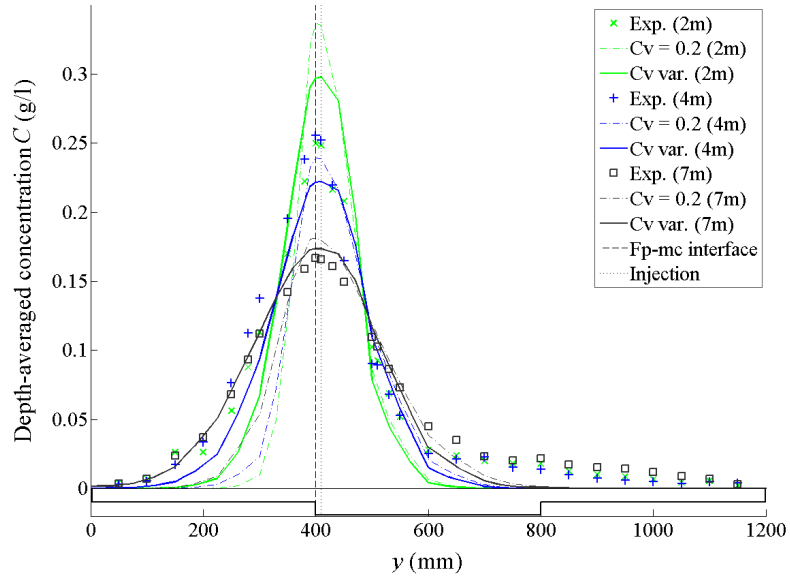


Figure 6.19: *Experimental and numerical (Telemac) depth-averaged concentrations $C(g/l)$ (I_4 injection case). Mixing coefficients are chosen as $\nu_{T,y} = C_v \times u_* h$*

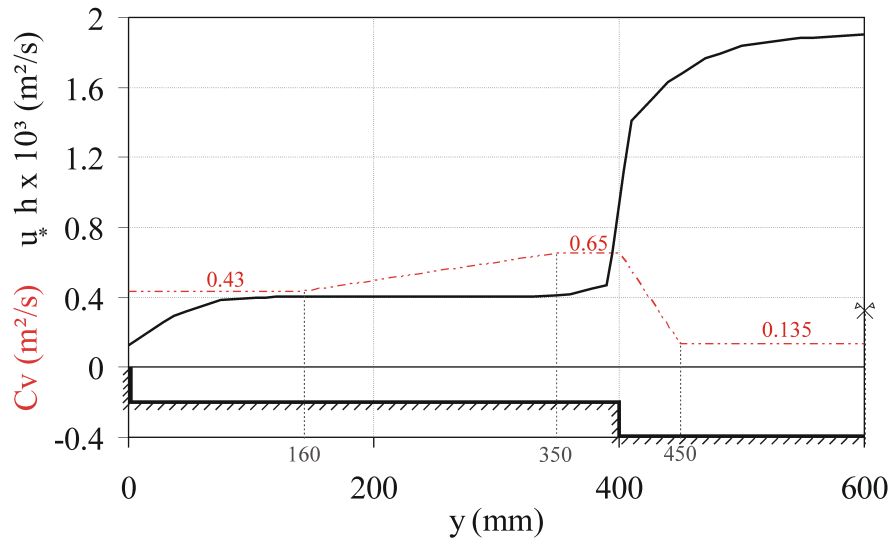


Figure 6.20: *Assumed transverse distribution of C_v and plot of $u_* h$ for $Q = 18.55$ l/s, $h_r = 0.33$ and $S_0 = 1.0/1000$ in geometry A.*

the sake of simplicity, but also because they provided correct concentration estimations regarding the experimental data. As discussed above, many other combinations are possible, and should be investigated in the future. To assess the effect of this lateral mixing coefficient on lateral mass transfer, the major concern of this section, different constant values of mixing coefficients were applied to a $m = 1 \text{ g/s}$ injection in geometry B (Figure 6.2b). A mixing coefficient of $\nu_T = 3 \times 10^{-4} \pm 50 \%$ was chosen and the percentage of mass flow rate transferred onto both floodplains is presented in Figure 6.21. This mass flow rate is computed numerically, through:

$$m_{tot} = \int_0^B ChU \, dy \quad \text{and} \quad m_{fp} = \int_0^{B_{fp}} ChU \, dy \quad (6.12)$$

with $m \text{ (g/s)}$ the tracer mass flow rate, C the concentration (g/m^3), h the water depth (m), U the longitudinal velocity (m/s), $B \text{ (m)}$ the width of the whole flume and $B_{fp} \text{ (m)}$ the width of the floodplain. Figure 6.21 also represents the numerical error on m compared to the injected mass along the flume, which stays fortunately lower than 3 %. The error on $m \text{ (%)}$ is computed by:

$$\text{error} = \frac{|m_{inj} - m_{tot}|}{m_{inj}} \times 100 \quad (6.13)$$

This sensitivity analysis shows that for a variation of $\pm 50 \%$ of the (constant) mixing coefficient, the effect on lateral transfer of mass flow rate is consequent ($>20 \%$). Such observation justifies future studies which aim at better describing this mixing coefficient.

6.4 Sediment transport and deposit study with *Telemac-2D*

6.4.1 Introduction

A sediment transport model has the task of providing, for a particular flow and particular situation, the distribution of suspended sediment concentration in the water body and the associated suspended load as well as the deposition or entrainment rates. Desirable features of a model are that it be widely applicable, but on the other hand be relatively simple and economic to use (Celik & Rodi, 1988). In general, sediment transport, like the associated flow, is a three-dimensional and time-dependent process. Depth-averaged models are able to deal with large transverse concentration variations, but rely on an

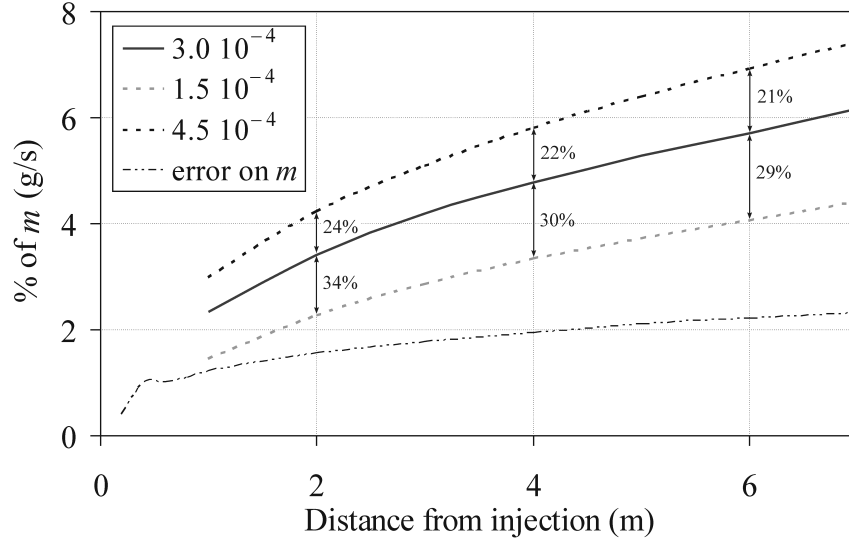


Figure 6.21: Percentage % of mass flow rate m (g/l) transferred onto the floodplains for $\nu_T = 3 \times 10^{-4} \pm 50\%$ in geometry B. Increased or decreased transfer on the floodplains is indicated as a percentage of the transfer for $\nu_T = 3 \times 10^{-4}$. The numerical error on m compared to the injected mass is also drawn along the flume.

idealized vertical distribution of sediment concentration. If such approach is chosen in this section, appendix B presents alternative simplified sediment transport models, also investigated in the framework of this thesis, to the interested reader.

On the heels of experimental observations of fine sand deposits on the floodplains, reported in Chapter 5, this section intends to feed the discussion through 2D-Horizontal numerical modelling of sand transport and floodplain deposition in compound channel flow. *Telemac-2D* is coupled to its sediment transport module *Sisyphe* to provide a depth-averaged hydrodynamic flow field (velocities, water depths) which evolves at each time step, given the updated bottom elevation due to erosion and deposition.

As for the experiments, two test cases are investigated. Fine sediment transport is modelled over a compound channel with rigid bed and upstream imposition of sediment concentration in the main channel (cross section visible in Figure 5.10b). Mobile bed experiments with recirculation are also simulated defining erodible and non-erodible zones properly while imposing an upstream sand concentration which corresponds to equilibrium main channel concentrations observed experimentally (Figure 5.10a). In both cases, the bottom topography of the experimental flume was precisely introduced in the model.

Modelled irregularities account for any transverse flow structures and helps to optimize the interpretation of deposition zones compared to the experiments.

The objective of this study is to impose at the inlet of the main channel the equilibrium transport capacity and observe convection/diffusion processes and deposition on the floodplains where the transport capacity is reduced. Deposition formation on the floodplains is a result of complex mechanisms, starting with the lateral transfer of fine materials from the main channel towards the floodplains. Solute is thus injected in similar flow conditions as for the experiments, using different combinations of mixing coefficients, to establish a relationship between lateral transfer and hydrodynamic steady-state conditions. When dealing with sediments, the influence of floodplain roughness and "trapping" ability on deposition rate is investigated, through the identification and calibration of the most relevant numerical parameters which condition the process. Further, preferential deposition zones are drawn and correlated to the hydrodynamic field.

As for all mathematical models, general simplification hypotheses are adopted, reducing the complexity together with the degrees of abstraction, at the expense of physical realism. Such hypothesis will be described next.

6.4.2 Modelling hypotheses

A 2-D-horizontal model will require much less computational resources than 3-D modelling, but will not be able to model explicitly secondary circulations and their effect on sediment. Large scale structures with vertical axis could be modelled using the approach described in the last section. Despite their proven role in lateral matter transfer, steady-state hydrodynamic conditions are adopted in this section. Even if depth-averaged concentrations constitute a major hypothesis in sediment transport modelling, *Telemac-2D* coupled with *Sisyphe* is a more powerful tool than simplified models, such as James' (1985) model (Appendix B), which is not capable to deal with complex geometries. The mathematical model contains several modelling hypotheses and intrinsic limitations (see section 6.2.2). In this work, further hypotheses are adopted, based on trial and error, together with logical choices regarding the experimental conditions described in Chapter 5.

One major choice is to run simulations with *Suspended load only*. The reasons for this are the following : (1) the inadequacy of bed load transport formulas regarding the size of the sediment used in this study (e.g. Chang, 1988) ; (2) the necessity to

take convection/diffusion phenomena into account to model lateral transfer towards the floodplains, discarding total load formulas such as Engelund-Hansen formula described in Eq. (3.74) which require to perform simulations in *Sisyphe* activating the optional *Bed load only*; (3) trial and error observations which proved the aberration of *Combined bed load and suspended load* simulations, as shown later on. The drawback of such an hypothesis is that mobile deposition on the floodplains are not modelled, as well as the very fine bed load layer over the mobile bed, both observed experimentally. Deposited sediment mass on a floodplain is computed with:

$$M_s = \int \int_{Fp} (z_{b,f} - z_{b,i}) (1 - n) \rho_s dS \quad (6.14)$$

and $z_{b,f}$ and $z_{b,i}$ (m) are the final and initial bed levels, respectively ; n is the bed porosity (set to 0.4) and $\rho_s = 2650 \text{ kg/m}^3$ is the volumetric mass of sediment.

As concluded previously, differential mixing intensities are expected in the main channel and over the floodplains. Sand diffusion coefficients are chosen accordingly. A manual imposition of mixing coefficients is adopted throughout the runs, with a constant value in each subsection using the calibrated non-dimensional mixing coefficients of Table 6.1 (0.65 over the floodplains and 0.135 in the main channel with a 5 cm transition zone in the main channel shear layer). The implications of such a choice is analyzed in a following paragraph. Three-dimensional transport models which use advanced turbulence models are able to compute an eddy viscosity which account for most turbulent features of the flow. A strong relationship is suggested by many authors (e.g. Simoes & Wang, 1997) with the eddy diffusivity for mass transport, or turbulent mixing coefficient, using the *Schmidt number*:

$$\sigma_t = \frac{\nu_t}{\varepsilon_t} \quad (6.15)$$

with ν_t the eddy viscosity and ε_t the eddy diffusivity. A powerful turbulence model, combined with an appropriate choice for the Schmidt number, gives thus higher insights in the values for mixing intensities. Turbulence modelling in this study is reduced to its simplest form using an Elder model and adopting more complex formulations for the mixing coefficients would be based on pure speculations.

6.4.3 Flow conditions

The uniform flow field is computed with an Elder turbulence model from equation (3.29) which provides, as input to the scalar sediment transport model, the distribution of ve-

Table 6.2: *Telemac-2D numerical hydrodynamic parameters*

	Q (l/s)	h_{av} (m)	α_l (—)	n_{mc} (m/s ^{1/3})	n_{fp} (m/s ^{1/3})
RBS	15	0.063	0.1	0.01	0.01
	20	0.069	0.18	0.01	0.01
RBR	15	0.0665	0.3	0.01	0.0118
	20	0.0742	0.21	0.01	0.014
MBS	9.1	0.062	0.15	0.0275	0.0099
	15	0.072	0.15	0.023	0.01
MBR	9.1	0.0653	0.15	0.024	0.013
	14	0.0746	0.15	0.022	0.013
MBV	9	0.072	0.15	0.024	0.033
	14	0.094	0.15	0.024	0.065

locity and the corresponding friction velocity. The Elder model or the constant dimensionless (or relative) eddy viscosity model gives in a compound channel better agreement with measurements than does the depth-averaged k - ϵ model with its standard empirical constants (Radojkovic & Djordjevic, 1985).

Roughness coefficients are chosen to fit the numerical velocity profiles to experimental velocities (Fig. 5.16). Uniform water depths from table 5.4 are imposed downstream of the compound channel numerical models. Both rigid bed and mobile bed geometries are modelled in the study with the same cross section as for the experiments (Figure 5.10). If a mean slope of $S_0 = 1.9/1000$ is observed in the laboratory flume, many irregularities remain and the floodplain distortion is represented in Figure 6.22. The bottom topography of the flume is precisely introduced in the numerical models to treat any subsequent effect on sediment transport and deposition.

Table 6.2 summarizes the *Telemac-2D* parameters used for the numerical runs. Roughness coefficients fortunately show some resemblance to those of Table 5.5. As the objective was to reproduce the velocity distribution as precisely as possible, the α_l coefficient of the Elder turbulence model is also varied. The experimental data and the numerical velocity distributions for the different test cases are presented in Figure 6.23. For RBR 15 l/s, a numerical velocity profile is presented with inappropriate parameter settings ($n_{fp} = 0.014$ and $\alpha_l = 0.1$).

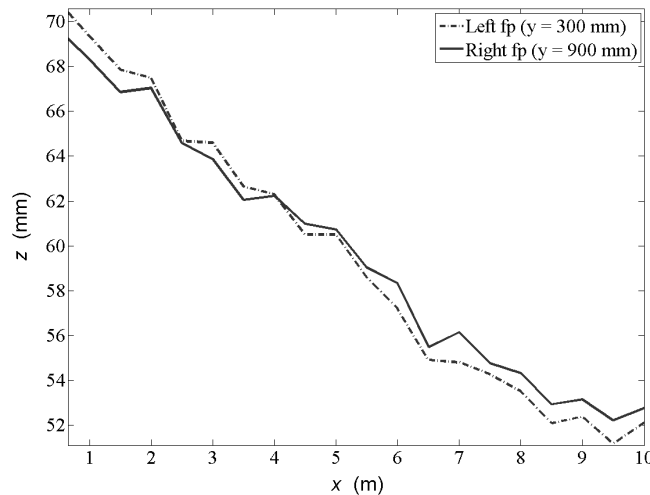


Figure 6.22: *Left ($y = 300$ mm) an right ($y = 900$ mm) floodplain topography (at equal distance from the flume's side wall). Distorsions of $\cong 1$ mm are present.*

6.4.4 Transfer conditions

Tracer injections instead of sediments precise how diffusion/dispersion and thus lateral transfer is influenced following the different flow conditions in the numerical runs. Solute acts very differently than sediment, which has a certain mass and is capable to settle on the floodplains and changes the transfer conditions, but is capable to illustrate the mixing conditions of each situation.

The mixing coefficients are supposed to translate in the model any mass transfer due to molecular diffusion and dispersion at a lower scale or due to turbulent diffusion at a higher scale. A constant mixing coefficient throughout the computational domain and the different flow conditions would be too simplistic and unrealistic. As seen previously, a coefficient proportional to the local shear velocity and the water depth is widely used in literature but much uncertainties remain concerning the proportionality constant (Eq. (6.11)). Because satisfactory calibration was possible regarding experimental tracer injections while selecting a constant value in each subsection, the calibrated non-dimensional mixing coefficients of Table 6.1 are used and scaled to the hydraulic conditions of all numerical runs ; this provides the coefficients of Table 6.3. The major weakness of this approach is that the aforementioned calibration was obtained for a given turbulent state with relatively low vertical axis vortices action (Figure 6.17) and for precise tracer injec-

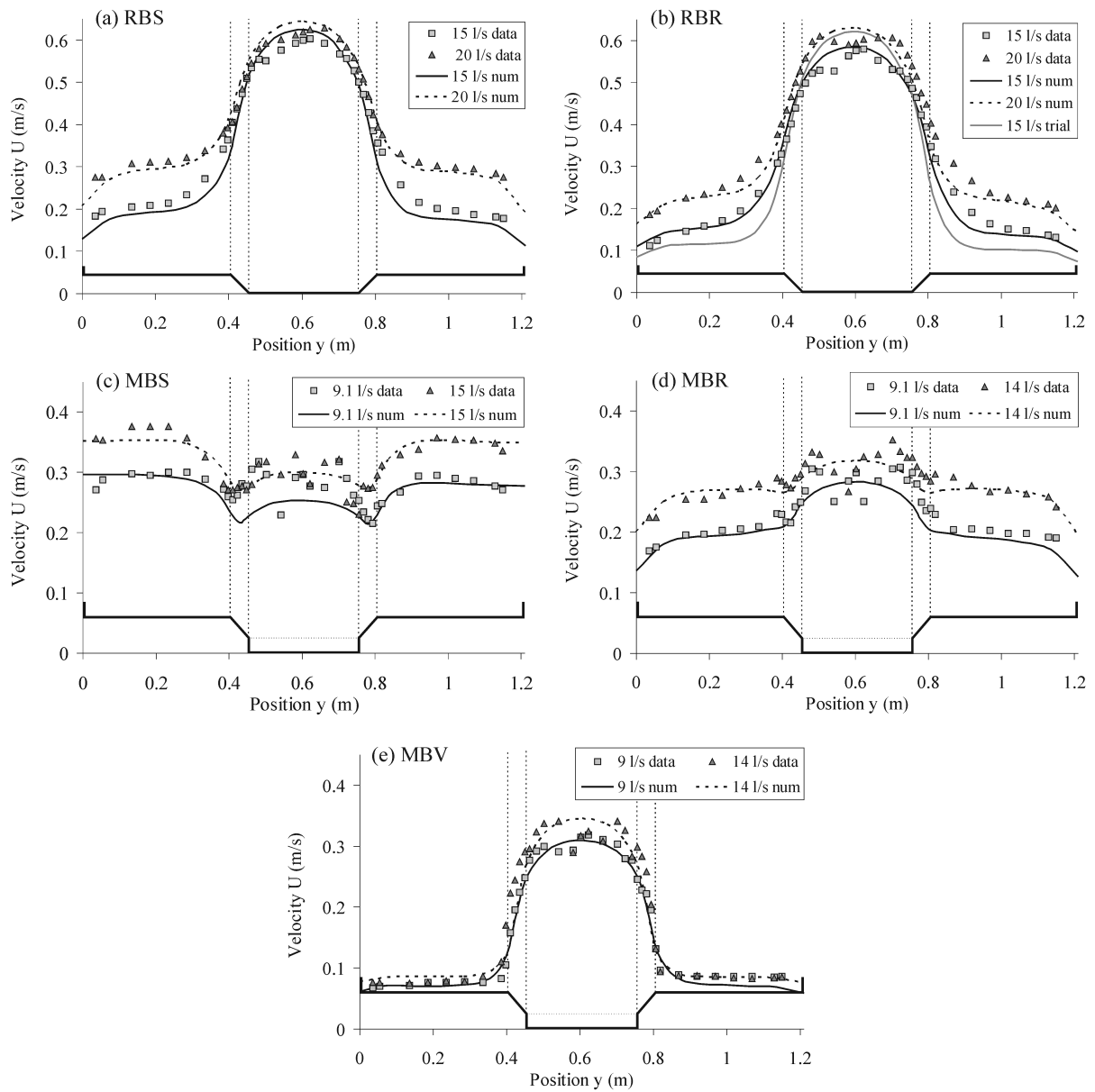


Figure 6.23: Depth-averaged velocity distribution (experimental and numerical): (a) RBS, (b) RBR, (c) MBS, (d) MBR, and (e) MBV

tion positions. The proven effect of turbulent structures on solute, which develop following the extent of the velocity gradient, is thus neglected. Fortunately, the diffusion/dispersion intensity will not reveal itself as the only parameter which conditions the quantity of sediment collected on the floodplains M_s from Eq. (6.14).

Table 6.3: *Scaled subsectional diffusion coefficients (m^2/s)*

$\nu_T \times 10^4$	RBS		RBR		MBS		MBR		MBV	
	15	20	15	20	9.1	15	9.1	14	9	14
$0.135 (u_*h)_{mc}$	2.56	3.03	2.82	3.28	2.59	3.26	2.78	3.44	3.24	4.83
$0.65 (u_*h)_{fp}$	1.04	2.13	1.63	2.8	1.51	3.35	1.96	3.76	3.19	8.08

Meanwhile, tracer was injected in all flow configurations upstream of the main channel at $x = 0$ m. Lateral transfer of tracer mass flow rate was analyzed using the procedure described by (6.12) while setting : (a) a constant diffusion coefficient $\nu_T = 3 \times 10^{-4}$, (b) $\nu_T = 0.2 u_* h$ (Djordjevic, 1993) or (c) scaled constant diffusion coefficients in each subsection (Table 6.3) with a transition at the interface. A concentration C of 1 g/l is imposed for case (a) and (b) while the equilibrium concentration (mobile bed) or experimentally injected (rigid bed) concentration is imposed for case (c). Tracer mass flow rate is chosen for analysis, as it determines the available sediment mass susceptible to deposition per unit time.

Table 6.4: *Average solute mass flow rate transferred towards the floodplains (%)*

ν_T	RBS		RBR		MBS		MBR		MBV	
	15	20	15	20	9.1	15	9.1	14	9	14
(a) 3×10^{-4}	1.82	3.79	3.42	4.23	6.69	8.5	5.71	7.94	4.62	6.0
(b) $0.2 u_* h$	1.67	3.73	3.36	4.19	6.39	8.36	5.45	7.82	4.49	5.98
(c) Table 6.3 val.	1.69	3.76	3.37	4.22	6.46	8.54	5.56	8.03	4.64	6.55

Table 6.4 gives the average solute mass flow rate transferred towards the floodplains (in % of the injected mass) for all three diffusion coefficient combinations. The averaging procedure is made on tracer mass flow rate computed on the floodplains every meter from the injection point to the end of the flume $x = 10$ m. Compared to the evolution of hydraulic conditions (water depths h and velocities), the different combinations of diffusion coefficients show relatively low influence on the transferred mass towards the

floodplains. The effect of hydraulic conditions on concentration profiles is visible in Figure 6.24 where constant diffusion is imposed and concentrations are represented 6 m away from the source. Higher water depths combined with lower longitudinal velocities (Figure 6.23) favor an increased lateral diffusion flux, as concentrations for RBR 15 l/s appear noticeably larger on the floodplains than for the RBS 15 l/s case. If the MBV 14 l/s case presents the highest concentration values on the floodplains for those reasons, mass conservation explains that the mass on the floodplains transferred from the main channel per unit time and available for deposition will be greater for MBS cases than for others (Table 6.4a).

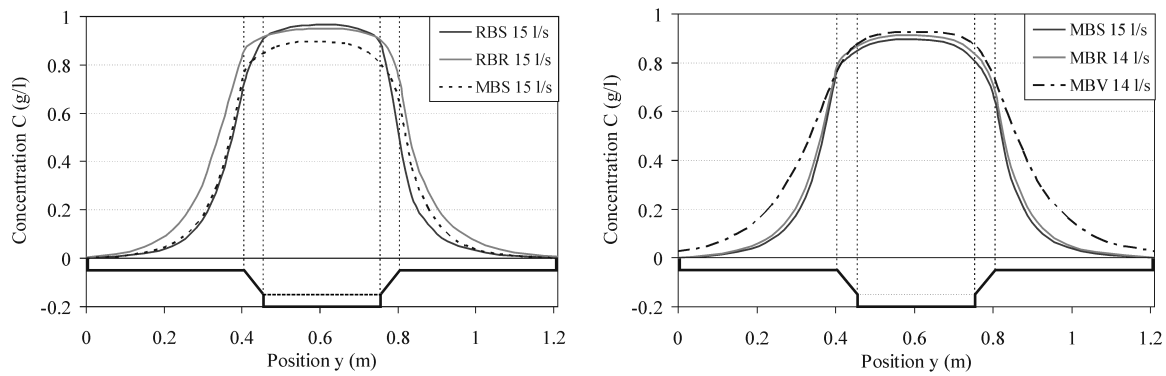


Figure 6.24: Solute concentration profiles at $x = 6$ m with injection of 1 g/l in the main channel ($x = 0$ m). The diffusion coefficient is constant throughout the computational domain : $\nu_T = 3 \times 10^{-4}$

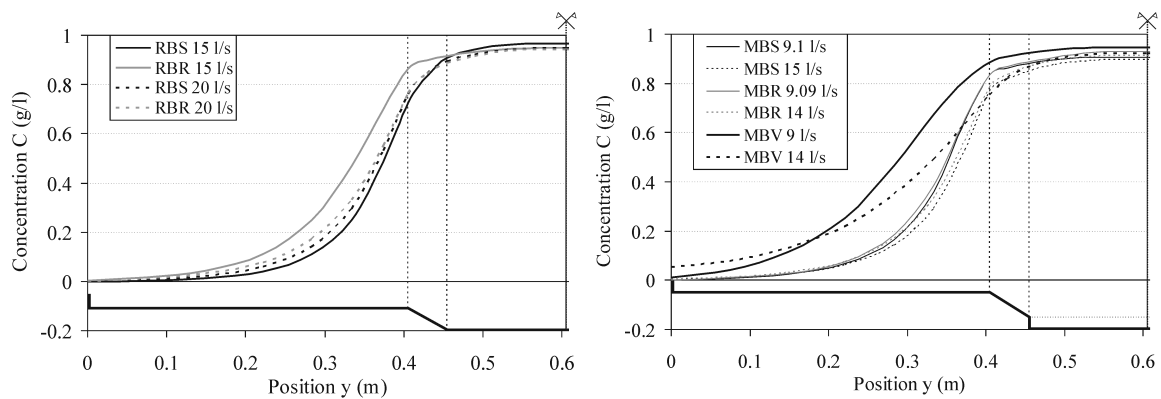


Figure 6.25: Solute concentration profiles at $x = 6$ m with injection of 1 g/l in the main channel ($x = 0$ m). The diffusion coefficients are values from Table 6.3

If constant diffusion isolates the effect of hydraulic conditions on solute transport, scaled diffusion coefficients from Table 6.3 aspire to represent the effect of turbulence on lateral transfer. Figure 6.25 illustrates the concentrations for all modelled hydraulic conditions if those coefficients are used and a $C = 1 \text{ g/l}$ concentration is imposed upstream. Those profiles give a good idea of the mixing conditions that are expected in all configurations, if sediment had no mass and could not deposit.

Figure 6.24 presented asymmetric concentration profiles which are undoubtedly the result of topographic forcing on the tracer plume due to irregularities. The effect on the plume of subsequent transverse flow is presented on Figure 6.26 for the RBS 15 l/s case. The forcing on the concentrations is particularly visible in the $x = 4.4\text{--}5.6 \text{ m}$ zone where large outflow from the right floodplain comes together with a successive inflow onto the left floodplain.

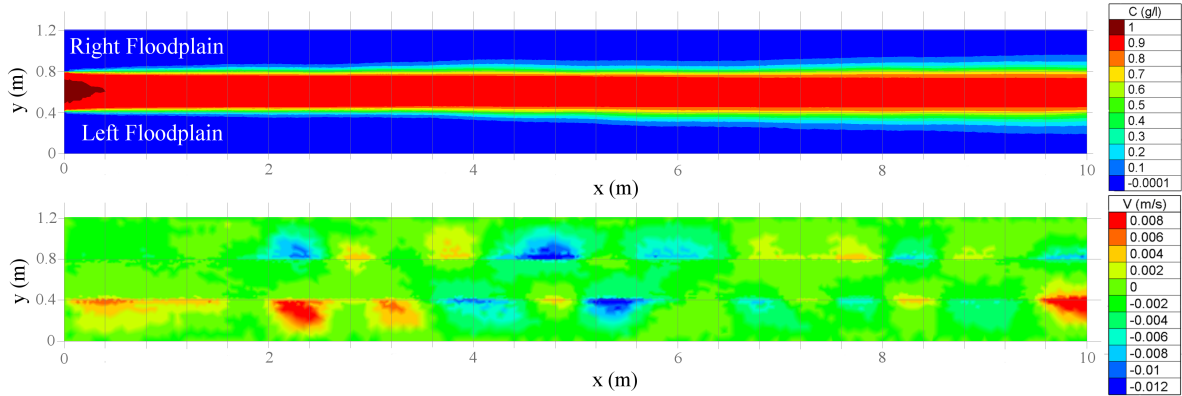


Figure 6.26: Solute concentration $C \text{ (g/l)}$ with injection of 1 g/l in the main channel ($x = 0 \text{ m}$) and transverse velocities $V \text{ (m/s)}$ for RBS 15 l/s. The diffusion coefficient is constant throughout the computational domain : $\nu_T = 3 \times 10^{-4}$.

A general trend over the different flow configurations is extracted, and illustrated for the RBR 15 l/s case on Figure 6.27. At first, a higher diffusion towards the right floodplain is observed with higher floodplain water depth compared to the left floodplain. At approximately $x = 4 \text{ m}$, this tendency is inverted and the left floodplain demonstrates relatively higher lateral mixing and water depths. Mixing is thus very sensitive to light topographic irregularities (Figure 6.22) which causes water depths variations together with transverse flow. In fact, this disparity between the left and the right floodplain was observed experimentally (Chapter 5). Figure 6.28 presents the numerical tracer mass flow rate transfer (%) towards the left and right floodplain which follows the above-mentioned

tendency shift for the RBS 15 l/s case. The collected sediment mass on both floodplain after an experiment, along with the view of the final deposition profile expresses also this initially higher transfer towards the right floodplain before a sensitively higher deposition for the $x = 5-7.5$ m and $x = 7.5-10$ m collection zones.

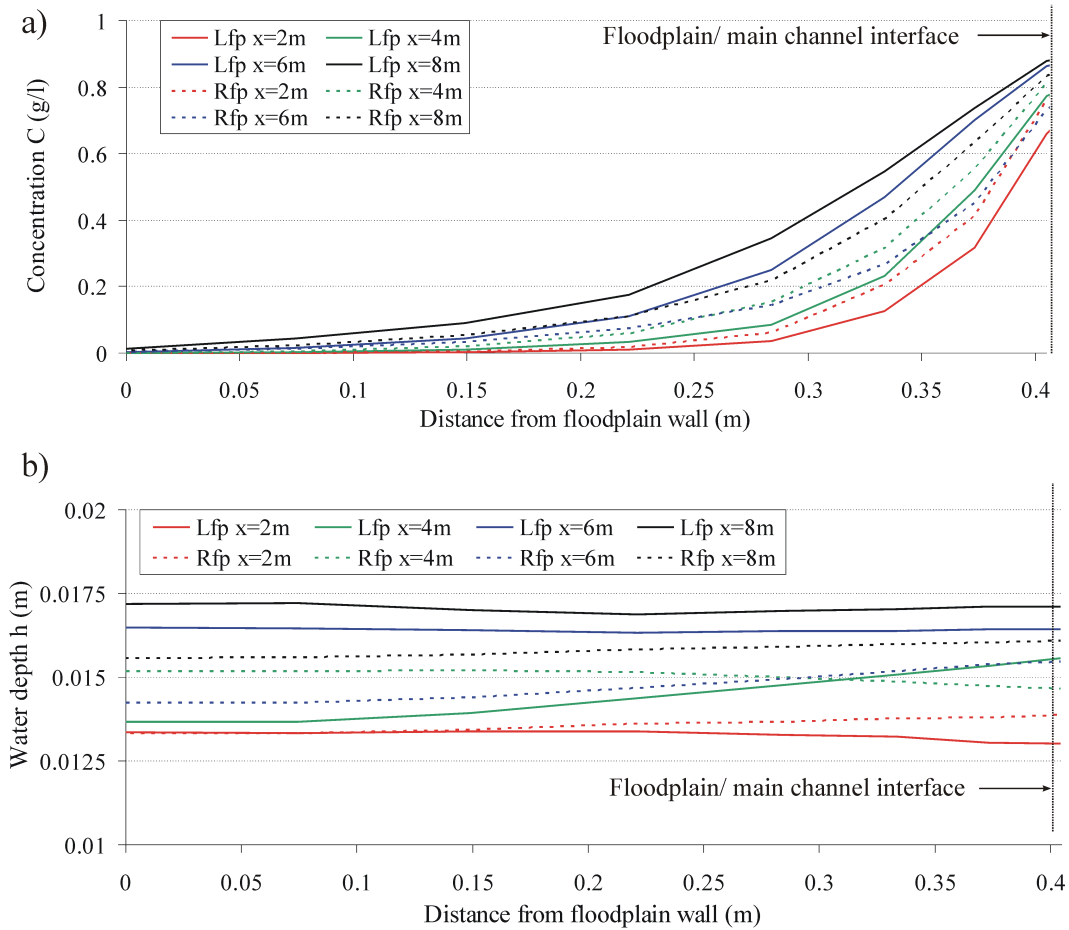


Figure 6.27: RBR 15 l/s left and right floodplain profiles at different positions from injection $C = 1$ g/l in the main channel ($x = 0$ m). (a) Solute concentration profiles C (g/l). Diffusion coefficients are values from Table 6.3. (b) Floodplain water depth profiles h (m)

Mesh convergence study and numerical diffusion

As sedimentological runs may take a great amount of time, a relatively coarse mesh was used for the simulations in this chapter. The numerical diffusion was investigated through its effect on concentration profiles in two different meshes. The Mesh A, used throughout this section for mixing analysis and in the rest of the chapter, counts 7083 mesh points

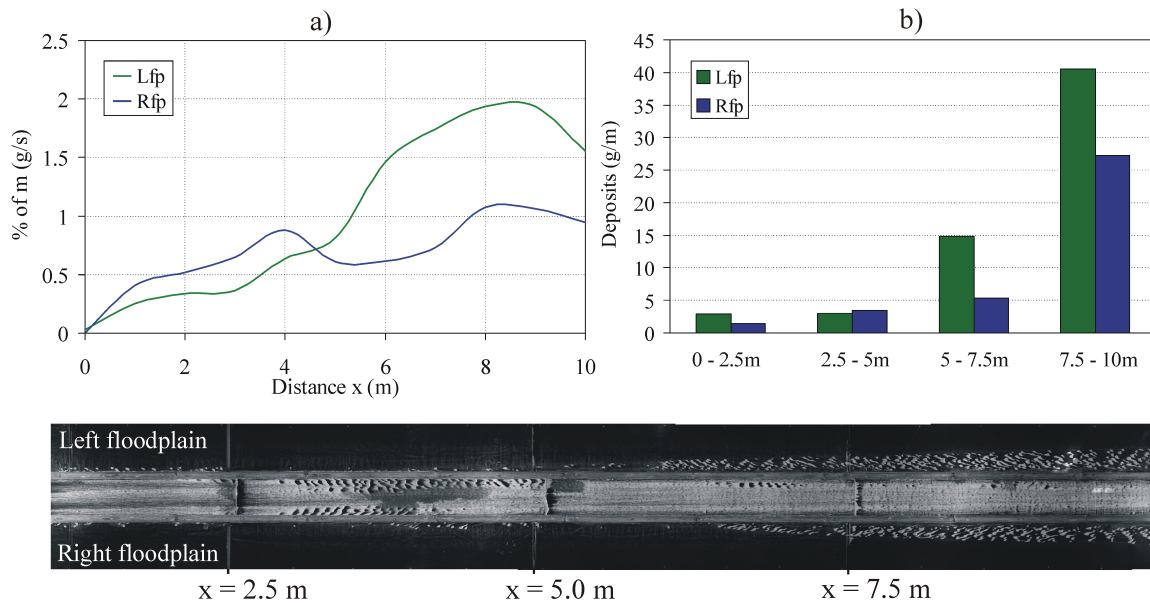


Figure 6.28: *RBS 15 l/s. (a) Tracer mass flow rate transfer towards left and right floodplain along the channel for case (c) diffusion in Table 6.4. (b) Experimentally collected sediments along the flume on right and left floodplains. Final deposition profile is also represented.*

with 3 cm elements in the subsection's interface region and 8 cm elements elsewhere. The mesh B, used to analyse numerical diffusion, counts 18333 mesh points and reaches almost the maximum limit recommended by the *Telemac* software developers (20000 pts). Mesh elements present a characteristic size of 3 cm. Figure 6.29 gives the concentration profiles for RBR 15 l/s simulations with both meshes and a constant diffusion coefficient. Concentrations stay relatively close in the region next to the interface, where most of the deposition is expected. In terms of average solute mass flow rate transferred towards the floodplain (Table 6.4), if simulations with Mesh A gave a 3.42 % transfer, simulations with Mesh B give 3.22 % of transfer and a more or less 5 % of error is expected on this coarser mesh compared to the finer one.

More generally, numerical diffusion was investigated using a method described by Simoes & Wang (1997). The longitudinal variation of the transverse concentration variance in case of nul diffusion coefficient was analyzed. For all meshes used throughout this study, and presented in appendix E, numerical diffusion was found to be at least an order of magnitude lower than the physical turbulent diffusion, predicted with coefficients calibrated e.g. in table 6.3.

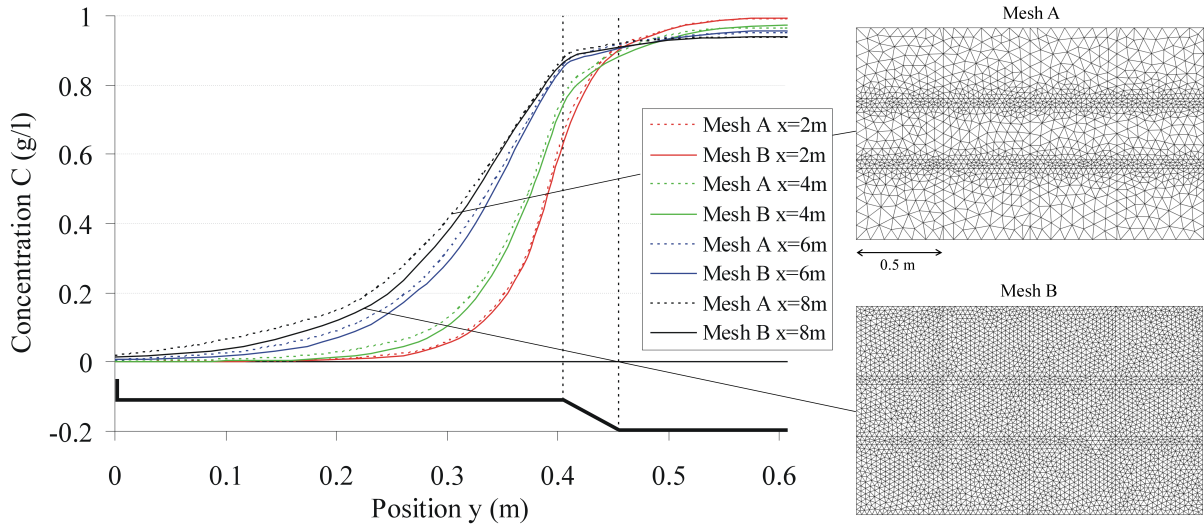


Figure 6.29: *RBR 15 l/s concentration profiles with constant diffusion ($\nu_T = 3 \times 10^{-4}$) in mesh A or mesh B. Profiles are given for the left hand side of the flume at different distances from the injection.*

6.4.5 Sediment transfer and deposition

If the transferred sediment mass towards the floodplains and the surface deposition rate are both constitutive elements of the overall process of deposition formation on floodplains, engineering practice requests to provide estimations for both parts. The fine sand transport and deposition experiments developed in Chapter 5 illustrated the composition and the result of both interrelated phenomena. A correctly calibrated numerical model is capable of distinguishing the percentage of sediment mass transferred from the main channel towards the floodplains and the proportion that will finally deposit. The transferred mass depends on diffusion/convection processes which was recognized earlier to be a function of local water depths, velocities and turbulence intensities. This section will illustrate the role on mass transfer of deposition which acts as a concentration sink, together with a refeeding process. Concerning the net deposition rate, Eq. (3.62) expresses the interplay between deposition and erosion fluxes. In practice, the net deposition will thus depend on local hydraulic conditions and surface roughness with possible sediment trapping ability. Numerical models request further information, as the user is invited to set manually an intrinsic motion threshold as defined earlier by Eq. (3.56). The influence of the critical shields parameter θ_{cr} on deposition will be discussed below.

Table 6.5: *Experimental and numerical deposition time*

Experiments		Numerical runs	
Code	Deposition time (s)	Time step Δt (s)	Number of time steps (N)
RBS 15 l/s	1680	0.01	168000
RBS 20 l/s	1568	0.008	196000
RBR 15 l/s	1620	0.01	162000
RBR 20 l/s	1256	0.008	157500

Rigid bed and mobile main channel bed simulation runs are presented successively, as very different phenomena were recorded in either cases. Non-erodible bed is imposed everywhere for rigid bed simulations and are runned over the same time as for Chapter 5 experiments (Table 6.5). General tendencies only are observed for mobile bed simulations with a constant time step $\Delta t = 0.01s$ and runs of 100s ($N=10000$). Indeed, the presence of non-erodible bed on the floodplains and erodible bed in the main channel make those simulations difficult to control.

Rigid bed

Motion/suspension initiation threshold definitions by van Rijn (1984a, 1984b) are given in Section 3.5.2, and must be used with great care, as they usually apply to sand bed surfaces. This threshold θ_{cr} intervenes in the computation of erosion fluxes in Eq. (3.62) through c_a^* (the equilibrium near-bed concentration) defined in Eq. (3.63) by Zyserman & Fredsoe (1994). The higher this parameter, the more the sediment "sticks" to the surface.

Besides the critical Shields parameter θ_{cr} , erosion will be determined by local velocity and roughness through the mobility parameter:

$$\theta_s = \frac{u_*^2}{(s-1)gd_{50}} \quad (6.16)$$

with the shear velocity u_* in 2-D flows defined by Eq. (3.27). Subsectional means of θ_s for a few highlighted experiments are drawn on Figure 6.30 to give an indication on the difference between the floodplain's and main channel's mobility parameters. The different theoretical thresholds are also defined on this figure, which correspond to a sediment mean diameter of $d_{50} = 91 \mu m$. This subsectional mean mobility is to be taken with great care, especially in the interaction zone where a relatively strong velocity gradient may cause sudden variations. However, as already explained earlier in section 5.2.4, all experiments

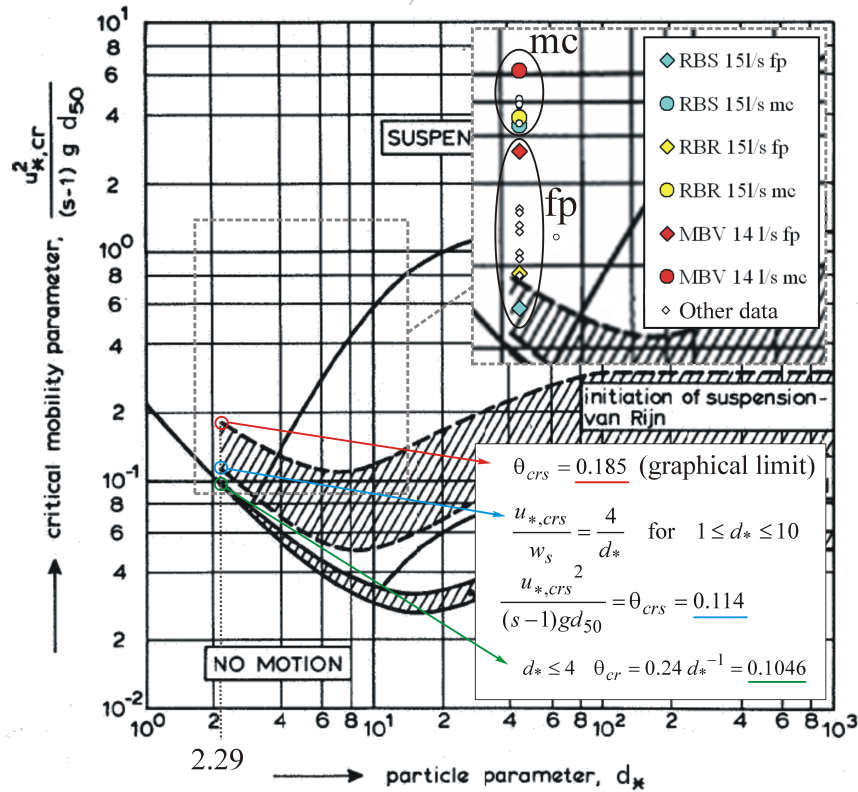


Figure 6.30: *van Rijn's limits and experimental mobility parameter values.*

seem to show conditions where the theoretical motion threshold $\theta_{cr} = 0.1046$ is exceeded and a *dynamic* equilibrium could be expected.

RBR 15 l/s simulation runs were performed to evaluate the influence of this θ_{cr} parameter on the deposited sediment mass on the floodplains, or M_S from Eq. (6.14), against the injected total mass in the main channel. Table 6.6 expresses the percentage of sediment collected (transferred and then deposited) on the floodplains compared to the quantities injected with a critical shields $\theta_{cr} = 0.1046 \pm 50\%$. The difference in sediment mass collected is comparable to the difference in θ_{cr} from one simulation to another and places the calibration of the diffusion coefficients discussed in the previous sections into a certain perspective.

The choice of appropriate diffusion coefficients is source of many uncertainties and modelling errors, as discussed earlier. The critical shields parameter θ_{cr} is identified as a second potential source of error, as much confusion exist concerning its definition, espe-

Table 6.6: *Percentage of sand collected on the floodplains against the injected mass for RBR 15 l/s*

θ_{cr}	Transfer/depos. (%) (difference with (a))
(a) 0.1046	5.21 (-)
(b) 0.0523 (- 50 %)	2.84 (-46% diff.)
(c) 0.1569 (+ 50 %)	7.75 (+49% diff.)

cially when considering the initiation of suspension. Much effort was already spent on discussing the mixing coefficients applied to solute transport. Further criticism is possible as sediment acts very differently compared to solute. Indeed, Figure 6.31 presents RBS 15 l/s concentration profiles for injection of 0.8 g/l of tracer or sediment in the main channel. The diffusion coefficients from Table 6.3 are applied in either cases. Deposition of sand on the floodplains act as a concentration sink and lower concentrations are observed for sediments. 1.7 % of transfer towards the floodplains was detected for solute (Table 6.4), while 3.05 % of the total sediment mass injected is collected on the floodplain surface, together with the observation of an average sediment transfer of 5.08 % where a deposition and refeeding process increases the mass transferred towards the plains.

This sink effect is marked for all runs. Figure 6.32 presents the two-dimensional view of Figure 6.31 for RBS 15 l/s simulation, together with the bottom evolution after 1620s and using the theoretical $\theta_{cr} = 0.1046$.

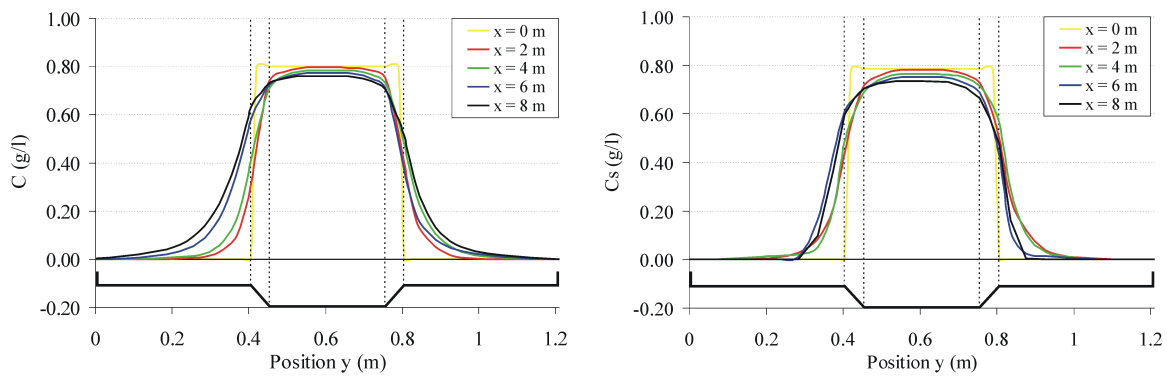


Figure 6.31: *RBS 15 l/s concentration profiles with Table 6.3 diffusion coefficients. (left) solute concentration C (g/l). (Right) Fine sediment concentrations C_s (g/l) and $\theta_{cr} = 0.1046$.*

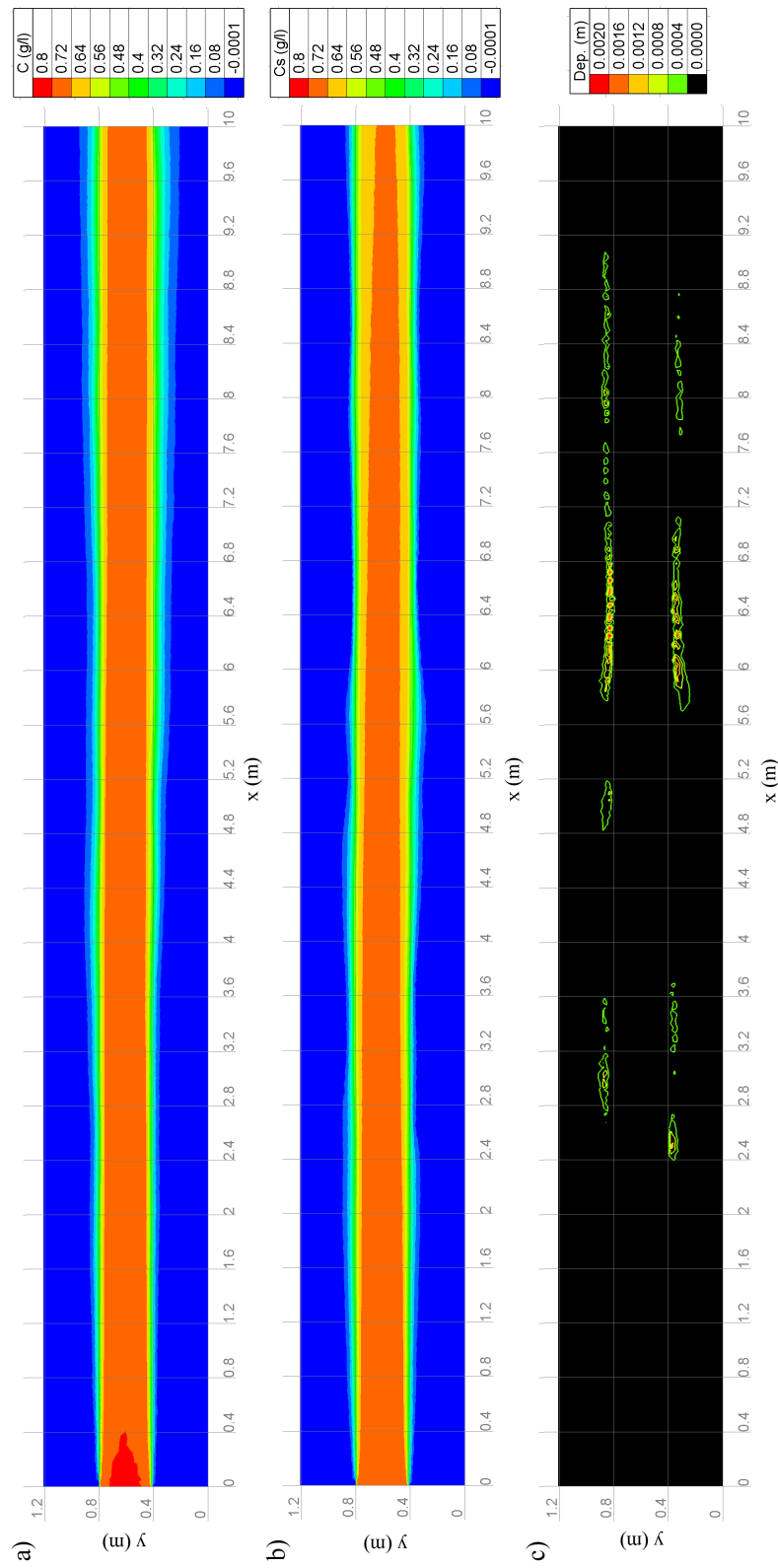


Figure 6.32: RBS 15 l/s 2-D view for Figure 6.31. (a) Solute concentration C (g/l). (b) Fine sediment concentrations C_s (g/l). (c) Bottom evolution (m) after 1620s simulation superposed to sediment concentrations

Sediment deposits will form when the balance between erosion and deposition fluxes favours the latter. As dictated by Eq. (3.62), this situation will arise when sufficient sediment concentration is present in the water column, combined with a relatively low mobility parameter θ_s compared to the defined threshold θ_{cr} . Floodplains offer favourable conditions, with lower velocities and higher roughness (at least for RBR simulations). The preferential deposition zones for RBR 20 l/s are related to lower local velocity, as pictured in Figure 6.33. An increase in θ_{cr} shifts the deposition zone in higher velocity situations and increases the net flux which leads to higher deposition heights. Figure 6.34 illustrates this evolution of deposition with θ_{cr} . 2.038 % of the injected sand was collected on the floodplains in the $\theta_{cr} = 0.1046$ case, and 8.58 % was collected in the $\theta_{cr} = 0.185$ case.

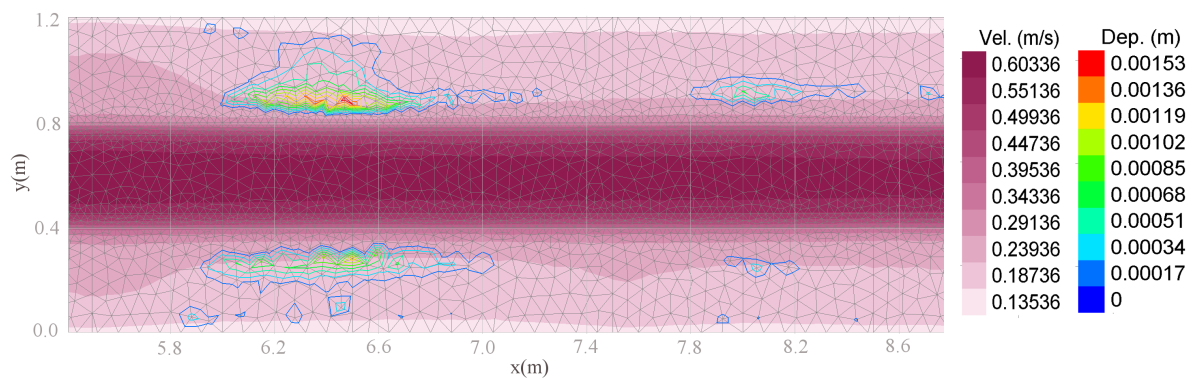


Figure 6.33: RBR 20 l/s with $\theta_{cr} = 0.1046$. Deposition zones (m) superposed to velocity field (m/s). The mesh is also presented to explain deposition contours.

The critical Shields parameter acts finally as a major actor in the deposition process. Theoretically, it represents the energy requested to initiate sediment movement. One seizes this opportunity to represent in the model the "trapping" ability of smooth or rough surfaces.

Table 6.7 summarizes the solute transfer conditions for each rigid bed geometry and using the scaled diffusion coefficients from Table 6.3. The experimental and numerical sediment mass collected on the floodplains compared to the injected mass (%) is also presented. Sediment deposits with $\theta_{cr} = 0.1046$ are presented for all rigid bed geometries in Figure 6.35 and the collected mass is given in Table 6.7. Compared to experiments with rough surface, the numerical deposition is too low, as if the trapping ability of the steel plates increased the energy required to bring deposited sediments back into suspension.

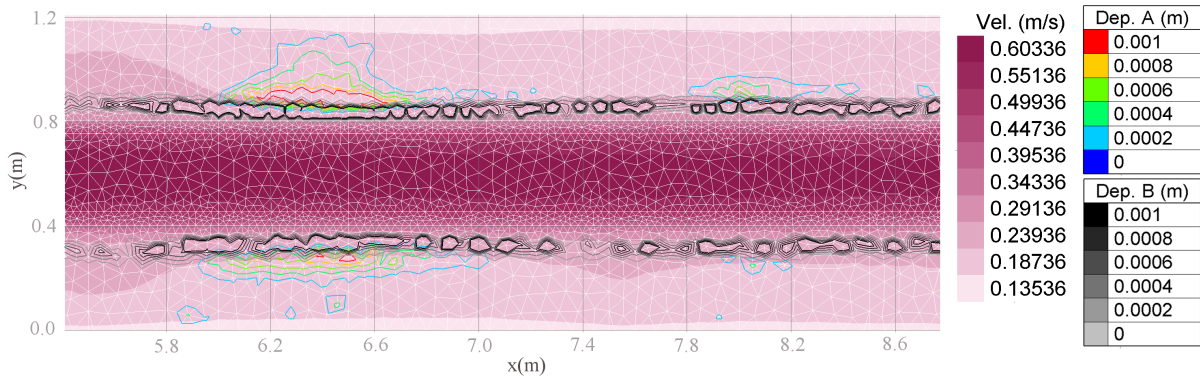


Figure 6.34: RBR 20 l/s with $\theta_{cr} = 0.1046$ for deposition zones A (m) and $\theta_{cr} = 0.185$ for deposition zones B (m).

On the other hand, RBS 15 l/s numerical deposition are overevaluated compared to experiments, as if smooth surfaces requested a lower initiation of motion threshold than that defined for sand beds.

Table 6.7 also presents θ_{cr} values which aim at reproducing experimental deposition, where smooth surface offer poor conditions for deposition $\theta_{cr} = 0.072$ and where steel plates traps sediment so efficiently that the upper limit of initiation of suspension $\theta_{cr} = 0.185$ is needed for RBR 20 l/s.

This analysis may appear simplistic, but it gives interesting perspectives to numerical models which aim at expressing deposition efficiency, conditional to surface roughness and trapping ability. Mobile deposition is not modelled in this *suspension only* mode and this deposit mobility is responsible for the "washing out" of much sediment. This hypothesis, together with errors in velocity estimations, urges to great care. The particular case of RBS 20 l/s proves the limits of numerical modelling of deposits on the floodplains. The model is not capable to express deposition in this environment where intense velocities take place and deposition occur near the edges of the numerical flume (Figure 6.35) in very little quantities.

Mobile bed

Mobile main channel bed simulations are a little more laborious, as they request the manual definition of the non-erodible bed of Figure 5.10(a). Attempts were made to model the mobile bed experiments conducted in the laboratory flume, without great success. The main reason for this are the ill-controlled erosion and deposition sediment fluxes on the different bottom boundaries.

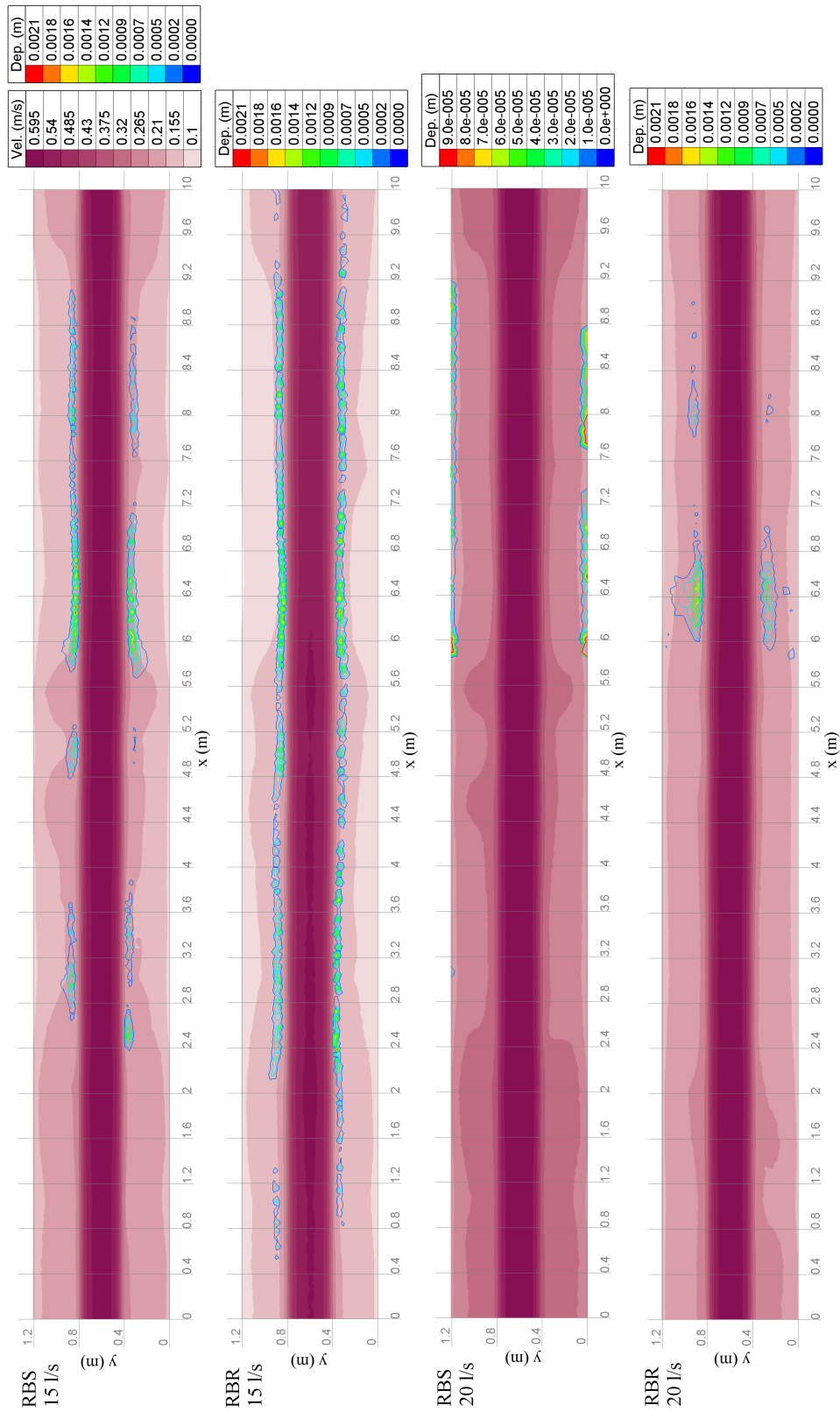


Figure 6.35: Sediment deposition (m) in velocity field (m/s) for rigid bed simulations. Simulation time is given in Table 6.5 and $\theta_{cr} = 0.1046$.

Table 6.7: *Experimental and numerical sediment transfer and deposition on the floodplains compared to the injected mass (%) together with solute average transfer rates (%) for rigid bed cases.*

	Solute (%)	Experiments (%)	Numerical deposition (%)	
			$\theta_{cr} = 0.1046$	"fitted" θ_{cr}
RBS 15 l/s	1.69	1.8	3.05	1.7 ($\theta_{cr} = 0.072$)
RBR 15 l/s	3.37	6.02	5.21	5.97 ($\theta_{cr} = 0.12$)
RBS 20 l/s	3.75	0.82	0.077	—
RBR 20 l/s	4.22	8.48	2.038	8.58 ($\theta_{cr} = 0.185$)

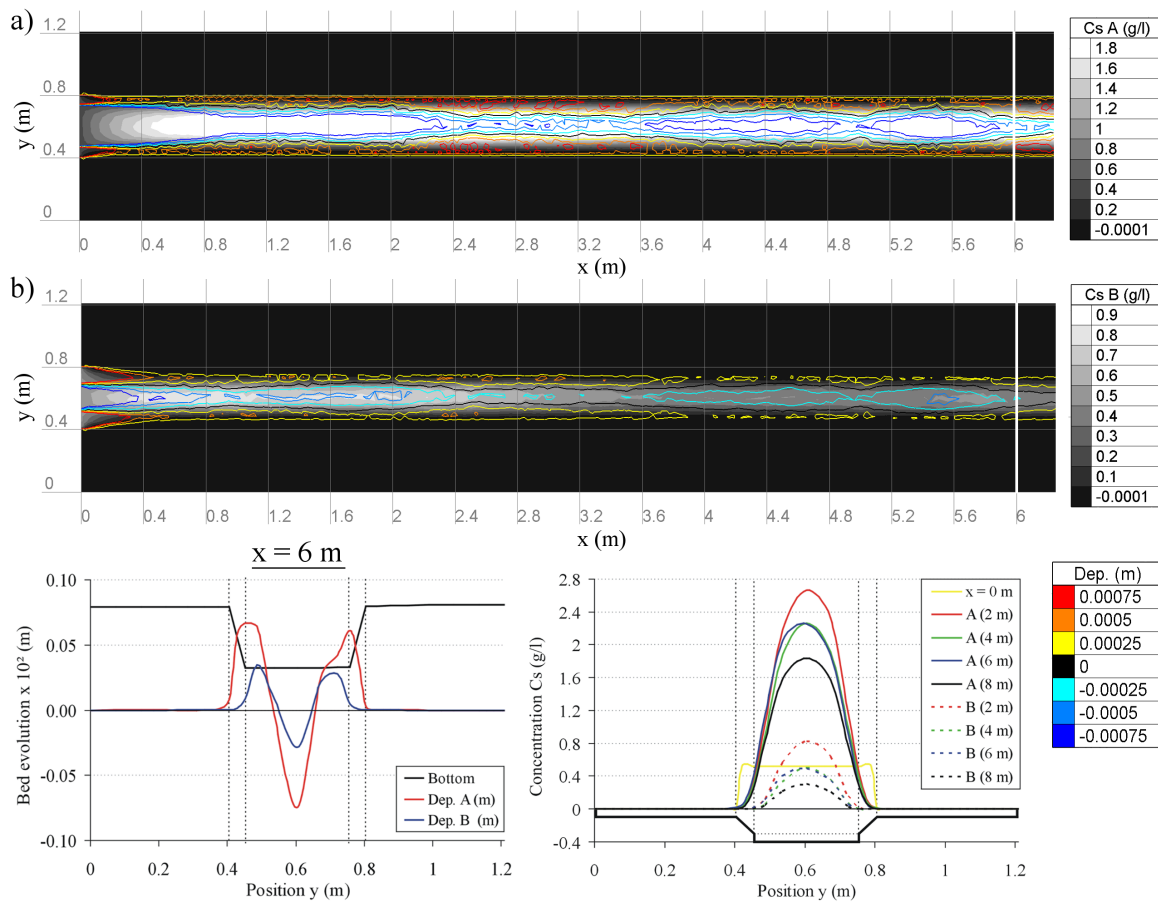


Figure 6.36: *MBV 9 l/s with (a) $\theta_{cr} = 0.1046$ and (b) $\theta_{cr} = 0.185$. Deposition contours (m) are superposed to sand concentrations C_s (g/l) after 100s simulations. At $x = 6$ m, the initial bottom is pictured, with the bottom evolution after 100s due to erosion/deposition multiplied by a factor 100.*

Figure 6.36 presents the MBV 9 l/s bed evolution ($z_{b,f} - z_{b,i}$) or deposits isocontours (m) superposed to the concentrations after 100s simulations for different values of the θ_{cr} parameter. The initial bottom level (m) together with the bed evolution (m) is given in both cases for the $x = 6$ m transverse section. Those 2-D results express the difficulty to adress a single θ_{cr} value on the domain to, on the one side, avoid too great concentrations in the main channel due to excessive resuspension of mobile bed sand and, on the other side, observe floodplain deposits. When $\theta_{cr} = 0.1046$, sediment resuspension/erosion flux leads to concentrations in the main channel that are much bigger than the observed experimental equilibrium concentration, as if the imposed energy threshold above which sediments are brought in suspension was too low. Imposing the superior limit of van Rijn's (1984b) initiation of suspension zone (Figure 6.30) lowers the concentrations and translates the additional energy required to bring sediments in suspension, which is higher than the level of energy at which sediments that are already in suspension will settle. Unfortunately, Figure 6.36(b) points out that by doing so, deposition occur in the main channel and hardly reaches the floodplains.

This contradiction between the request of a higher energy threshold over the mobile bed and a lower threshold where deposition is supposed to form is present in all mobile bed simulations. This scenario is inevitable as long as no spatial differentiation of θ_{cr} is possible with this mathematical representation of erosion and deposition fluxes : deposition is a function of the local concentration and erosion is a function of a bottom shear threshold and the bed shear. Figure 6.37 presents MBS 15 l/s simulation results at two different x locations where deposition and erosion obviously follow the bed shear τ_b profiles.

6.4.6 Perspectives

The experiments of Chapter 5 present complex deposit situations where two-dimensional modelling struggles for plausible estimations. The decision, motivated in section 6.4.2, to work in *suspension only* mode is responsible for a great deal of the inherent approximations. Experimentally observed mobile deposits and sand grains which rolled and glided above the main channel mobile bed, without being lifted completely into suspension, proves that this mathematical representation is not optimal.

Coupled bed load and suspended load simulations were tempted, and Figure 6.38 gives the results of an MBS 15 l/s run where both sediment loads are active. The situation appears pretty chaotic after 100s simulation and severe erosion/deposition already took

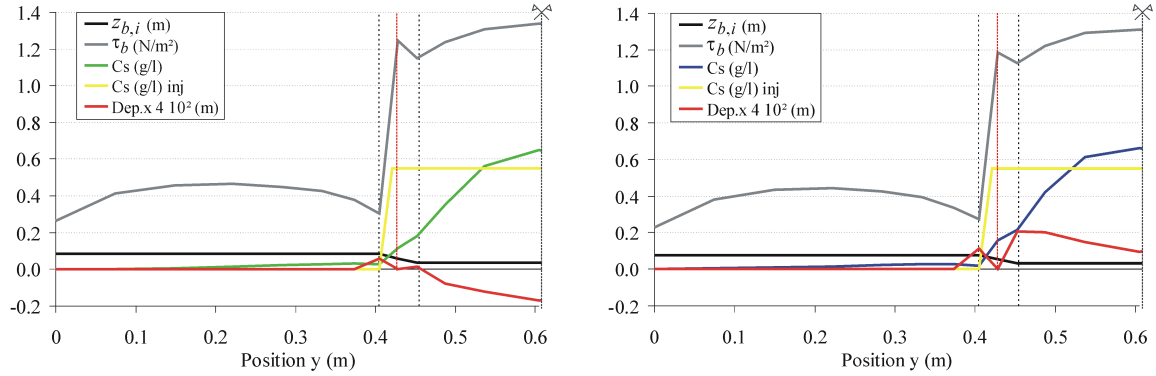


Figure 6.37: *MBS 15 l/s with $\theta_{cr} = 0.185$. Transverse profiles at (Left) $x = 4$ m and (Right) $x = 6$ m. Are represented (1) the initial bottom $z_{b,i}$ (m), (2) the bed shear τ_b (N/m^2), (3) the sediment concentration C_s (g/l) and the injected concentration profile ($x = 0$ m) and (4) the bed evolution times 400 (m).*

place in the main channel. This simulation used van Rijn's (1984a) bed load transport formula which is not adapted to the sediment used in the computations as it provides bed load transport rates predictions for particles in the range $200\text{--}2000\ \mu m$.

When bed load comes into play, an additional parameter β requests some calibration and hardens the task of the modeller. Without going into thorough details, this parameter controls the effect of a sloping bottom and reduces the sand transport rate upslope. The effect of this parameter on sand concentrations is obvious in Figure 6.39 and the default value of $\beta = 1.3$ restrains all sediment loads in the main channel, as if the lateral main channel slope at the interface hindered the diffusion of sediment towards the floodplains. The figure shows a simulation in the RBS 15 l/s geometry and confirms how suspended load transport Q_s (Figure 6.39d) is predominant compared to the bed load transport Q_b (Figure 6.39c), at least in the main channel. Figure 6.39c shows an intriguing periodic transport rate in the main channel which resemble to ripple structures which form over a mobile bed. This artefact reminds of another main limitation of this two-dimensional numerical model which is not capable to model bedforms, as bottom turbulence is not explicitly integrated in depth-averaged computations.

An efficient coupled bed load and suspended load approach, together with an improved treatment of erosion and deposition fluxes, starting with a spatial differentiation of a bottom shear threshold θ_{cr} as discussed earlier, is the key to performant two-dimensional modelling which integrate the possibility of moving deposits together with the main cur-

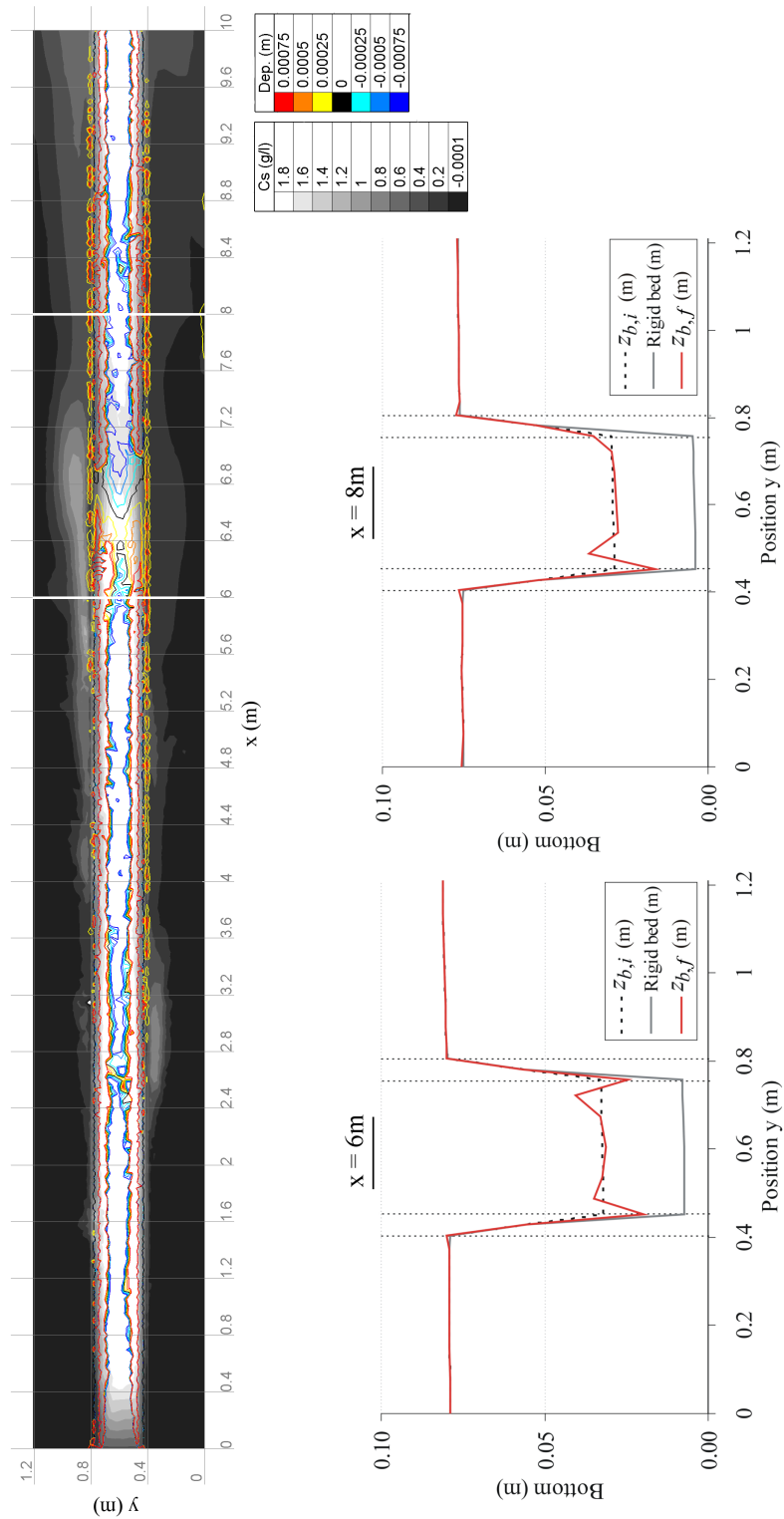


Figure 6.38: *MBS 15 l/s with $\theta_{cr} = 0.15$ and combined bed load (van Rijn, 1984a) and suspended load. Deposition contours (m) are superposed to sand concentrations C_s (g/l) after 100s simulations. At $x = 6$ m and $x = 8$ m, the initial bottom and the non-erodible bottom are pictured, together with the bottom after 100s.*

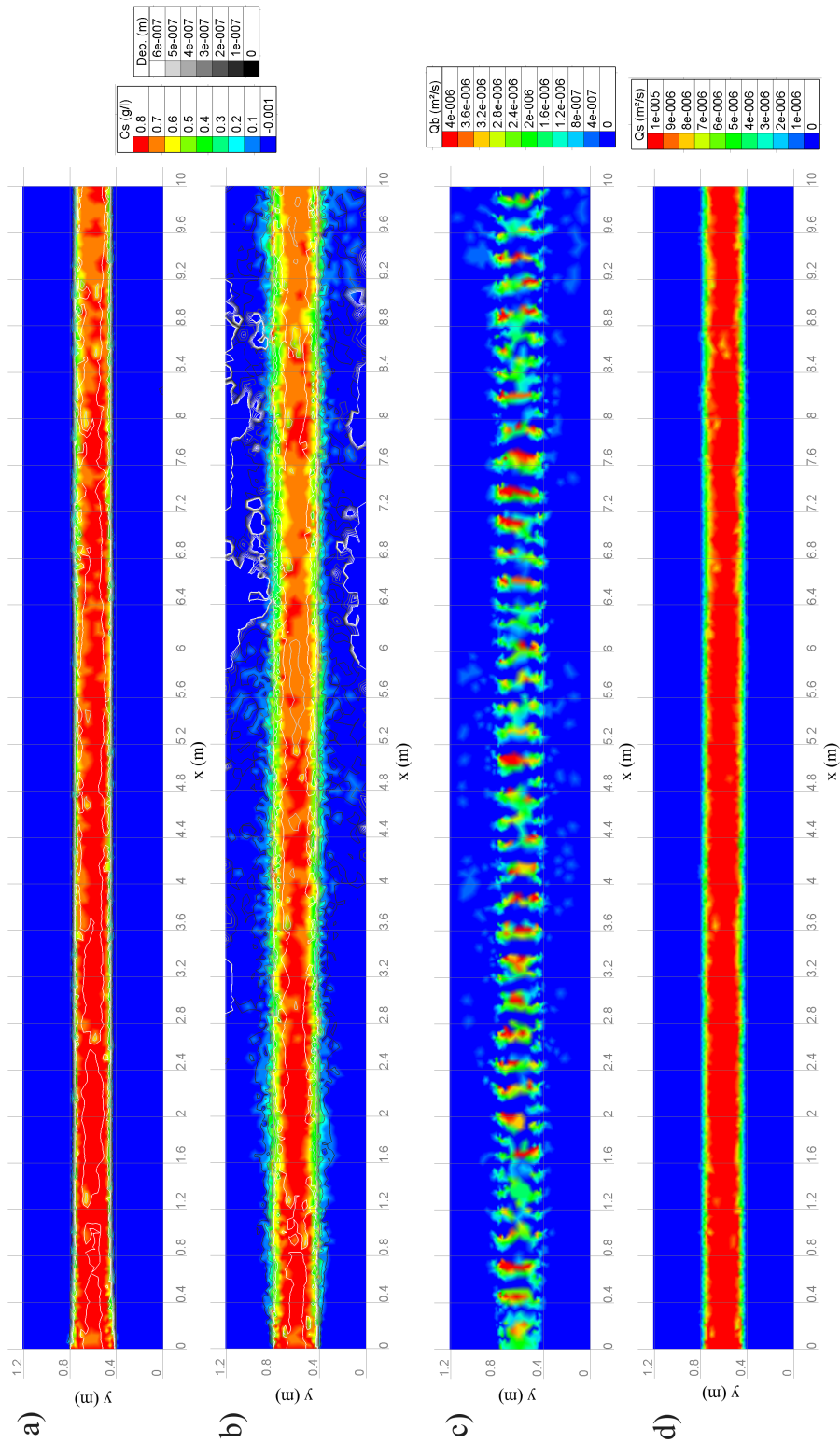


Figure 6.39: RBS 15 l/s with $\theta_{cr} = 0.15$ and combined bed load and suspended transport after 100s. Sediment deposition (m) in concentration field C_s (g/l) with (a) $\beta = 1.3$ and (b) $\beta = 0$. (c) Bed load transport Q_b (m²/s) for case "b". (d) Suspended load transport Q_s (m²/s) for case "b"

rents. For rigid bed simulations in Figure 6.35, the longitudinal distribution of deposits does not follow the experiments (Figure 6.28) and even slightly higher sediment mass is collected on the right floodplain in place of the left for the RBS 15 l/s and RBR 20 l/s cases. Transverse currents directed towards the left floodplain and away from the right floodplain could increase the relative deposition on the left floodplain (as it is observed for solute in this irregular convection/diffusion field). They have less impact in this *suspension only* mode, where deposition is particularly sensible to velocity intensity. As explained above, deposition zones are very much conditioned by the velocity field which thus requests by the same occasion important efforts to be as precise as possible, especially in the upstream zone, submitted to the influence of inlet boundary conditions.

6.5 Summary and Conclusions

The *Telemac* system is a group of numerical modelling softwares which were used in this Chapter for two-dimensional free surface flow computations in compound channels with a study of diffusion/dispersion processes (*Telemac-2D*), as well as for sediment transport and deposition modelling (*Telemac-2D* and *Sisyphe*). Both numerical study rely on experimental data reported in Chapter 5.

Despite the known three-dimensional flow structure of compound channel flow, *Telemac-2D* is first used with its solute transport module to model hydrodynamics and solute transport in a compound channel. Parallel to stable simulations where classical $Q-h$ boundary conditions are used, periodic boundary conditions are imposed which results in an oscillating velocity field with resolved large-scale horizontal unsteady motions. Those vertical axis eddies are observed experimentally in such geometry and their effect on tracer transport and lateral matter transfer is investigated. Main channel tracer injections in both stable and oscillating situations, together with concentrations comparison, proves the enhanced mixing due to the effect on the tracer plume of those planform vortices, especially in the shear zone near to the interface where strong vorticity is present.

The transfer analysis is central in this work, as it is recognized to influence the quantities of sediment deposited on the floodplains. Transverse mixing coefficients in the solute transport equation are calibrated using tracer injections data. Despite its important effects on lateral transfer of mass flow rate onto the floodplains, shown through sensibility analysis, literature values of diffusion coefficients suffer from an absence of consensus. In the future, numerical simulations of transport could evolve towards two different direc-

tions. Complex 3-D turbulent structures can be modelled with more powerful turbulence models to avoid speculations on turbulent intensity, and where mixing coefficients contain always less implicit turbulent effects. On the other side, while focusing on simplified 2-D simulations, mixing coefficients should be able to represent the precise turbulent state of the flow. To do so, turbulent structures require to be isolated, as it is done here for vertical axis vortices.

The lateral transfer is also evaluated in the rigid and mobile bed geometries from the laboratory flume. Topographic singularities which induce spatial depth variations, but also transverse flow, and thus tracer plume irregularities, could explain experimental spatial differentiation of deposition, with globally more sediment deposited on the left floodplain. A constant mixing is adopted in each subsection using scaled previously calibrated coefficients. This hypothesis is brought into perspective, as sediment is recognized to behave very differently from solute. Deposition acts as a concentration sink and the re-feeding process increases the global sand transfer towards the floodplains.

Sisyphe sedimentological simulations show how the critical mobility parameter θ_{cr} and the bed shear τ_b (function of the roughness and the velocities) control the erosion and deposition fluxes as expected by the mathematical model. Throughout rigid bed simulation runs, θ_{cr} needs to be adapted to the floodplain surface conditions (micro-topography) to find the same collected sediment mass on the floodplains as for the experiments. The trapping ability of rough floodplains is characterized by a higher θ_{cr} or mobility threshold, to limit the re-suspension flux imposed by the steel plates.

Mobile bed simulations express the difficulty to address a single threshold θ_{cr} value on the domain to, on the one side, avoid too great concentrations in the main channel due to excessive re-suspension of mobile bed sand and, on the other side, observe floodplain deposits. van Rijn's mobility and suspension initiation theoretical limits are invoked to address a differentiation with a higher energy threshold over the mobile bed and a lower threshold where deposition is supposed to form.

Chapter 7

Conclusions and recommendations

Sediment transfer from the main channel towards the floodplain and deposition formation have been investigated to obtain better insights in the conditions requested for such phenomena to take place and to form paths to develop prediction tools for deposition trends in the practice of civil and environmental engineering. Two methodologies have been followed: laboratory experiments and numerical modelling. The results of these methods converge to a characterization of the mechanisms behind sediment transfer on the one hand and deposition formation on the other hand, together with the influential parameters on both processes. Understanding fine sediment transport requested a beforehand determination of advective/dispersive capacity of the flow in compound channels (Section 7.1), in the absence of any consensus found in the literature. Complex and broadened scale turbulent structures with the subsequent strongly three-dimensional character of compound channel flows explains the tracer injection campaign presented in this dissertation, together with the numerical investigation of the particular effect of interfacial vertical axis eddies, observed experimentally, on mixing intensities. Laboratory experiments also featured fine sediment transport investigations, which were confronted to two-dimensional results from the hydrodynamic and sedimentological modelling software *Telemac* (Section 7.2), able to model the sediment deposits laid on the river banks. Finally, questions that came up during the project and questions that still remain are discussed in Section 7.3, together with suggestions for further research.

7.1 Dispersive capacity of compound channels

7.1.1 Tracer injection experiments

Mixing processes in compound channels were investigated through Salt (NaCl) solution injections. Evolution of the dispersion plume was appreciated through conductivity measurements using specially designed conductivity probes. The presence of strong turbulent features at the interface between the subsections enhanced mixing in this region as well as lateral solute transfer towards the floodplains. Results from the different injection positions were treated with a numerical simulation tool based on the development of simplified advection/diffusion equation in the same conditions as the physical model. Simulations were matched to experimental concentrations using a depth-averaged approach. When injected on the flume symmetrical axis, the plume behaviour is correctly described with transverse mixing coefficients in the main channel whose values lie within a range given in literature for straight channels ($D_{y,mc} \approx 0.18 h_{mc} u_{*,mc}$). Furthermore, injections in the shear zone between the main channel and its floodplain show interesting trend of the tracer plume at the interface, with a skewed solute distribution on the floodplain. Larger lateral mixing coefficient on the floodplains was required to match the simulated concentrations with the experiments ($D_{y,mc} \approx 1.7 h_{fp} u_{*,fp}$), confirming the enhanced action of complex turbulence mechanisms in the interaction zone on solute transport.

7.1.2 Influence of large scale structures

With two-dimensional dispersive characteristics determined for a given flow configuration, *Telemac-2D* was used together with its solute transport module to model hydrodynamics and solute transport in a compound channel. In parallel to stable simulations where classical Q - h boundary conditions are used, periodic boundary conditions are imposed which produce an oscillating velocity field with some kind of explicit modelling of large scale turbulent vortices, using a Sub-Depth-Scale modelling approach. Tracer was injected at a constant rate in the main channel and time-averaged concentrations plotted at different distance from the source for hydrodynamically stable and unstable numerical simulations. A comparison proves the enhanced mixing due to the effect on the tracer plume of those planform vortices, especially in the shear zone near to the interface where strong vorticity is present.

7.2 Sediment transport and deposition in compound channels

7.2.1 Laboratory exploration

The novel laboratory campaign reported in this thesis was initiated to improve our understanding of fine sediment transport and deposition during floods. The study focuses on sediment exchanges between main channel and floodplains in a flume featuring prismatic geometry. More precisely, the lateral sediment transfer is gauged through meticulous weighting and the subsequent deposition patterns are analyzed using e.g. digital imaging.

Experiments were either performed over a mobile main channel bed with recirculation of the main channel sediment-laden water, or with sediment feeding at the upstream end of the main channel section and transport observed in fresh water over a rigid bed. If the former configuration offers conditions closer from natural rivers, the latter presents higher experimental control. In both cases, the collected sediment on the floodplains gave a measure of the transfer rate of sediments towards the floodplains from the main channel, deposited and not resuspended. The study clearly identifies the following parameters that govern floodplain deposition: (a) turbulence intensity generated by the lateral velocity gradient in the shear zone at the interface between main channel and floodplain which acts on the transferred sediment and influences the available sediment for deposition (C_{SL}); and (b) the floodplain suspension efficiency (η_{SL}) which is presumed to show a link with the floodplain micro-topography combined with the hydraulic conditions, taken into account through the calculated stream-power, and characterizes the conditions for deposition.

A quantitative and qualitative study of sediment deposition is also performed in correlation with various floodplain roughness or configurations. Smooth floodplains basically showed a certain threshold in stream-power where increase in re-suspension of deposited sediments is more efficient than increase in lateral sediment transfer, due to higher main channel transport capacity. Roughened and absorbing floodplains presented a constant evolution of the deposition with the stream-power over the floodplains, which comes along with an increased sediment concentration over the floodplains due to an enhanced lateral sediment transfer, especially when high velocity gradient is present in the interaction zone. "Vegetated" floodplains reduce drastically the velocities over the floodplains and optimize the sediment transfer towards the floodplains, but also the conditions for deposition. This indicates that when rivers invade their floodplains, great quantities of sediments could be

collected especially in these vegetated zones.

7.2.2 *Telemac-2D* deposition formation modelling

The laboratory campaign reports comprehensive experimental data which could be used to validate different numerical/theoretical models. A first use of the data is made to study floodplain deposition using the sedimentological module of *Telemac-2D: Sisyphé*. With hydrodynamic conditions well reproduced, the various experimental configurations are explored and the relevant numerical parameters that condition the sand transfer and deposition process are identified.

Solute study was again performed in all rigid and mobile bed geometries from the laboratory flume. Topographic singularities are shown to influence significantly the transfer process as they induce tracer plume irregularities due to light spatial depth variations, but also transverse flow. This effect could explain the spatial deposit distribution observed in the experiments, with globally more sediment deposited on the left floodplain.

Suspension transport is proven to be the dominant process at stake and the programmed dispersive characteristics of the flow follows the previous conclusions concerning tracer transport, depending on a scaling of the diffusion coefficient with subsection-averaged water depth and shear velocities. This hypothesis should be considered with great care, as sediment is recognized to behave rather differently from solute. Deposition acts as a concentration sink and the re-feeding process from the main channel increases the global sand transfer towards the floodplains.

Rigid bed experiments are most reliably modelled and present the critical mobility θ_{cr} parameter as the best way to characterize deposition and resuspension fluxes. Together with the bed shear τ_b (function of the roughness and the velocities), θ_{cr} controls erosion and deposition. Even if the spatial distribution lacks of realism due to the impossibility of the model to represent mobile deposition, modelled sediment deposition on the floodplains concurs with collected sediment mass during experiments, if however the θ_{cr} is correctly adapted to the floodplain surface conditions (micro-topography). Compared to smooth floodplains, rough floodplains require a noticeably higher θ_{cr} where the trapping ability of steel plates limits the resuspension flux.

While modelling mobile bed experiments, the critical mobility parameter proves its limited capacity to provide accurate deposition previsions, at least in its homogeneous form. Indeed, the theoretical higher energy requested to mobilise sediments in the main channel and its mobile bed into suspension than the energy level which forces deposition

over the floodplains is problematic. Modelling mobile bed experiments would be a further improvement to this thesis.

7.3 Perspectives

A number of questions were not answered in the course of this project. Maybe the most striking unresolved question deals with the relation between flow configurations (conditioned by relative depth, slope, discharge and subsectional roughnesses) and mixing capacity of the flow. Knight & Shiono (1996) insist on the strong dependence of the mixing mechanisms with relative depth and the need to investigate different geometrical configurations. Throughout this work, the intensity of lateral matter transfer was determined with a scaling of non-dimensional calibrated diffusion coefficients obtained with tracer injections. Diffusion-dispersion study was made with a given vortex/turbulent activity (for a given relative depth) and is hardly generalisable to further situations.

Respective role of turbulence and secondary currents on transport rate is rather complex. Due to a lack of knowledge over secondary circulations, the model used to compute lateral diffusion coefficients considered the migrations of solute introduced by the last as transport due to diffusion only. Considerable efforts could be made to improve the quality of the observations and of the analysis. Lower tracer concentrations could be used to avoid the downwards plunging of the plume due to density effects (Wood & Liang, 1989) and density driven secondary circulations (Ishigaki *et al.*, 2001) which could disrupt the flow field. Improvements will be brought to the analysis tool including three-dimensional velocity measurements (e.g. using PIV). Enhanced comprehension of which part of the transport is attributed to advection or turbulent diffusion is expected.

Different tendencies in numerical transport modelling exist. In the future, complex 3-D turbulent structures could be modelled explicitly with more complex turbulence models to avoid speculations on turbulent intensity, and where mixing coefficients contain always less implicit turbulent effects. Those developments request for validation a comprehensive amount of three-dimensional dye tracing data. On the other side, focus could persist on simplified two-dimensional simulations. Further investigation of turbulent dispersion leading to development of empirical correlations between transverse diffusivities and geometrical and flow parameters will be possible by carrying out new experiments, planning non-prismatic geometry, several other flume discharges and injection positions (e.g. Guymer *et al.*, 1998). Aware of the work that would be required, the proposed 2-D approach plans on a numerical study of mixing intensity, by isolating the turbulent

structures, as it is done here for vertical axis eddies which are undoubtedly responsible for a great proportion of increased lateral transport rate due to turbulence. Diffusion coefficients for numerical models should reliably represent the actual turbulent state of all given situations.

When sediment comes into play, experiments and their interpretation become even more complex. The sedimentological experiments over rigid and mobile beds allowed to highlight the conditions favourable to the formation of substantial floodplain deposition. They suffer however from the incapacity to isolate sediment transfer and thus the available sediment for deposition (C_{SL}) from other phenomena. Future experiments should focus on three-dimensional fine sediment concentration measurements in the main channel and on the floodplains, through the use of appropriate instruments, such as the FOS-LIM probe (Delft Hydraulics). Sediment diffusivities extracted from those concentration measurements would be also valuable, as solute behaves differently from suspended sediment. Further, the river power-balance equation, proposed by Verbanck (1995), suggests a thorough investigation of the floodplain suspension efficiency η_{SL} which is the key to interpretation of the threshold stream-power where resuspension becomes significant for smooth floodplain cases. It is further speculated that it could link those experiments to the roughened floodplain cases. Qualitative data is available for deposition formation on floodplains during the experiments, by use of digital imaging. A system has been developed for future experiments whose purpose is to measure the instantaneous deposit height on the floodplains at 2 longitudinal positions spaced by 50 cm and with a 7 points lateral resolution (GCM, developed by Delft hydraulics and Fraselle, 2009). If mathematical models reach one day the capacity to model instantaneous deposits precisely, those experiments could be of great help.

Two-dimensional sedimentological numerical simulations were runned with *Telemac-2D* and *Sisyphe* which struggled to produce plausible estimations of complex deposition situations. The main weaknesses of the numerical simulations were pointed out. First, depth-averaged suspended sediment concentrations are severe simplifications, as the equilibrium vertical concentration profile is inherently irregular (Rouse, 1937), and three-dimensional deposition modelling should be initiated fast. Two-dimensional numerical model is not able to model bedforms, as bottom turbulence is not explicitly integrated in depth-averaged computations. Further, it proved its inability to model mobile deposition on the floodplains and rolling and gliding sand grains above the main channel bed, as no equivalence was found in the chosen mathematical representation. *Suspension only* with

rigid bed makes the bottom evolution easily predictable, with a net deposition flux in zones where local bed shear stress decrease, but combined action of bed load and suspension transport was not efficient. Indeed, no bed load transport law is adapted to the sediments used in this work. Mobile bed experiments models were attempted, but the numerical software reached its limitations hereby and suggests a spatially variable mobility threshold θ_{cr} . An efficient coupled bed load and suspended load approach, combined with an improved treatment of erosion and deposition fluxes, where suspension efficiency η_{SL} finds in a certain way a mathematical representation, is the key to performant two-dimensional modelling which integrate the possibility of moving deposits together with the main currents.

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Appendix A

Notation

Roman symbols

a	(m)	reference near sand-bed level
A	(m^2)	section area
B	(m)	channel width
c	$(-)$	solute concentration in water
c	(m^3/m^3)	volumetric concentration of suspended sediment
C	(g/l)	tracer/solute concentration
C	(g/l)	depth-averaged solute concentration (<i>telemac</i>)
C	(m^3/m^3)	depth-averaged sediment concentration (<i>telemac</i>)
C	$(m^{1/2}/s)$	Chezy coefficient
C_*	$(m^{1/2}/s)$	dimensionless Chezy coefficient
c_a	(m^3/m^3)	reference near-bed sand concentration
c_a^*	(m^3/m^3)	equilibrium near-bed sand concentration
C_f	$(-)$	friction coefficient
C_s	(g/l)	sand concentration
C_t	(Kg/Kg)	sediment concentration by weight
C_s	$(-)$	sand concentration
C_v	$(-)$	dimensionless mixing coefficient
C_{SL}	$(-)$	suspended particle matter concentration
d	(m)	sediment grain diameter
d_{50}	(m)	sand median diameter
d_*	$(-)$	particle parameter
D_b	(m/s)	sediment deposition rate per unit bed area
D_y	(m^2/s)	lateral mixing coefficient
D_z	(m^2/s)	vertical mixing coefficient
E_b	(m/s)	sediment erosion rate per unit bed area

E_s	$(-)$	entrainment of bed sediment into suspension
f	$(-)$	Darcy-Weisbach resistance coefficient
f_φ	$((-)/s)$	transport of φ by diffusion and sources or sink
Fr	$(-)$	Froude number
g	(m/s^2)	gravity constant
G_s	(g/s)	sediment discharge (in mass)
h	(m)	water depth
h_{fp}	(m)	floodplain water depth
h_{mc}	(m)	main channel water depth
h_r	$(-)$	relative depth
k	(m^2/s^2)	turbulent kinetic energy
k_s	(m)	equivalent wall surface roughness
K	(m^3/s)	section conveyance
$\mathbf{K}$	$(-)$	relative roughness
l_m	(m)	Prandtl's mixing length
L_d	(m)	turbulence length scale
m	(g/l)	(solute) mass flow rate
M_s	(Kg)	deposited sediment mass on floodplain
n	$(s/m^{1/3})$	Manning roughness coefficient
n	$(-)$	bed porosity
n_b	$(s/m^{1/3})$	Manning roughness for the sand bed
$\mathbf{n}$	$(-)$	unit vector orthogonal to surface S
p	$(Kg/m\ s^2)$	pressure
p	$(-)$	deposition probability
P	(m)	wetted perimeter
q	(m^2/s)	discharge per unit width
q_b	(m^2/s)	bed load transport rate per unit width
q_s	(m^2/s)	suspended load transport rate per unit width
q_t	(m^2/s)	total load transport rate per unit width
q_t^*	$(-)$	dimensionless sediment discharge
q_φ	$((-)/s)$	source of φ
Q	(m^3/s)	section discharge
Q_{mc}	(m^3/s)	main channel discharge
Q_{fp}	(m^3/s)	floodplain discharge
$\mathbf{Q}_b$	(m^3/s)	bed load transport rate vector

$\mathbf{Q}_s$	(m^3/s)	suspended load transport rate vector
$\mathbf{Q}_t$	(m^3/s)	total load transport rate vector
R	(m)	cross-section hydraulic radius
R_s	(m)	hydraulic radius due to skin friction
R_f	(m)	hydraulic radius due to form drag
R_b	(m)	hydraulic radius for the sand bed
Re	$(-)$	Reynolds number
Re_*	$(-)$	shear Reynolds number
s	$(-)$	sand relative density
S	(m^2)	surface
S	$(-)$	slope
S_c	$(g/l\ s)$	depth averaged source of tracer (<i>telemac</i>)
S_f	$(-)$	friction slope
S_0	$(-)$	channel bed slope
t	(s)	time
T	$(^\circ)$	temperature
u	(m/s)	streamwise velocity
u_i	(m/s)	velocity components in direction x_i
u_*	(m/s)	bed shear velocity
$\mathbf{u}$	(m/s)	velocity vector
U	(m/s)	time- and depth-averaged streamwise velocity
U	(m/s)	mean section velocity (1-D)
U_c	(m/s)	critical mean flow velocity for initiation of motion
$\mathbf{U}$	(m/s)	time- and depth-averaged velocity vector
v	(m/s)	transverse velocity
V	(m/s)	time- and depth-averaged transverse velocity
w	(m/s)	spanwise velocity
w_s	(m/s)	sediment fall velocity
x	(m)	streamwise coordinate
y	(m)	transverse coordinate
z	(m)	spanwise coordinate
z_b	(m)	bottom elevation
z_0	(m)	bed roughness length
z_w	(m)	free surface elevation
z_*	$(-)$	Rouse number

Greek symbols

α	(—)	temperature dependent variable
α_c	(—)	adaptation coefficient
α_L, α_l	(—)	longitudinal and lateral Elder coefficients
χ	($\mu S/cm$)	water conductivity
Δ	(m)	bed form wave height
ϵ	(m^2/s^3)	turbulent kinetic energy dissipation rate
ϵ_m	(m^2/s)	molecular diffusivity
ϵ_i	(m^2/s)	turbulent/eddy diffusivity in direction x_i
ϵ_s	(m^2/s)	sediment diffusivity
ϵ_t	(m^2/s)	eddy diffusivity
$\epsilon_{s,i}$	(m^2/s)	eddy diffusivity for sand particles in direction x_i
φ	(—)	general property
γ	($Kg/m^2 s^2$)	water specific weight
γ_s	($Kg/m^2 s^2$)	grain specific weight
Γ	($Kg m^2 (-) / m^3 s$)	diffusivity for φ
η_{SL}	(—)	suspension efficiency
κ	(—)	Von Karman constant
λ	(m)	bed form wave length
λ	(—)	turbulence proportionality constant
θ_{cr}	(—)	critical shields/mobility parameter
θ_s	(—)	mobility parameter
ν	(m^2/s)	molecular viscosity
ν_t	(m^2/s)	turbulent/eddy viscosity
ν_T	(m^2/s)	tracer turbulent diffusion coefficient tracer (<i>telemac</i>)
ρ	(Kg/m^3)	fluid density
ρ_s	(Kg/m^3)	sand density
σ_t	(—)	Schmidt number
$\sigma_{t,z}$	(—)	proportionality for sand diffusivity and eddy viscosity
τ	($Kg/m s^2$)	shear stress
τ_b	($Kg/m s^2$)	bed shear stress
τ_{ij}	($Kg/m s^2$)	Reynolds (turbulent) stresses

τ_*	$(-)$	dimensionless shear stress
ω	(Kg/s^3)	stream power per unit length and unit width
Ω	(m^3)	volume

Abbreviations

2-D	two-dimensional
3-D	three-dimensional
CM	control mass
CV	control volume
EDM	Exchange Discharge Model
EPS	expanded polystyrene
FFT	Fast Fourier Transform
LDA	Laser Doppler Anemometer
LIF	Laser Induced Fluorescence
MBS	Mobile Bed Smooth floodplains
MBR	Mobile Bed Rough floodplains
MBV	Mobile Bed Vegetated floodplains
PIV	Particle Image Velocimetry
PTV	Particle Tracking Velocimetry
RANS	Reynolds Averaged Navier Stokes
RBS	Rigid Bed Smooth floodplains
RBR	Rigid Bed Rough floodplains

Appendix B

Simplified sediment transport and deposit modelling

Introduction

Sediment transport modelling may sustain different approximation levels. Keeping in mind the omitted features is of primary matter since simplified models contain hypothesis which make them applicable to very particular situations. In this chapter, numerical models of suspended sediment transport and deposit will be developed using a finite difference discretization method of simplified transport equations. The aim of the work is to analyse vertical and lateral diffusion coefficients in compound or rectangular channel flumes and experiment boundary conditions which enable us to judge deposition of fine sediments. In addition, it will provide user-friendly codes which are easily used for data analysis during experimental campaigns.

The adopted procedure is as follows : first, a longitudinal transport and deposit model (2-D: X-Z plane) is developed, following the method employed by Sharpe (2003). By nature, this model is restricted to rectangular channels and could be used to analyze simplified fine sediment transport experiments. Secondly, a transverse transport and deposit model over the floodplains (2-D: Y-Z Plane), based on the work by James (1985), is presented. The model is used to predict deposition areas over floodplains using empirical diffusion coefficients in prismatic rectangular compound channels. Finally a quasi-3D (decoupled) transport model is proposed. This model merges longitudinal and transverse suspended sediment transport models previously developed. This work was motivated by the fact that deposit distribution in compound channels obviously depends on the three-

dimensional transport of suspended material. Instead of using a fully developed 3-D mesh, the proposed model would be able to appreciate deposits for very long river reaches since low computational cost is required due to its decoupled nature. The development of this model will stay as a perspective, as no results are presented.

All developments are based on the equation developed in section 3.2.2. The distribution of sediment concentration (C) in suspended transport may be described by the diffusion-convection equation (Graf, 1971):

$$\frac{\partial C}{\partial t} = -u_i \frac{\partial C}{\partial x_i} + \frac{\partial}{\partial x_i} \left(\varepsilon_i \frac{\partial C}{\partial x_i} \right) \quad (\text{B.1})$$

where t is the time, u_i are velocity components in direction x_i and ε_i are the sediment diffusivities in the corresponding directions.

Longitudinal transport

Hypothesis

The diffusion-convection equation was used to develop a 2-D suspended sediment transport longitudinal model in uniform flow conditions. The transverse (y) components in Eq. (B.1) are ignored which restricts the solution to rectangular channels with uniform lateral mixing. The sediment is assumed to move downwards in the vertical (z) direction at its settling velocity ($w = -w_s$), and longitudinal diffusion is assumed to be negligible compared with convection ($\varepsilon_x \approx 0$). For steady state conditions the change in concentration with time is zero ($\frac{\partial}{\partial t} = 0$). Introducing those hypothesis into Eq.(B.1) gives the two-dimensional transport equation for suspended sediment under steady state conditions on a vertical-longitudinal plane, i.e.:

$$u \frac{\partial C}{\partial x} = w_s \frac{\partial C}{\partial z} + \frac{\partial}{\partial z} \left(\varepsilon_z \frac{\partial C}{\partial z} \right) \quad (\text{B.2})$$

The longitudinal distribution of suspended particles in a channel is thus described by a hyperbolic-parabolic partial differential equation in the longitudinal direction.

The boundary condition at the free surface states that there is no net transport across the surface and can be written (Sharpe, 2003):

$$\varepsilon_z \frac{\partial C}{\partial z} + w_s C = 0 \quad (\text{B.3})$$

the above expression may be simplified further assuming the vertical sediment diffusivity to be related to the conventional parabolic vertical distribution of eddy viscosity (Graf,

1971)(see chapter 3 and Eq.(3.49)). The diffusivity at the surface is zero and Eq.(B.3) shows that the concentration at the surface reads also zero and will therefore be taken as so.

Near the bed surface, where suspended transport begins ($z = a$), the rate of deposition at any location on the bed can be described by imposing the boundary condition (James, 1984):

$$\varepsilon_z \frac{\partial C}{\partial z} + (1 - p) w_s C = 0 \quad (\text{B.4})$$

where p is the probability that a particle reaching the bed will be permanently deposited. The probability of deposition can be interpreted as the proportion of particles reaching the bed that will be deposited and depends on the characteristics of the plain surface (bed roughness), the particles and the flow. This type of erosion-deposition criteria assumes no entrainment from the bed and must be applied carefully in case of experimental validation since part of the physical phenomena is hereby shrouded. This probability can be estimated by using formulations applied in sediment transport models, such as those of Engelund & Fredsoe (1976) or Einstein (1950).

Solutions to Eq. (B.2) provide predictions of sediment concentrations and deposition distribution patterns and requires knowledge of vertical variation of longitudinal velocity (u) and sediment diffusivity values (ε_z). As mentioned above, a parabolic diffusivity profile is used which implies a logarithmic vertical velocity profile (Chang, 1988):

$$\frac{u}{u_*} = C_{st} + 2.5 \ln \frac{z}{k_s} \quad (\text{B.5})$$

and k_s is known as *Nikuradse's sand roughness* (mean diameter of sand used by Nikuradse). The constant C_{st} was adjusted so that the average velocity would fall at a height 0.4 of the flow depth (due to logarithmic velocity profile).

Finite difference development

Equation (B.2) is solved numerically using a finite difference discretization method. The finite-difference mesh used is depicted on Figure B.1 with N_z nodes on the vertical and N_x nodes defining the horizontal increments. The grid spacing is denoted by Δz in the vertical direction and Δx in the longitudinal direction.

The following finite difference expressions are used for approximation of derivatives in the vertical direction:

$$w_s \frac{\partial C}{\partial z} \Rightarrow w_s \frac{C_{k+1} - C_k}{\Delta z} \quad (\text{B.6})$$

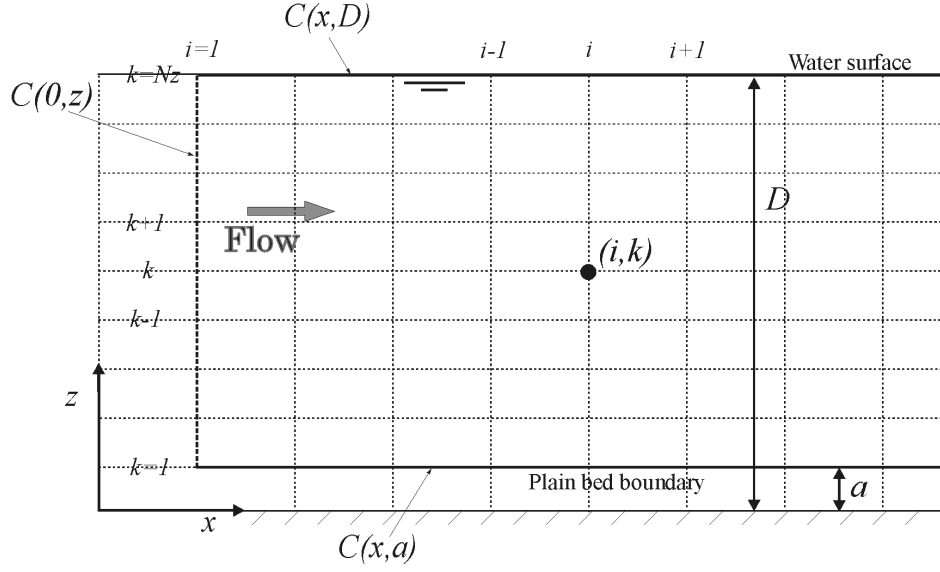


Figure B.1: Finite difference mesh for longitudinal transport model

$$\frac{\partial}{\partial z} \left(\varepsilon_z \frac{\partial C}{\partial z} \right) \Rightarrow \frac{\varepsilon_{k+1/2} (C_{k+1} - C_k) - \varepsilon_{k-1/2} (C_k - C_{k-1})}{\Delta z^2} \quad (\text{B.7})$$

with

$$\varepsilon_{k+1/2} = \frac{\varepsilon_{k+1} + \varepsilon_k}{2} \quad (\text{B.8})$$

$$\varepsilon_{k-1/2} = \frac{\varepsilon_k + \varepsilon_{k-1}}{2} \quad (\text{B.9})$$

The convective term is discretized with an upwind scheme while the diffusion term is discretized using central differences for well-known stability matters (see e.g. Versteeg & Malalasekera, 1995).

The discretization of the longitudinal term is more problematic, since Sharpe (2003) uses an explicit scheme inadequate to this type equation (hyperbolic-parabolic). To avoid stability problems, a more appropriate Crank-Nicholson scheme (see e.g. Hirsch, 1988) was chosen to reach an unconditionally stable scheme and the fully discretized equation finally reads:

$$u_{i+1/2,k} \frac{C_{i+1,k} - C_{i,k}}{\Delta x} = \vartheta (\dots)^{i+1} + (1 - \vartheta) (\dots)^i \quad \text{with } \vartheta \geq 1/2 \quad (\text{B.10})$$

The expression under brackets stands for vertical discretization terms. Solving equation (B.10) implies the resolution of a *tridiagonal* system every progression in the x direction (i index). The TDMA algorithm (tridiagonal matrix algorithm or Thomas algorithm) is used to find the solution to the set of linear equations (Press *et al.*, 2007). Initial

starting concentrations at $x = 0$ are required to begin the computations. Two initial concentration profiles were used : a point source at Δz from water surface or a constant initial concentration at each z .

Deposition profiles

The longitudinal model is applied to a hypothetical channel to identify the factors which have the greatest influence on longitudinal distributions. In particular, the influence of the deposition probability will be demonstrated. The channel hydraulic parameters were chosen to fit the observations of Sharpe (2003). Nikuradse's roughness parameter was taken as $k_s = 0.005$ m, while $a = 0.05D$. The average velocity was calculated using Manning's n taken as that measured in the flume study. The shear velocity was calculated through $u_* = \sqrt{gR_bS_0}$. R_b is the hydraulic radius (equivalent here to the water depth since an infinitely large channel is considered) and S_0 is the slope. In all coming sections, the deposition at any location is taken as that which would settle out from the concentration at $z = a$, providing a sediment mass flux per unit surface, i.e.:

$$\text{Deposition}(x) = w_s C_{a,x} \quad (kg/m^2s) \quad (B.11)$$

Influence of deposition probability p

The objective of this section is to show the influence of the deposition probability (p) on deposition profiles. A fully absorbing surface, physically represented by a rubber matting in James (1985), may be modelled using $p = 1.0$ which expresses that all sediment particles reaching the bed will deposit. The resulting qualitative concentration profile for sediment injection near water surface and resulting deposition profiles are depicted on Figure B.2. The sediment used was $d_{50} = 91 \mu\text{m}$, the slope was set to $S_0 = 0.001873$ and the flow depth reached $D = 61.7$ mm.

Using the same parameter settings, but with a fully reflective surface ($p = 0$), e.g. steel plates covering the bottom of the flume, the deposition follows a different trend, visible on Figure B.3.

As expected, the absorbing boundary expresses mass loss out of the integration domain which explains lower deposition rates for the $p = 1.0$ case and decreasing concentrations downstream. Moreover, the deposition will be maximum when the convection current, a combination of longitudinal water velocity and sediment downward fall velocity, reaches the bottom. On the other side, a reflective boundary ($p = 0$) (e.g. smooth, steel surface)

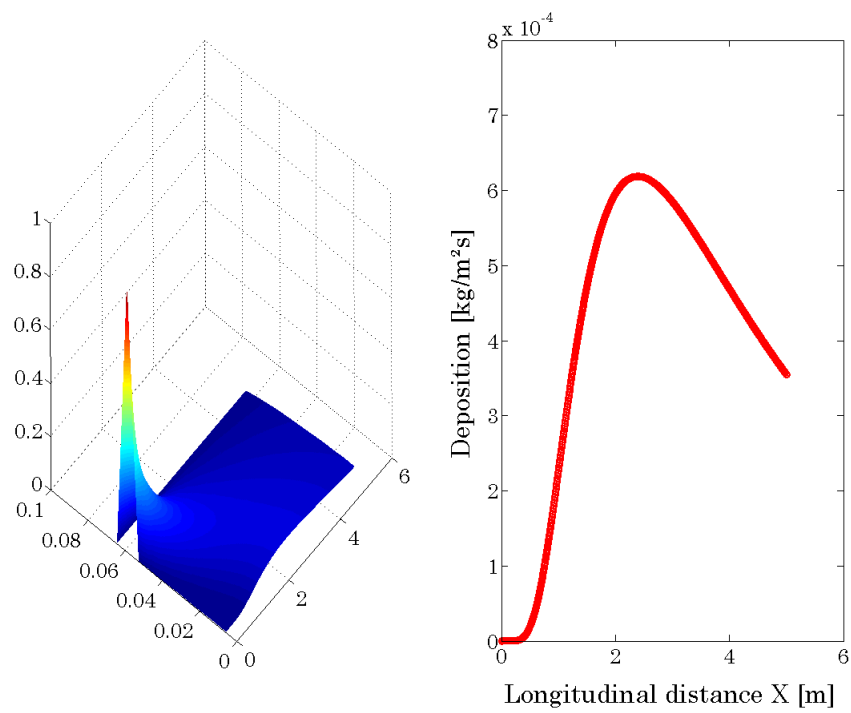


Figure B.2: *Qualitative sediment concentration in a 5 m long flume with $d_{50} = 91\mu\text{m}$ sand when injection near the surface with $p = 1.0$ (left) and resulting deposition profiles on the bed (right).*

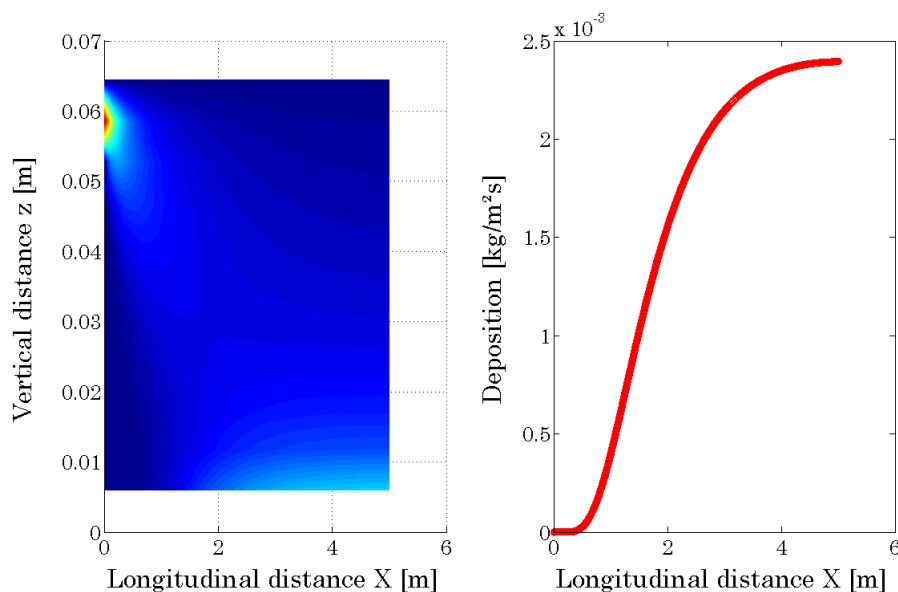


Figure B.3: *Qualitative sediment concentration in a 5 m long flume with $d_{50} = 91\mu\text{m}$ sand when injection near the surface with $p = 0.0$ (left) and resulting deposition profiles on the bed (right).*

leads to sediment accumulation near the bottom submitted to longitudinal convection. Quantitative deposits in terms of mass flux per unit area on the bottom should not be brought much attention. The deposition should then more be taken in a relative sense, expressing an increase towards downstream end of flume and a progressive stabilisation of the vertical concentration profile due to equilibrium between the sediment's fall velocity and vertical diffusion stresses. To conclude, the experimenter can thus await a strong influence on deposition distribution following the type of surface covering his physical model (James, 1985).

Lateral 2D vertical transport

Model hypothesis

The distribution of particles in suspension over a floodplain is described by the following two-dimensional elliptic partial differential equation which includes terms for diffusion

and convection in the vertical and transverse directions:

$$0 = w_s \frac{\partial C}{\partial z} - v \frac{\partial C}{\partial y} + \frac{\partial}{\partial y} \left(\varepsilon_y \frac{\partial C}{\partial y} \right) + \frac{\partial}{\partial z} \left(\varepsilon_z \frac{\partial C}{\partial z} \right) \quad \text{for} \quad \begin{cases} 0 \leq z \leq (D - h) \\ 0 \leq y \leq b \end{cases} \quad (\text{B.12})$$

To obtain equation (B.12), longitudinal (x) components were ignored in equation (B.1). For steady, uniform flow $\partial C / \partial x = 0$ may be accepted for the compound channel section as a whole, provided erosion is equal to deposition. For the plain region, the strong hypothesis resides in assuming that the rates of deposition, erosion and supply from the main channel are constant over relatively short distances and $\partial C / \partial x = 0$. A possible lateral convection current (v) normal to the channel above the plain level was conscientiously left in case of skewed main channel (not parallel to the steepest gradient). Again, for steady state conditions, variations of concentrations with time are zero ($\frac{\partial}{\partial t} = 0$). Finally, the model has no other choice than to implicitly include in the transverse diffusivity ε_y the effects resulting from secondary circulations in compound channels (see chapter 2).

Numerical integration of this equation in the flow region above the plain, showed on Figure B.4 gives the distribution of sediment by means of definition of appropriate boundary conditions on all domain boundaries.

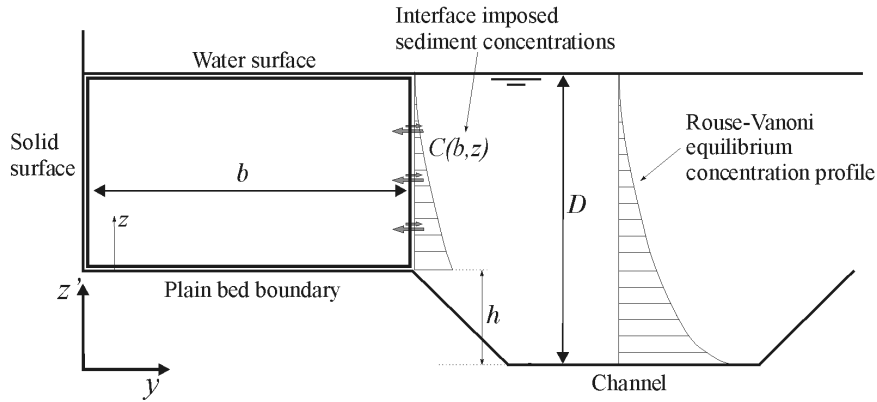


Figure B.4: *Domain of integration for lateral transport model.*

The boundary conditions at the surface and the bed are the same as for longitudinal transport case whereas the solid vertical boundary is simplified to null gradient condition (James, 1985):

$$\varepsilon_y \frac{\partial C}{\partial y} = 0 \quad (\text{B.13})$$

The main channel-floodplain interface boundary condition is defined in terms of concentration values calculated by a theoretical solution of a one-dimensional diffusion-settling equation applied to sediment in the main channel (Rouse, 1937). The vertical distribution of sediment concentration at equilibrium is given by:

$$\frac{C}{C_a} = \left(\frac{D-z}{z} \frac{a}{D-a} \right)^{w_s/\kappa u_*} \quad (\text{B.14})$$

where

C_a is a reference concentration at height a above the bed

u_* is the shear velocity

κ is the Von Karman constant

D is the main channel water depth

C_a values may be calibrated on basis of depth-averaged concentration values. In this section however, the model is used to predict relative concentrations and deposits, which implies arbitrary values for C_a , set to 100.0.

Solution of the transverse distribution equation requires estimates of turbulent diffusivities and particle velocities in both the transverse and vertical directions. The considered longitudinally linear geometry implies zero transverse velocity ($v = 0$). Sediment grain size determined their settling velocities taken as downward vertical convection current (w_s).

Turbulent diffusivities for sediment are related to momentum diffusivities and can be defined through empirical relationships using flow geometry parameters, all of those expressions are found in the work of James (1985). Basically, vertical diffusivity for sediment is given by equation (3.49), with a local, y dependent, shear velocity ($u_*(y)$) given over the floodplains by Rajaratnam and Ahmadi (1981). The shear velocity u_* is related to the bed shear stress τ_b by the equation:

$$u_* = \sqrt{\frac{\tau_b}{\rho}} \quad (\text{B.15})$$

and the transfer of momentum from the main channel to the plain increases the bed shear stress on the plain surface in the interaction zone (see below):

$$\tau_b = \tau_{fp} + (\tau'_b - \tau_{fp}) e^{-0.693(y-b/b_\tau)^2} \quad (\text{B.16})$$

with

τ_{fp} is the undisturbed bed shear stress on the plain ($\tau_{fp} = \gamma R_{fp} S_0$)

y is the distance over the plain measured from the side wall

τ'_b is the bed shear stress at the interface $= \tau_{fp} (1 + 0.24 (D / (D - h) - 1))$

$b_\tau = 0.64 (D - h) (D / (D - h) - 1)$ is a length scale

$2b_\tau$ is defined as the width of the interaction zone

h is the floodplains height

Applying Eq.(B.16) to the calculation of (B.15) gives the tranverse distribution of shear velocity over the floodplains which is depicted for $D = 75$ mm and $h = 50$ mm on Figure B.5.

Transverse diffusivity within the interaction zone is calculated by an empirical equation

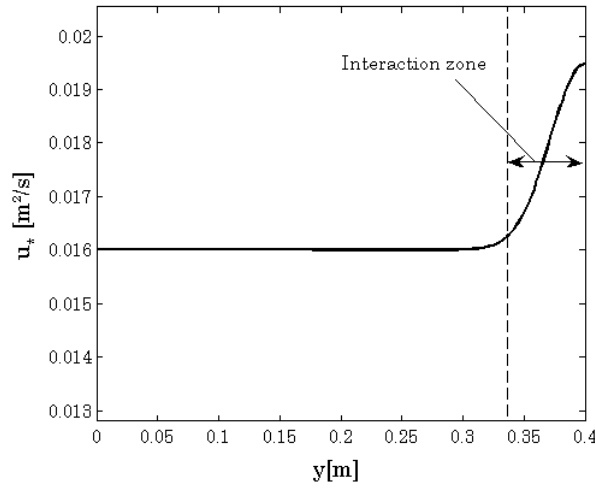


Figure B.5: *Lateral variations of shear velocity over the floodplains according to Rajaratnam and Ahmadi (1981) with $D = 75$ mm and $h = 50$ mm.*

derived by Rajaratnam and Ahmadi (1981):

$$\frac{\varepsilon_y}{(u_{mc} - u_{fp})(D - h)} = \frac{0.3 (D / (D - h) - 1)^3}{((u_{mc} - u_{fp}) / u_{*,fp})^2} A(\eta) \quad \text{with } \eta = y / b_\tau \quad (\text{B.17})$$

in which u_{mc} and u_{fp} are the undisturbed average flow velocities for the main channel and the floodplain calculated using Manning's equation, $u_{*,fp}$ is the undisturbed shear velocity on the plain and $A(\eta)$ is an experimentally derived function.

Beyond the interaction zone, the transverse diffusivity is estimated using equation (B.18) derived from data obtained by Lau and Krishnappan (1977):

$$\frac{\varepsilon_y}{u_* W} = 0.001 (32.77 - 8.03 \ln (W / (D - h))) \quad (\text{B.18})$$

with W the flow width, assumed to be the width beyond the interaction zone $y < (b - 2b_\tau)$. If at anytime of the calculation mentioned above, the lateral diffusivity goes lower than an empirical value prescribed for rectangular channels by Rutherford (1994), then, the mixing coefficient is set to this minimum value:

$$\varepsilon_y = \frac{0.1 + 0.26}{2} (D - h) u_{*,fp} \quad (\text{B.19})$$

Numerical development

As for the longitudinal model, a finite difference grid is defined, represented on Figure B.6. The domain of integration is divided into N_y equal horizontal increments and N_z

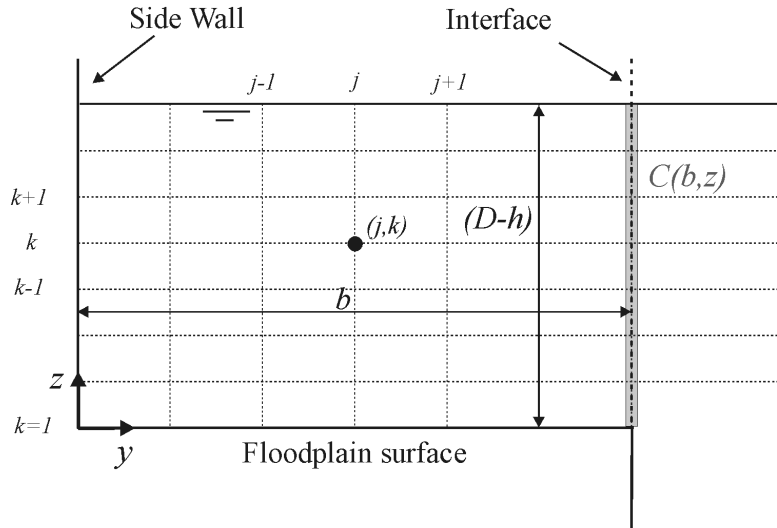


Figure B.6: *Finite difference mesh for lateral transport model over the floodplains.*

vertical increments. The grid spacing is Δy in the transverse direction and Δz in the vertical direction.

Exactly the same finite difference expressions than for the longitudinal model are used for approximation of the vertical convection term (B.6) and diffusion term (B.7).

The forms of the finite difference approximations for the horizontal direction are obtained likewise, except that for the transverse convection component the approximation is (considering $v > 0$):

$$v \frac{\partial C}{\partial u} \Rightarrow v \frac{C_j - C_{j-1}}{\Delta z} \quad (\text{B.20})$$

The elliptic nature of equation (B.12) requires boundary conditions all around the computation domain and iterative resolution of the system of simultaneous equations resulting from applying appropriate finite difference equations to all grid points. The solution to the concentration at a node (j, k) is dependent on the concentrations at each surrounding that point and is given by:

$$\begin{aligned} & \left(\frac{w_s}{\Delta z} + \frac{\varepsilon_{z,k+1/2}}{\Delta z^2} \right) C_{j,k+1} + \\ & + \left(\frac{\varepsilon_{y,j+1/2}}{\Delta y^2} \right) C_{j+1,k} + \left(\frac{\varepsilon_{z,k-1/2}}{\Delta z^2} \right) C_{j,k-1} + \left(\frac{\varepsilon_{y,j-1/2}}{\Delta y^2} + \frac{v}{\Delta y} \right) C_{j-1,k} - \\ & - \left(\frac{\varepsilon_{y,j+1/2} + \varepsilon_{y,j-1/2}}{\Delta y^2} + \frac{\varepsilon_{z,k+1/2} + \varepsilon_{z,k-1/2}}{\Delta z^2} + \frac{w_s}{\Delta z} + \frac{v}{\Delta y} \right) C_{j,k} = 0 \quad (\text{B.21}) \end{aligned}$$

The calculations of the transport components and boundary conditions are described in detail by James (1985). The model has been coded in MATLAB for solution by computer. Input data describe the channel system, flow and sediment characteristics and the finite difference grid dimensions. The channel system is defined by the shape, size and direction of the channel, the width of the plain, the longitudinal gradient and resistance coefficients. The sediment is described by particle size, fall velocity and relative density. A reference concentration near the main channel bed (C_a) is also specified (in arbitrary units or not).

Deposition profiles over the floodplains

Lateral fine sediment transport and deposit profiles are modelled over the floodplains under permanent flow conditions with equilibrium sediment profile in the main channel given by Rouse's equation. Instead of a reference concentration near the bottom, a depth averaged concentration is employed and the vertical distribution is computed using the equation. The model was applied to a prismatic compound channel with dimensions and conditions chosen to reproduce flow conditions in the UCL compound channel flume (see Figure B.7). The slope was set to $S_0 = 0.00111$. The sediment considered had a particle size of $91\mu\text{m}$ and a fall velocity of $w_s = 0.00743 \text{ m/s}$, and a relative density of 2.65. Resistance coefficients were assumed constant all over the compound section and $n = 0.0105$. A suitable finite difference grid was selected by progressively decreasing the spacing until further decrease had negligible effects on the results. In the absence of a transverse convective transport component, distributions on both floodplains were symmetrical and deposits are shown for one side of the channel only.

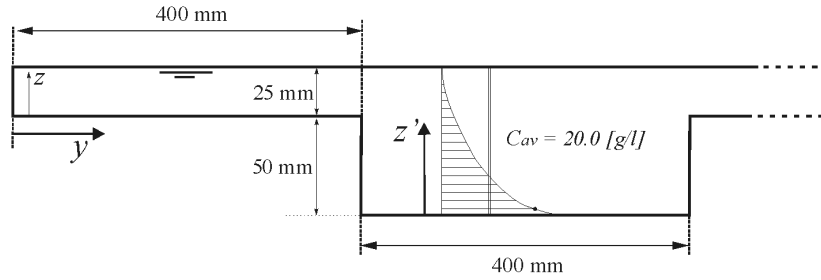


Figure B.7: Channel section for lateral model run.

The following Figures B.8 and B.9 shows 2-D concentrations on the left floodplain and corresponding deposition profile for deposition probability assumed equal to 1.0. In comparison, Figures B.10 and B.11 show the results for $p = 0.0$. The shapes of distribu-

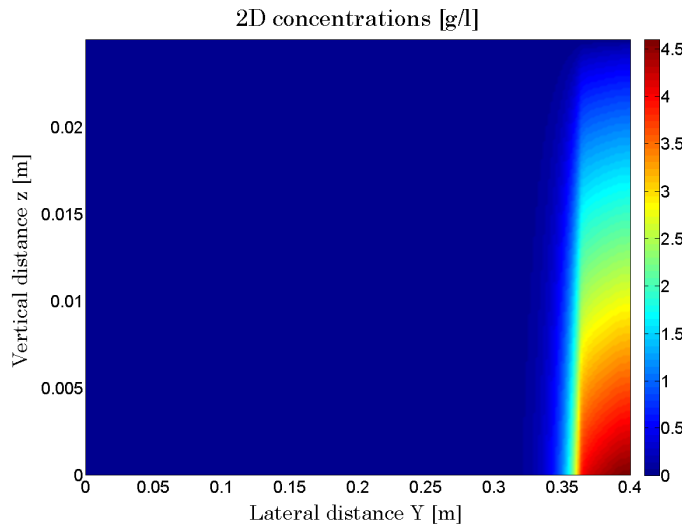


Figure B.8: 2D concentrations over floodplains for $p = 0.0$

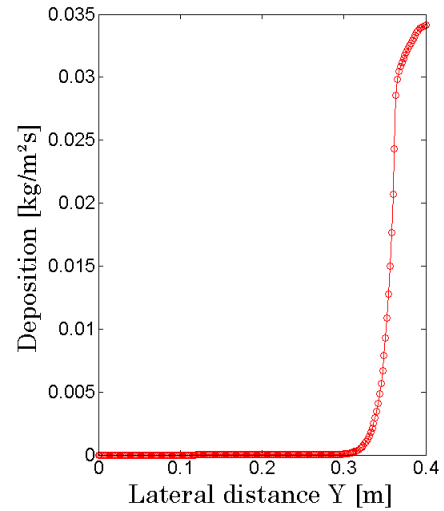


Figure B.9: Deposition profile over floodplains for $p = 0.0$

tions for absorbing and non-absorbing surfaces are proven to differ. Peak shifting away from the interface for low deposition probability occurs because the inhibited deposition results in high suspended concentration near the bed and concentration gradients close to the interface which are directed towards the main channel. Those gradients near the bed and interface will thus transfer sediment back towards the channel, shifting the position of the maximum concentration and hence the highest deposition.

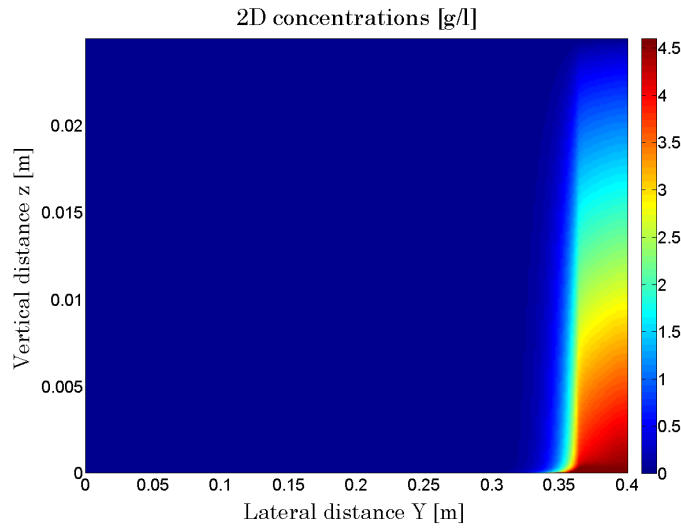


Figure B.10: *2D concentrations over floodplains for $p = 1.0$*

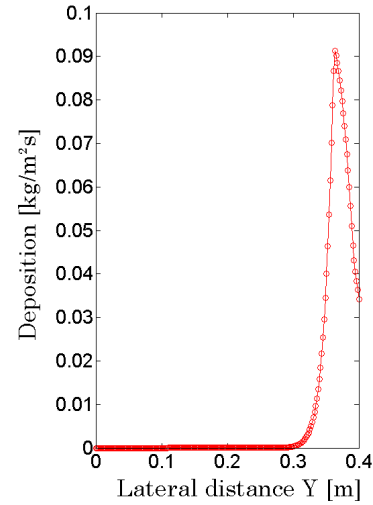


Figure B.11: *Deposition profile over floodplains for $p = 1.0$*

Quasi-3D (decoupled) modelling

Motivation and model description

Overbank transverse deposition and along the length of a channel are both determined by the transfer of suspended material from the channel to the plain and both depend on the vertical distribution of suspended material. The distribution of deposit in compound channel system therefore depends on the distribution of suspended material in three dimensions during transport. However, scales of variations of deposits are much more gradual longitudinally than transversely. Because of the level of detail required to describe the transverse distribution, a three dimensional model capable of describing the relatively gradual longitudinal variations of deposits as well would be extremely large and the computational effort required for its solution would be prohibitive for common calculators. The 3-D problem will therefore be decomposed and solved using two 2-D models.

The three-dimensional distribution of fine sediment concentration and deposit in a compound channel system is modelled using both previously developed models. It merges transverse transport over the floodplains and longitudinal transport in the main channel. Loss of sediment towards the floodplains and through deposit formation is accounted for in the longitudinal model. This first model (model A) describes the longitudinal and vertical sediment distribution in the main channel, and is an extension of the longitudinal model

already developed, showing additional terms which stand for sediment flux (convective or diffusive) from the main channel towards the floodplains. This model will be developed further in the following sections. The second model (model B) describes the transverse and vertical distributions across the floodplains and all details of the model is found in the previous section. Care should be taken however to the fact that the finite difference discretization of the transport equation will differ in case of positive or negative transverse convective currents ($v > 0$ or $v < 0$).

The rate of transfer to the plain depends on transverse concentration gradients which are determined by model B. Model A uses this output from the first to determine longitudinal concentration variations. The transverse model is applied at intervals determined by accuracy requirements ; concentration gradients are scaled at intermediate sections to avoid excessive approximations. Table B.1 gives a summarization of both models. Once again, steady state and uniform flow conditions are assumed. Longitudinal diffusion is once again neglected in the main channel and $\frac{\partial C}{\partial x} = 0$ over the plains, which implies that sediment concentration will not vary with longitudinal distance over short distances. This condition makes the model incapable of describing sediment diffusion plume over the floodplains.

Table B.1: *2D models description*

Model A (longitudinal domain)	Model B (transverse domain)
- Applied in main channel	- Applied over floodplains
- Vertical and longitudinal distribution of suspended sediment concentration	- Vertical and transverse distribution of suspended sediment concentration
- Sediments subject to deposition and transfer towards the plain	- Deposition of particles on the plain surface

Computation sequence

Figure B.12 gives an explanation of the computation sequence, the interactions existing between both models and required input and provided outputs of the model. For simulation of the distribution of sediment deposits the channel is divided into a number of reaches (X). First, the transverse distribution model (Model B) is applied at the beginning of each reach (with $C_1(z)$) and provides the distribution of deposits over the plain (A) and concentration gradients across the plain-channel interfaces (B). Second, these

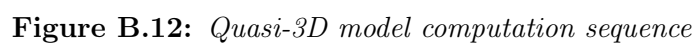


Figure B.12: *Quasi-3D model computation sequence*

concentration gradients are then used in the longitudinal distribution model (model A) in determining the variations of sediment concentration ($C_2(z)$) and the deposition along the reach (C). At the end of the reach the computed concentration profile is used as input to the transverse distribution model for calculations at the beginning of the following reach.

Longitudinal transport in river system

The longitudinal transport model describes sediment concentration distribution in the main channel of the river system with sediment submitted to deposition and transfer to the plain. Sediment concentrations are uniformly distributed across the width (perfect lateral mixing is assumed). Equation (B.22) describes the sediment distribution in the main channel:

$$u \frac{\partial C}{\partial x} = w_s \frac{\partial C}{\partial z} + \frac{\partial}{\partial z} \left(\varepsilon_z \frac{\partial C}{\partial z} \right) - v \frac{\partial C}{\partial y} + \frac{\partial}{\partial y} \left(\varepsilon_y \frac{\partial C}{\partial y} \right) \quad (\text{B.22})$$

The first three terms are sufficient to describe the longitudinal distribution of sediment in a channel with a simple cross section (see equation B.2). The last two terms account for the transverse movement of sediment across the interface between deep and shallow regions of a compound section. These two terms can be evaluated by applying the transverse model (model B) and are supplied as input to the model. They may therefore be treated as constants in the formulation of the longitudinal model. Equation (B.22) is thus in hyperbolic/parabolic form and can be treated as previously.

The equation is solved using the finite difference approach, and besides the 2D vertical-longitudinal discretization, a finite difference grid must also be specified in the transverse plane so that the transverse components can be specified. Points on both sides of the channel can be represented by the same grid points since uniformly distributed concentrations across the width are assumed. This grid, along with transverse fluxes definitions through a fluid element, are sketched on Figure B.13. Those convective and diffusive fluxes found their expression in the work made by James (1984). The finite difference equation can be formulated applying a mass balance of sediment on a fluid element. Equation (B.23) lists the net mass entering an interior element from all directions which equals zero for a steady process. The element has dimensions Δy , Δx and $(\Delta z + B)$, where B is the channel width. Transport components in the x direction are left undeveloped, since their

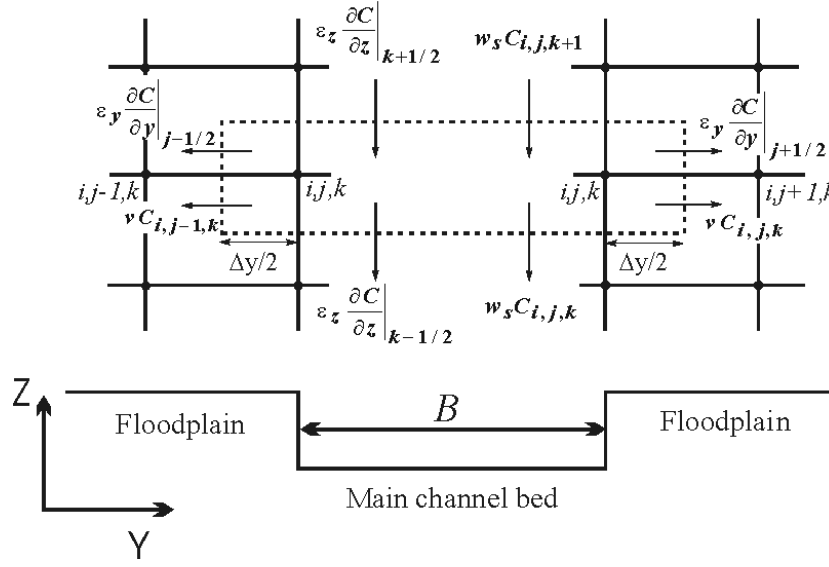


Figure B.13: Finite difference grid in the transverse plane as used by the longitudinal model and fluxes through a fluid element boundaries (for interior points).

expression have already been shown in section B.

$$\begin{aligned}
 M_x + & + \left(\varepsilon_z \frac{\partial C}{\partial z} \Big|_{k+1/2} + w_s C_{j,k+1} - \varepsilon_z \frac{\partial C}{\partial z} \Big|_{k-1/2} - w_s C_{j,k} \right) (\Delta y + B) \Delta x + \\
 & + \left(\varepsilon_y \frac{\partial C}{\partial y} \Big|_{j+1/2} + |v| C_{j-1,k} - \varepsilon_y \frac{\partial C}{\partial y} \Big|_{j-1/2} - |v| C_{j,k} \right) \Delta z \Delta x = 0
 \end{aligned} \quad (\text{B.23})$$

with M_x the net mass entering the element in the X direction.

The terms for concentration gradient in the y direction are evaluated using the transverse distribution model and supplied as input to the longitudinal model. In order to avoid multiplying unknowns in the problem definitions, $C_{j-1,k}$ is replaced by the corresponding gradient:

$$C_{j-1,k} = C_{j,k} - \frac{\partial C}{\partial y} \Big|_{j-1/2} \Delta y \quad (\text{B.24})$$

The vertical and transverse discretisation of equation (B.22) is obtained by substituting expression (B.24) in equation (B.23), dividing by $\Delta x \Delta z (\Delta y + B)$ and making appropriate

finite difference approximations for the remaining terms (see section B).

$$\begin{aligned}
 u_k \frac{\partial C_{j,k}}{\partial x} = & \left(\frac{\varepsilon_{k+1/2}}{\Delta z^2} + \frac{w_s}{\Delta z} \right) C_{j,k+1} + \frac{\varepsilon_{k-1/2}}{\Delta z^2} C_{j,k-1} - \\
 & - \left(\frac{\varepsilon_{k+1/2} + \varepsilon_{k-1/2}}{\Delta z^2} + \frac{w_s}{\Delta z} \right) C_{j,k} + \\
 & + \frac{\varepsilon_y}{(\Delta \mathbf{y} + \mathbf{B})} \frac{\partial \mathbf{C}}{\partial \mathbf{y}} \Big|_{j+1/2} - \left(\frac{\varepsilon_y}{(\Delta \mathbf{y} + \mathbf{B})} + \frac{\mathbf{v} \Delta \mathbf{y}}{(\Delta \mathbf{y} + \mathbf{B})} \right) \frac{\partial \mathbf{C}}{\partial \mathbf{y}} \Big|_{j-1/2}
 \end{aligned} \tag{B.25}$$

The last two terms in Equation (B.25) are expressed for $v > 0$ and are omitted for all points below the level of the plain surface, where transverse transport obviously does not occur.

The Crank-Nicholson scheme is applied once again for discretization of the longitudinal terms (Equation B.10) and the system of equations is closed using appropriate boundary conditions.

Appendix C

UCL flume : experimental set-up illustrations

In this section, secondary illustrations from the laboratory setup described in Chapter 4 are provided.

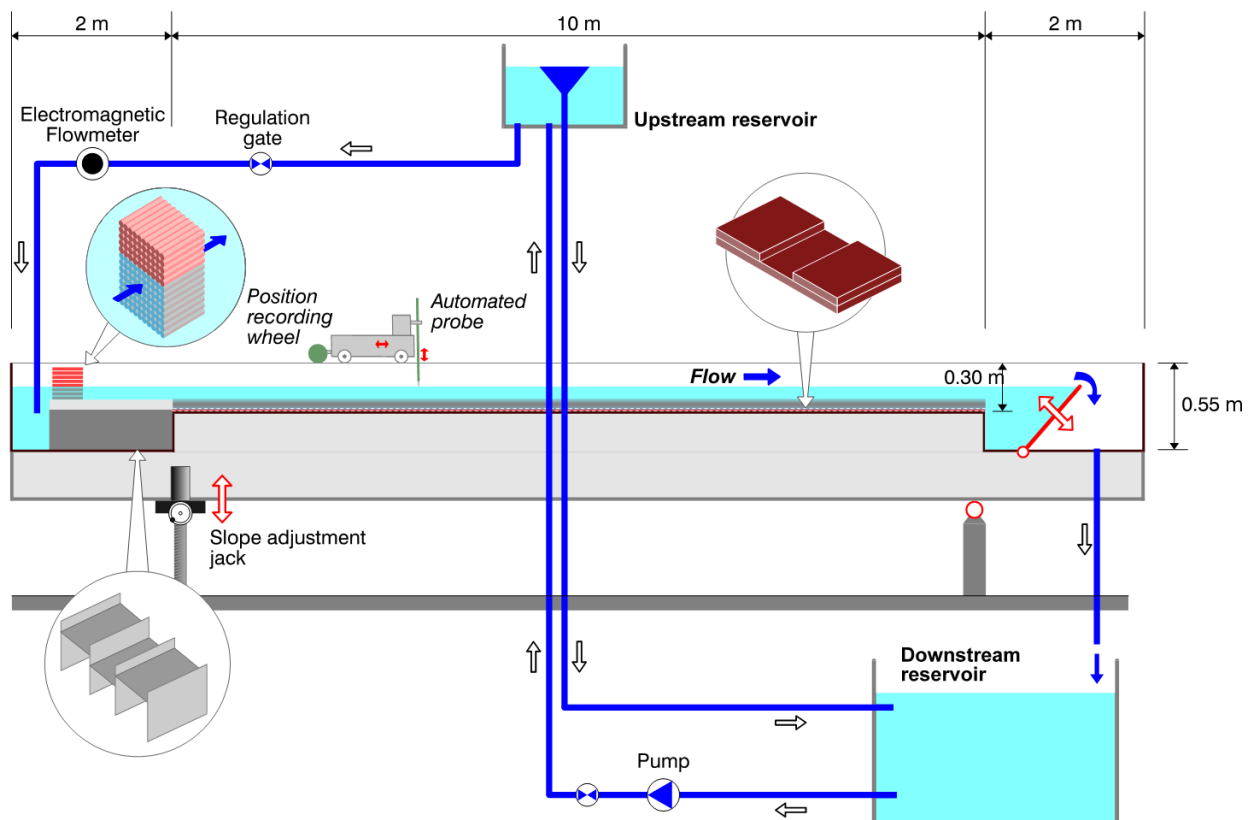


Figure C.1: *UCL compound channel flume. The discharge is supplied through an upstream reservoir with constant level (Bousmar, 2002)*

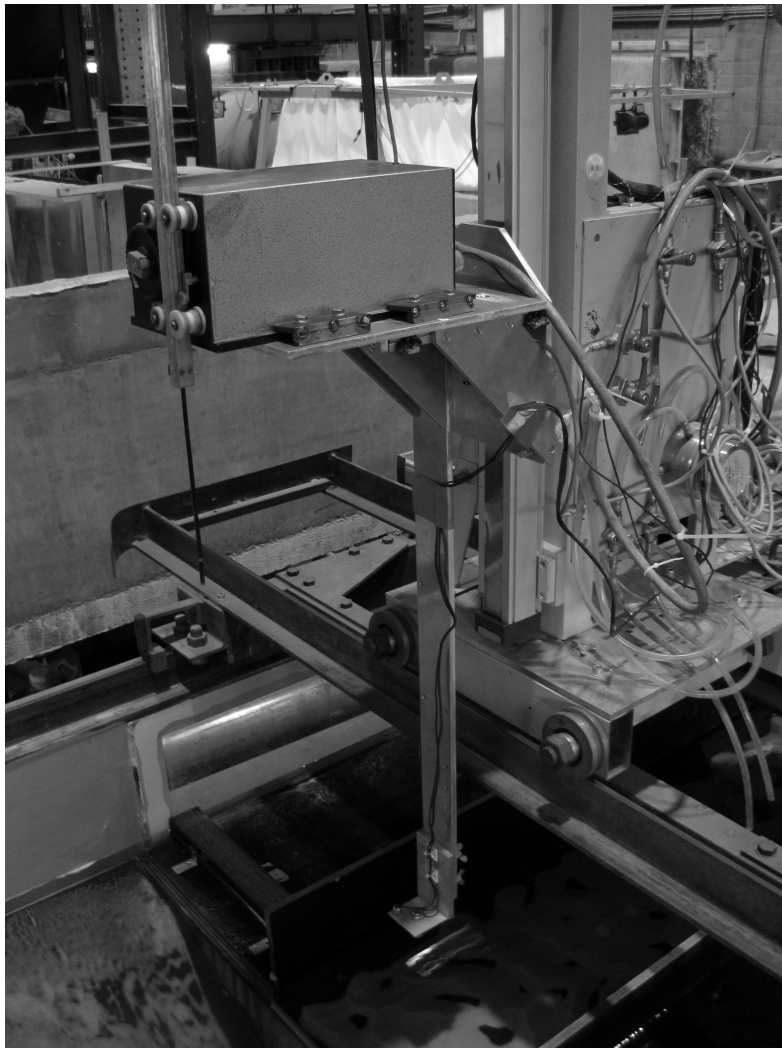


Figure C.2: *Automatic positioning device. Bed profiler (Delft hydraulics) is installed together with an ultrasonic probe (Keyence) for water and bottom level detection.*

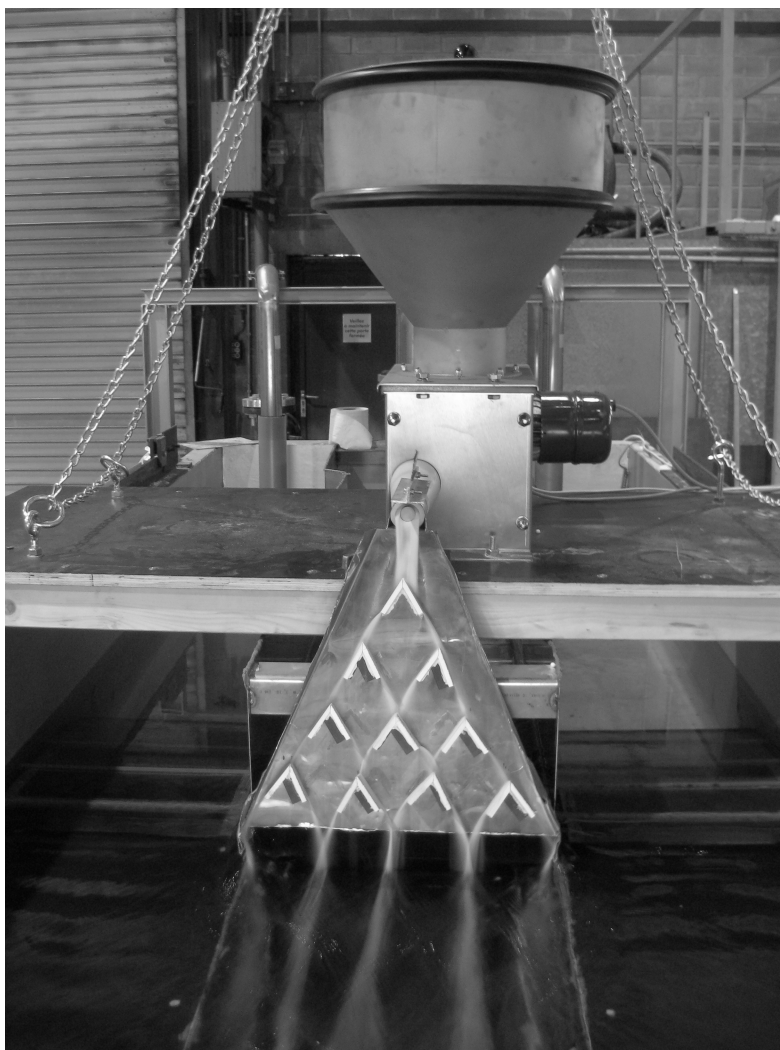


Figure C.3: *Sediment-feed system (archimedes screw) with lateral distribution system*

Appendix D

Laboratory experiments : deposition patterns

In this section, the sediment deposition profiles for the different families of experiments presented in Chapter 5 are provided.

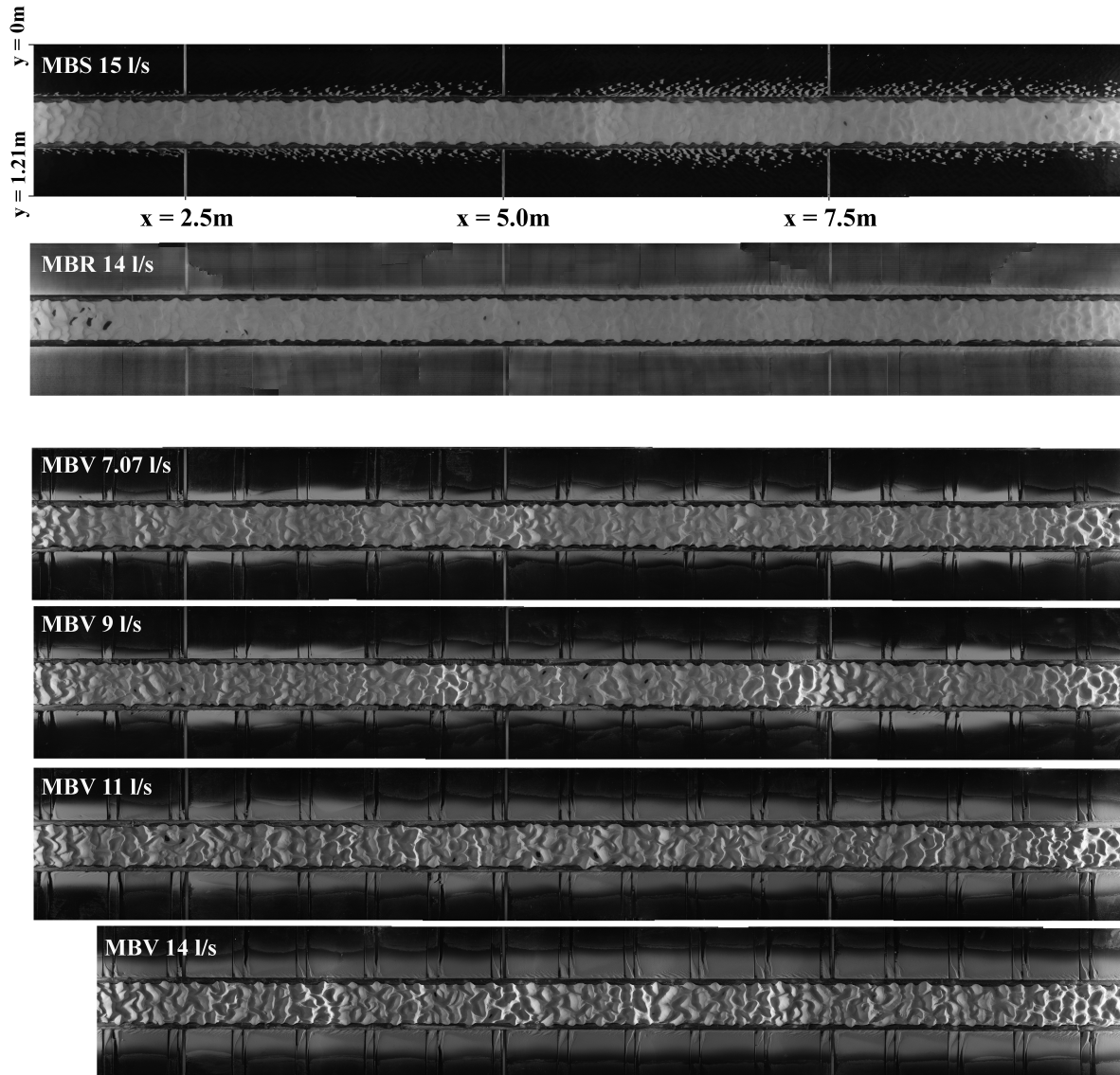


Figure D.1: *Deposition patterns for the different families of experiments over a movable main channel bed. Floodplains are either smooth, rough or "vegetated".*

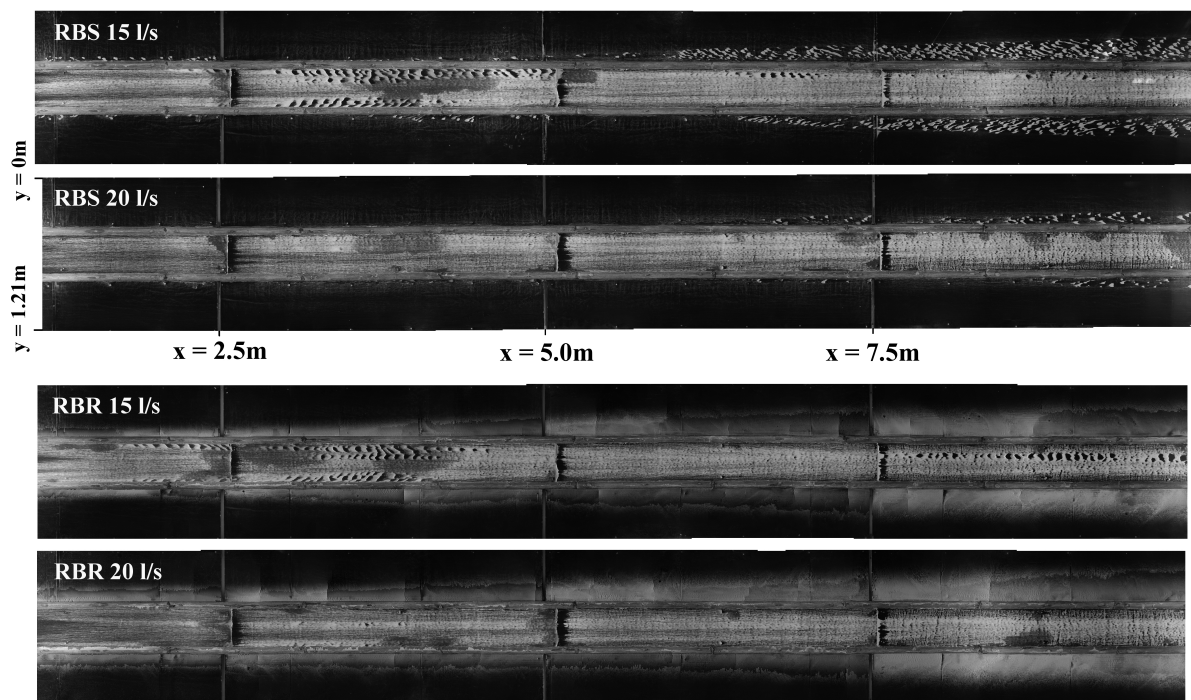


Figure D.2: *Deposition patterns for the different families of experiments over a rigid bed. Floodplains are either smooth or rough.*

Appendix E

Telemac simulations : meshes

In this section, the numerical meshes used in Chapter 6 are provided.

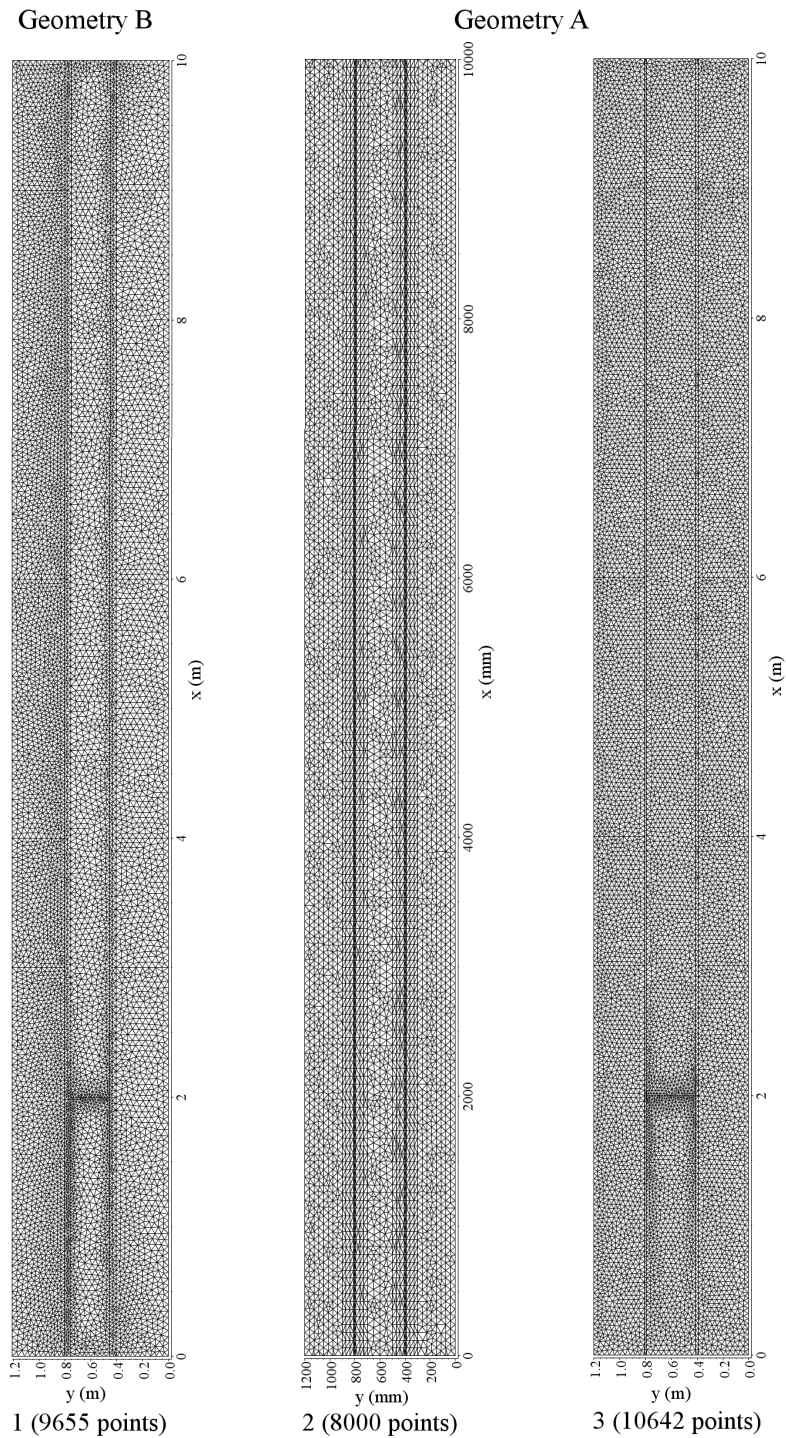


Figure E.1: *Telemac meshes for tracer injections (Section 6.3) and for (1) Geometry B in Fig. 6.2 or (2) and (3) Geometry A in Fig. 6.2*

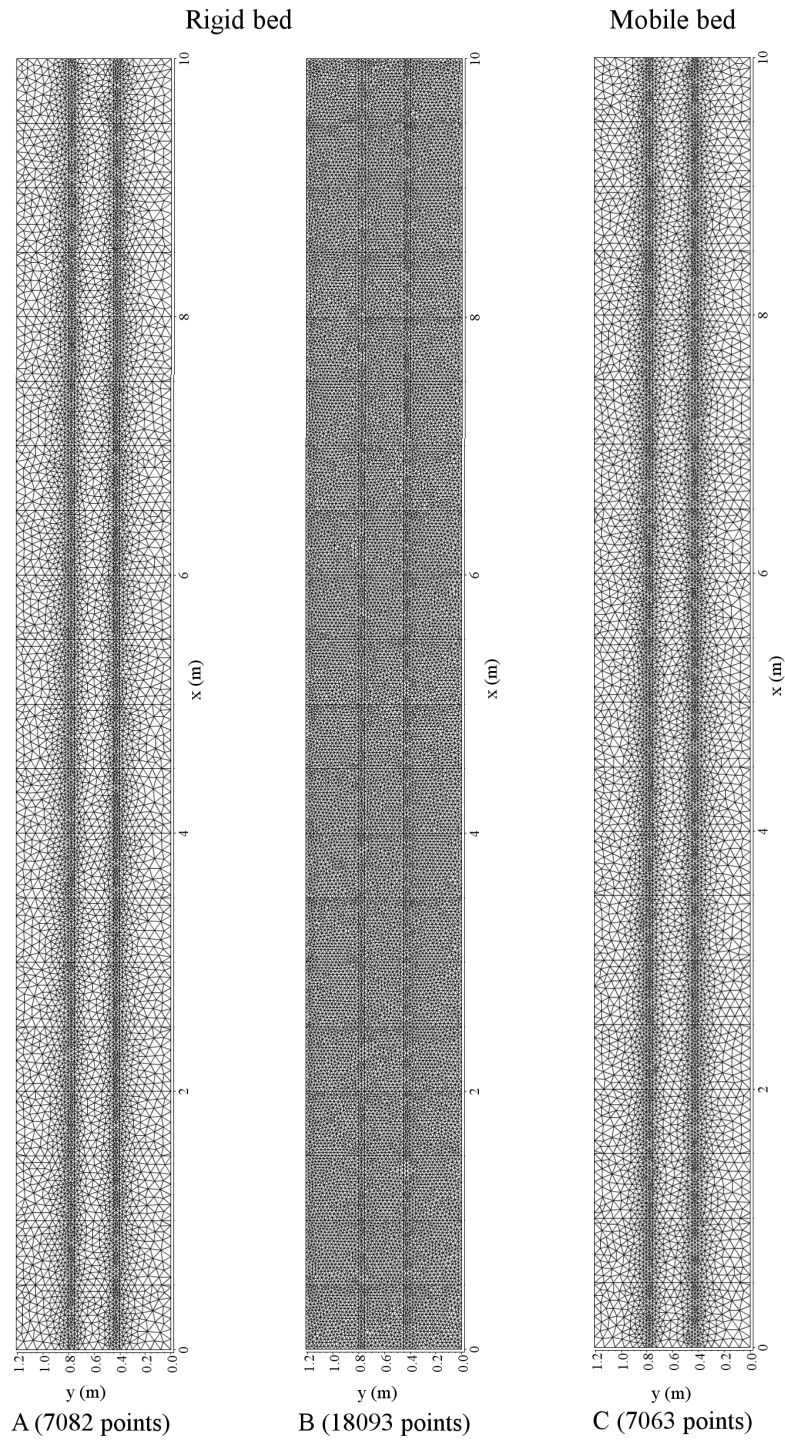


Figure E.2: *Telemac meshes for sediment simulations (Section 6.4) and for (A) and (B) rigid bed geometry from Fig. 5.10b or (C) mobile bed geometry from Fig. 5.10a*

Appendix F

Experimental vortices development

In this section, the experimental turbulent vortices observed for RBS 15 l/s experiment defined in Chapter 5 are presented.

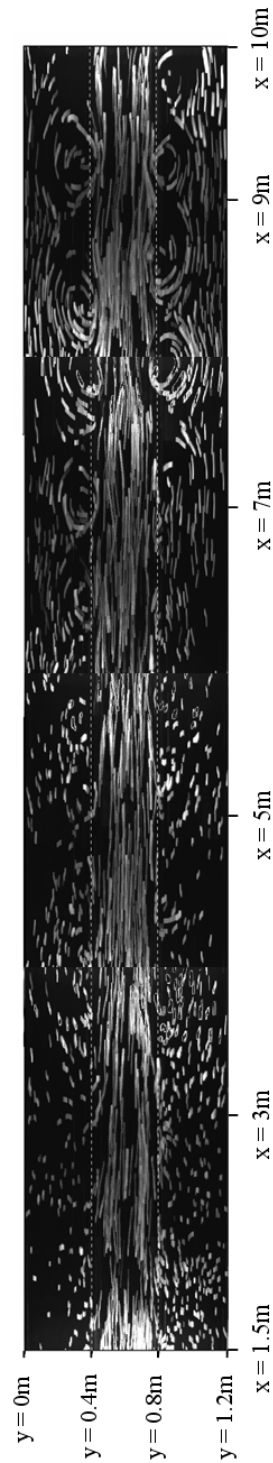


Figure F.1: *Vertical axis eddies appearing in downstream moving frame (0.314 m/s celerity) with $1.9/1000$ slope, 15 l/s (UCL compound channel flume, geometry B from Fig. 6.2b). This situation is found for RBS 15 l/s experiments.*

