

Study on Flow and Bed Evolution in Channels with Spur Dykes

(和文：水制群を有する河道の流れおよび河床変動に関する研究)

(中文：设置丁坝群河道的流场及河床变动)

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August 2005

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Abstract

Investigation of the flow structure and the bed evolution process in channels with spur dykes is an important subject in the campaign of both river training and river restoration. In contrast to the considerable applications of spur dykes in alluvial rivers from ancient time, the knowledge on the flow and sediment transport induced by this conventional structure is still far from enough. This is mostly due to the sophisticated interaction between the flow, sediment and riverbed. After the construction of spur dykes, the existing equilibrium or quasi-equilibrium of the river system is broken, which provokes a process of seeking a new balance by adjusting the above three aspects. And this process is generally a dynamic feedback process.

This thesis aims at characterizing the flow structure and bed evolution induced by spur dykes, identifying the underlying processes and mechanisms and developing numerical models to simulate the dynamic feedback process. Under these objectives, this research concentrates on the flow structure, local scour and fine sediment transport due to the construction of spur dykes.

The local scour around a single spur dyke is initiated and controlled by the complex flow structure, which may be separated into several components: the downward flow, the bow wave, the primary vortex and the wake vortices. Construction of a single spur dyke has only local effect on the flow and the river system. As a result, spur dykes are usually organized in sequences in engineering practices to increase the efficiency and enlarge the improved river stretch. In view of these arguments, two experiments have been carried out to investigate the flow and bed deformation induced by a group of impermeable and permeable spur dykes. A group of spur dykes are found to cause both local obstruction and channel width narrowing. The former leads to the local scour and the latter intensifies the scour as well as results in the main channel degradation. The experiments also indicate that impermeable spur dykes are passive structures followed by a variety of flow turbulences and extreme local scour, while permeable spur dykes are more active and can effectively control the bank-parallel flow and local scour. A combination of these two kinds of spur dykes is suggested for a comprehensive river management.

Then, a morphological model is developed to simulate the flow and bed evolution due to the construction of spur dykes, which includes a hydrodynamic module, a total sediment transport module and their integration.

In the hydrodynamic module, a 3D turbulence model based on an unstructured mesh is proposed. This model solves the Reynolds-averaged Navier-Stokes equations with the standard $k-\varepsilon$ model and some non-linear $k-\varepsilon$ models (optional) for the turbulence closure. The main points during the construction of the model include: (a) an effective data structure for the restoring of the mesh system in particular the connectivity between the control volumes; (b)

suitable discretization methods which have been confirmed to be efficient and stable; (c) necessary treatments for the mesh skewness which is almost unavoidable for unstructured mesh methods and (d) inclusion of mesh movement in the governing equations which makes it possible to track the moving boundaries simultaneously. This model is verified with experimental data and found to be capable of predicting flow phenomena in different geometries with different meshing strategies.

The total sediment load is divided into bed load and suspended load. A modified Ashida-Michiue formula is used for the bed load modeling. In order to account for the bed slope effect especially in the scour area, the threshold condition for the sediment entrainment is corrected with a new bed slope factor and a part of the particle gravity is incorporated into the effective shear stress. The transport of suspended load takes time and space, which is simulated by considering the diffusive and convective processes. Fine sediment has been paid special attention. Fine sediment in this thesis refers to the fine portion of suspended load and consists of cohesive particles. Construction of spur dykes is found to make it possible that fine sediment deposits and exchanges with the local bed. Fine sediment tends to aggregate and form floc system in the water column. And deposition of fine sediment may alter the bed cohesiveness which will affect the sediment erosion process. The deposition and erosion of fine sediment follows the well-known formulae in the cohesive sediment research field, i.e. the Krone's approach for deposition and the Partheniades's formula for erosion. The influence of the bed cohesiveness on the erosion process is treated as follows: (a) the bed is considered to consist of a non-cohesive particle cluster surrounded by a cohesive force field; (b) the erosion process is a two-stage process: sediment is firstly eroded from the non-cohesive bed and then passes through the cohesive force field and (c) the former is evaluated with non-cohesive based formulae and the latter is quantified as a linear function related to the clay content in the bed surface.

The proposed hydrodynamic module and the sediment module are integrated and constitute the morphological model, which is finally applied to predict the bed evolution in the Yodo River system and to simulate two laboratory experiments, i.e. the bed evolution due to a series of non-submerged spur dykes and the local scour around a submerged spur dyke. The computed results are in reasonable agreement with those of the field measurements and experiments. It demonstrates that the proposed model is able to simulate the complex process involving flow, sediment and riverbed. This model can serve as a promising tool in the investigation of flow and sediment related problems with its capability of treating complex geometries and movable boundaries, together with the transport of fine-coarse sediment mixtures.

Key words: Spur dykes, local scour, fine sediment, 3D $k-\varepsilon$ model, unstructured mesh

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Abbreviations

1D	One-dimensional
2D	Two-dimensional
2DH	2D depth-averaged (in the horizontal plane)
2DV	2D model in the vertical plane
3D	Three-dimensional
AES	Algebraic equation system
ARS	Algebraic Reynolds stress simulation
ASCE	American Society of Civil Engineers
Bi-CGSTAB	Bi-conjugate gradient stabilized method
CFD	Computational fluid dynamics
CHES	Chinese Hydraulic Engineering Society
CV	Control volume
DNS	Direct numerical simulation
FDM	Finite difference method
FVM	Finite volume method
GMRES	Generalized minimal residual method
IAHR	International association of hydraulic engineering and research
ILUTP	Incomplete LU factorization with threshold and pivoting
JSCE	Japan Society of Civil Engineers
LES	Large eddy simulation
LU	Lower-upper factorization
PDE	Partial differential equation
RANS	Reynolds-averaged Navier-Stokes equations
SIMPLE	Semi-implicit method for pressure-linked equations
TFQMR	Transpose-free variant of quasi-minimal residual method
TGP	Three Gorges Project

Chapter 1

Introduction

1.1 Motivation

Spur dykes are widely used hydraulic structures in the river engineering practices. They are equipped along the bank and extend to the river perpendicular or at an angle to the flow. According to the structure permeability, spur dykes may be categorized into two kinds: impermeable and permeable. The impermeable spur dykes can be built of the local soils, stones, gravels or rocks with suitable slopes and toe protection countermeasures. The permeable spur dykes usually consist of one or several rows of timber piles, steel piles or reinforced concrete piles. (Klaassen et al., 2002) Typical impermeable and permeable spur dykes are shown in Photo 1.1 and Photo 1.2.



Photo 1.1 Impermeable Spur Dykes along the Katsura River (June, 2005)



Photo 1.2 Permeable Spur Dykes along the Kizu River (May, 2005)

The conventional use of the spur dykes is from the standpoint of river training. For example, deflecting the currents away from the bank to prevent it from erosion, deepening the main channel to improve the navigation conditions, etc. According to a statistic report, a total number of 9,096 spur dykes had been constructed on a 647km-long Lower Yellow River reach by 1997 (Hu et al., 1998 and Hu, 1999). As a result, the main channel has been well regulated in terms of channel migration (Wu et al., 2005). On the other hand, the advantages of spur dykes to improve the riverine aquatic habitats are attracting more and more attention nowadays. The spur dykes are documented to be the most durable solution for river restoration compared with the other three commonly used structures, namely, bank covers, randomly placed boulders and weirs (Shields et al., 1995). According to Yamamoto (1996), the purposes of the construction of spur dykes may be detailedly categorized into 6 kinds: (a) Protection of the bank and revetment; (b) River course stabilization; (c) Improvement of the channel navigability; (d) Maintenance of the discharge and water depth; (e) River scenery and (f) River eco-system.

For either river training or river restoration, the investigation of the flow structure and sediment transport is of significant importance. Nevertheless, the knowledge on the flow and sediment transport around spur dykes to date is still far from enough in contrast to their considerable applications in actual rivers. This is mostly due to the complex transport system induced by the spur dykes involving flow, sediment and riverbed. The three aspects are strongly coupled and interacted with each other, which poses many new challenges for both researchers and practitioners.

1.1.1 Problem definition

Channels with spur dykes generally behave differently from those in the previous pristine conditions. Protruding of the spur dykes to the main channel distorts the existing equilibrium or quasi-equilibrium condition of a river, which may provoke a sophisticated process of seeking the subsequent balance by some kind of self-adjustments. There may be many varieties in this process due to different river characteristics, geometries and organization methods of spur dykes, flow conditions, sediment properties and so on. Some principal features and key problems of engineering interests are summarized as follows.

(1) Flow structure and local scour mechanism

It is well known that the 3D turbulent flow around the spur dyke is extremely complex due to the flow separation and the generation of multiple vortices. The complexity is further aggravated by the dynamic interaction between the flow field and the movable bed with the development of the local scour in the proximity of the spur dyke. According to numerous

researches on spur dykes and other similar structures such as bridge piers and abutments, a schematic diagram of the flow and scour system may be illustrated in Fig. 1.1. A brief overview of the related literatures will be given in the next sub-section.

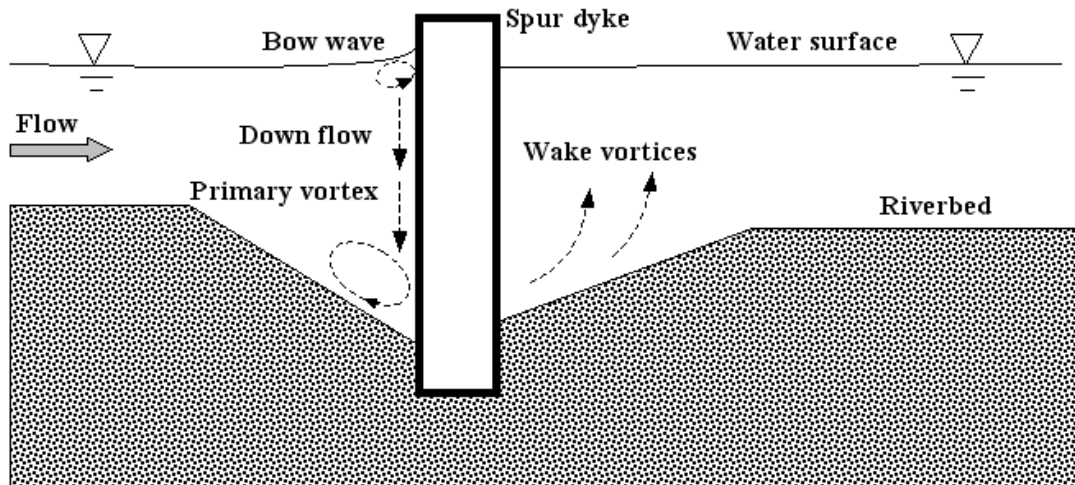


Fig. 1.1 Schematic Diagram of the Flow and Local Scour around a Spur Dyke

The flow field around a spur dyke is characterized by several vortex systems of different sizes, which initiates and controls the local scour and provides the basic scour mechanism. Due to the blockage effect, the flow will be divided into two parts as it approaches the spur dyke. One part approaches the edge of the spur dyke and separates. Another part goes directly ahead of the spur dyke, which decelerates and is deflected to form an upward flow and a downward flow. The upward flow makes some kind of surface roller which is generally termed bow wave. Near the free surface, the water depth is increased and the highest stagnation pressure occurs. The pressure is rapidly decreased downwards due to the steep vertical velocity gradient. As a result, the downward flow is driven and acts as an impinging jet on the channel bed. With the development of the scour hole, a primary vortex (also known as horseshoe vortex) develops. The primary vortex extends to the downstream of the spur dyke and loses its identity after some distances. The flow detaching from the spur dyke accelerates and leads to the development of concentrated cast-off vortices in the interface between the flow and the wake behind the spur dyke, which are termed wake vortices. The wake vortices are directed upward to the free surface and have almost vertical axes. These vortices are transported downstream and function as vacuum cleaners to suck and suspend the bed sediment into the main flow.

If the spur dyke is submerged, the flow structure becomes more complex. The over-topping flow has a significant impact on the nature of the vortices around the spur dyke. The primary

vortex gains an upward component in the lee of the spur dyke (Kuhnle et al., 1999, 2002). And a strong downward flow is introduced to the wake area behind the spur dyke (Ishigaki and Baba, 2004). Furthermore, the over-topping ratio (water depth to spur dyke height) is an important control on the geometry of the resulting scour hole.

(2) Sediment sorting and fine sediment transport

A spur dyke introduces a wide spectrum of turbulences around the structure, which is not only the engine of the local scour but also responsible for the new process of sediment sorting. The term *sediment sorting* here refers to the selective transport of sediment particles of different sizes, which is of significant meaning as the sediment size is an important indicator of the aquatic habitat quality (Shields and Milhous, 1992). On the one hand, the intensified flow velocity and the vortices around the spur dyke have a potential to remove coarser particles and even the armor layer formed in the previous river conditions. On the other hand, the wake zone behind the spur dyke may serve as a shelter for the fine sediment, i.e. the fine sediment will participate the bed shaping process. Moreover, the fine sediment is also well known for its environmental relevance due to easily collecting nutrients and pollutants.

Table 1.1 Sieve Analysis Results of the Bed Materials on a Cross-section of the Yodo River (Location: River mile 16.0k; Year: 1998; Data from: Newjec Inc., Osaka)

Sieve size (μ m)	Cumulative (%) 16.0 k (left)	Cumulative (%) 16.0 k (center)	Cumulative (%) 16.0 k (right)
9500	100	100	100
4750	100	98.5	100
2000	100	94.5	100
850	100	79	100
425	100	38.7	98.9
250	99	11.8	95.5
106	98.1	3.3	52.5
75	97.4	3.1	37.9
58	79.1	2.7	19.2
41	58	2.7	14.9
26	47.5	2.7	10.7
15	36.9	1.8	8.5
11	31.7	1.8	6.4
7.5	26.4	1.8	4.3
3.8	18.5	0.9	2.1
1.5	10.6	0.9	2.1

Table 1.1 shows the sieve analysis results of the bed materials on a cross-section of the Yodo River in West Japan. Spur dykes have been constructed along both the upstream and the downstream of this section. It may be found that the fine particles less than 75μ m play a

crucial role in the bed composition especially on the left side.

The behavior of the fine sediment is much sophisticated due to the cohesiveness. The problem is blurred to be more complex if the fine particles are mixed with the coarse ones, which is usually the actual case in river conditions. A mixture of fine and coarse particles manifests some new characteristics during transport. The fine sediment may aggregate and form floc system in the water column, and the behavior of which will be quite different from that of an individual particle. Furthermore, when the fine sediment participates the bed shaping process, it may alter the bed cohesiveness, which in return will affect the sediment erosion process.

1.1.2 Previous researches: a brief overview

(1) Modeling of local scour

Researches on local scour have a long history and are of continuous interest. These researches may be classified into two kinds. The first kind is concerned with the prediction of the equilibrium or design scour depth using empirical or semi-empirical formulae based on field data or experimental measurements or both. Literatures related to this kind are predominant. Despite the large number in amount, these formulae select a limited number of variables from the approach flow depth, flow intensity, sediment size, spur dyke shape, size and alignment, etc. For example, the earlier studies carried out by Laurrsen and Toch, 1956; Laursen, 1963; Shen, et al., 1969; Nakagawa and Suzuki, 1974 and Raudkivi and Ettema, 1983. The 1990s and the past several years witnessed the flourish of this kind of methods, which may be partially due to the further understanding of the scour mechanism and the availability of a great number of experimental data. The representative formula is that proposed by Melville based on a series of experiments conducted in the University of Auckland, New Zealand (Melville, 1992; Melville, 1995; Melville, 1997). Johnson made a comparison of 7 commonly used and cited formulae (including Melville's formula) with a large set of field data for both live-bed and clear water scour (Johnson, 1995). The results of his study pointed out the necessity of additional researches. Other researches may include, for example, Lim, 1997; Lim and Cheng, 1998; Ettema et al., 1998; Rahman, 1998; Rahman and Muramoto, 1999; Kuhnle et al., 1999; Rahman and Haque, 2004; Kuhnle et al., 2002; Coleman, S.E., 2005. The names and the publications may be too long to list. An extension of this kind of researches is to take into account the temporal variation of the local scour holes. Most of them are achieved by determining a function relating the time-dependent scour depth to the equilibrium scour depth, e.g. Cardoso and Bettess, 1999; Chiew and Melville, 1999; Kothyari, U.C. and Ranga Raju, 2001; Coleman, et al., 2003; Dey and Barbhuiya, 2005. Despite the significant expansion of the literature numbers, this kind of researches involves strong empiricism and introduces many uncertainties. They do not always

give reasonable estimates in field conditions and sometimes even in the experimental scale under conditions slightly different from which they were derived from (Salaheldin et al., 2004). Moreover, the geometry of the scour holes is unknown and the interaction between the flow and sediment transport is not included. In view of these limitations, the second kind of researches is more promising, i.e. the numerical methods.

Nevertheless, the numerical investigation of the local scour around spur dykes is still in its infant due to the complexity of the problem involving 3D flow, sediment and bed evolution. Some of the recent researches are summarized as below.

Ushijima et al. (1992) proposed a method to estimate the local scour due to the cooling water jets discharged from power stations. The calculation of the local scour in forms of both bed load and suspended load was performed unsteadily in parallel with the calculation of the flow which was solved through the $k-\varepsilon$ turbulence model. Ushijima (1996) later extended the model on the basis of Lagrangian-Eulerian formulation and found a great improvement compared with the previous one. Omitting the transient term, Olsen and Melaaen (1993) calculated the scour hole around a cylinder by solving the 3D RANS equation on a structured non-orthogonal grid with the $k-\varepsilon$ model for the Reynolds stresses and the convection-diffusion equation for the sediment transport. During the iteration process, the flow field and bed geometry at the previous iteration are used to calculate the new velocity and bed change. This procedure repeated until the maximum scour hole depth was equal to that observed in the experimental measurement. The model was later extended with transient terms by Olsen and Kjellesvig (1998). Bhuiyan et al. (2004) reported that the model was also able to simulate the flow and bed evolution around submerged and unsubmerged spurs. Fukuoka et al. (1994) developed a 3D flow model with an empirical eddy viscosity equation and under the assumption of hydrostatic pressure. The bed deformation model included the effect of non-equilibrium sediment transport processes. The simulation result for a bridge pier was in good agreement with the experimental result. The model was later improved and extended. According to the computation work carried out by Watanabe et al. (2001), it was also able to predict the flow and bed deformation around groins in a curved channel. Peng et al. (1998) assumed that the flow field within each time step of the bed deformation was steady. A modified $k-\varepsilon$ turbulence model integrated with a modified version of the Meyer-Peter and Muller formula for sediment transport was developed to simulate the equilibrium scour pattern around a spur dike. The model result was reported to be encouraging. Yen et al. (2001) employed an LES (Large eddy simulation) approach with Smagorinsky's sub-grid scale turbulence model to calculate the flow field around a bridge pier. The scour model solved the sediment continuity equation in conjunction with van Rijn's bed load transport formula. Without re-computing the 3D flow field as the bed deformed, the shear stress obtained from the 3D flow field under flatbed conditions was modified according to the

bed change. This has greatly reduced the computational time. Nagata et al. (2002) reported a model based on a moving boundary-fitted coordinate system. The velocity field and shear stress are calculated from a non-linear $k-\varepsilon$ turbulence model. The bed deformation around a cylindrical pier was simulated by coupling the momentum equation of sediment with stochastic models for sediment pick-up and deposition rate. It was found that the proposed numerical model was able to reproduce the flow and scour geometry in the laboratory experiment with a sufficient accuracy. All these studies employed structured mesh which is only suitable for relatively simple geometries. However the study domain in engineering practices is generally complicated, e.g. the irregular riverbed, spur dykes with complex shapes. Olsen (2003, 2004) also noted this and extended his structured mesh based model to an unstructured mesh system. The new model has been employed to simulate the formation of the meandering pattern in an initially straight alluvial channel. However, the model capability is not quite known as the local scour is quite different from a development of a meandering channel and moreover very little information has been given quantitatively in his paper.

(2) Fine sediment transport in rivers

The studies on the fine sediment transport are very few in the river engineering. This is due to the fact that the fine sediment does not significantly presented in the bed in natural river conditions and is generally neglected in the investigations.

Fine sediment in this thesis refers to the fine portion of the suspended load that moves in the form of flocs rather than individual particles. This should be distinguished from the conventionally used *wash load* because there is usually some confusion between the two terms. The generally accepted concept of *wash load* was firstly defined by Einstein (1950) as the part of the sediment load which consists of particles finer than those significantly presented in the bed (i.e. bed material load). According to this definition, wash load normally consists of, but is not limited to, very fine particles depending on the river and sediment conditions. And because of this, fine sediment has been quite often considered as wash load in the river engineering.

The use of fine sediment rather than wash load in this thesis is necessary and important because the former takes into account the *cause* of the sediment movement while the latter is defined from the *result* of the sediment motion. The difference between wash load and bed material load lies not so much on their mechanical composition but in their dependence on the flow parameters. As a result, the behavior of wash load is sometimes puzzling (Partheniades, 1977). The introduction of the term wash load did not push the researches on the right way to understand the fine sediment transport. On the contrary, fine sediment is poorly understood till now and there are many disputes on the term wash load itself (Richardson and Julien, 1986). In the author's opinion, it is no need to distinguish wash load from bed-material load because there

is no essential difference between them, but it is very important to distinguish fine sediment from coarse sediment because they differ in the motion mechanisms.

Einstein (1950) assumed that there was a transition between bed material load and wash load and that the transport rate of the sediment in the wash load range was a function of the upstream input and independent of the local flow. This has been widely accepted in the publications in the river engineering (e.g. Graf, 1971; Garde and Ranga Raju, 1985; Chang, 1986; van Rijn, 1993) and followed by most researchers after. The criteria for the transition are expressed by a cutoff size which may be derived from different ways, for example, a diameter according to experiences, the percentage in the bed materials, the suspension indicator $Z_s = w_s / (\kappa u_*)$ (where w_s = the falling velocity of the particle; κ = the von Karman constant and u_* = the shear velocity), the particle Reynolds number $R_{ep} = w_s d / \nu$ (where d = the particle diameter and ν = the molecular kinematic viscosity of the water) and the inflexion method (e.g. Einstein, 1950; Einstein and Chien, 1953; Egashira and Ashida, 1981; Chien and Wan, 1986; Yang et al., 1996; Qian et al., 1998; Zhong et al. 2000a and Wang et al., 2004). Zhong et al. suggested a more complex criterion considering the concentration gradient of the sediment near the bed (Zhong et al., 2000a and Zhong et al., 2000b). With this criterion, wash load is further divided into two parts: one part behaves like bed material load, i.e. the transport rate of which is related to the local flow conditions; while the other part is carried by water and has no exchange with the local bed. Partheniades (1977) noticed the cohesive effect of fine particles and provided a logical link between wash load and suspended bed-material load. This work was very encouraging. However, he did not walk out of the shadow of the wash load concept. In the final conclusion, he also divided wash load into two parts. Some researchers argued that it was no need to distinguish bed material load and wash load and that the transport rate formulae for suspended load could be used directly for wash load (e.g. Han and Wang, 1980). This theory contradicts the common belief that the transport of wash load only depends on the input as mentioned before. From this viewpoint, it coincides with the opinion of this thesis. Nevertheless this theory is still questionable because the suspended load formulae based on non-cohesive sediment cannot be extended to wash load straightforwardly if the particles of wash load are fine enough.

Very recently, Yang and Simoes (2005) calculated the wash load transport rate in the Yellow River with two dimensionless unit stream power formulae. They argued that wash load is related to the flow hydraulics and affects the river's bed-material transport capacity contrary to the conventional wisdom. Although they did not distinguish the different motion mechanisms of different sediment loads, their result further confirmed that the definition of wash load is questionable. It is interesting to mention that Yang et al. also calculated the bed material load transport in the Yellow River in 1996 (Yang et al., 1996). At that time, they defined wash load as

the particles finer than 0.01mm and having almost no exchange with the local bed.

1.2 About the dissertation

1.2.1 Objectives of the research

A reliable and effective design of spur dykes should be based on a clear insight into the physics of the flow and sediment transport. This study aims at contributing to the understanding and modeling of flow and bed evolution in channels with spur dykes. The main objectives of this study are summarized as follows.

- (1) Characterize the flow structure and bed evolution induced by spur dykes and identify the underlying processes and mechanisms.
- (2) Develop numerical models (hydrodynamic models and sediment transport models) that can model the spur-dyke-induced problems, in particular the local scour phenomenon and fine sediment transport.
- (3) Integrate the hydrodynamic models and sediment transport models in different levels to predict spur-dyke-induced flow and sediment problems.

1.2.2 Outlines of the dissertation

This dissertation concerns the flow and bed evolution in channels with spur dykes and concentrates on the understanding and modeling of local scour and fine sediment transport. To the best of the author's knowledge, it is the first time that a spur dyke related research includes both local scour and fine sediment transport.

The present dissertation is organized in 6 chapters. In this chapter, the motivation and the objectives of this study, together with a brief introduction of the dissertation contents are presented.

In Chapter 2, some experimental results and analyses will be given. Since very few detailed information is available concerning a series of non-submerged spur dykes, the experiments have been carried out with impermeable and permeable spur dykes organized in sequences. The experiments were conducted under the same hydraulic conditions, which made it possible to compare the flow structure and bed deformation due to these two different kinds of spur dykes. The impermeable spur dykes may be characterized as passive structures, which is followed by a wide spectrum of turbulences and extreme local scour at the toes of the spur dykes. The permeable spur dykes are found to be more active. Permeable spur dykes can get an effective

control of the river flow and the local scour. A combination of these two kinds of spur dykes is suggested from the perspective of comprehensive river management. The effect of a group of spur dykes to the flow and the river system is also found to be quite different from that of a single spur dyke. A single spur dyke has only local effect to the river expressed by the local scour phenomenon, while a group of spur dykes lead to not only local scour (mostly caused by the effect of local obstructions) but also main channel degradation (due to the artificial narrowing of the channel width).

In Chapter 3, the hydrodynamic models are presented. From the viewpoint of practicability, two different levels of hydrodynamic models are introduced for a more cost-effective solution. A 1D model is used for the long-term and large-scale prediction of river systems, while a 3D model is employed to resolve the local flow phenomena. Emphasis in this study has been put on the construction of the 3D model. This model is developed on a moving unstructured mesh and provides options of the standard $k-\varepsilon$ model and some non-linear $k-\varepsilon$ models. Arbitrary mesh up to six faces may be used with this model. The governing equations, the discretization methods, the boundary conditions and the solution algorithms are described in detail. The main points during the construction of the model are: (a) an effective data structure for the restoring of the mesh system in particular the connectivity between the control volumes; (b) suitable discretization methods which have been confirmed to be efficient and stable; (c) necessary treatments for the mesh skewness which is almost unavoidable for unstructured mesh methods and (d) inclusion of mesh movement in the governing equations which makes it possible to track the moving boundaries simultaneously. The model has finally been applied to predict some typical flow phenomena in laboratory flumes and exhibits the capability.

In Chapter 4, the sediment transport models are presented. The total sediment load is divided into bed load and suspended load. In the bed load module, the Ashida-Michiue formula is employed. In order to simulate the local bed slope effect, a bed slope factor has been introduced to extend the formula from a horizontal bed to a sloping bed. In addition, part of the particle gravity is also incorporated into the effective shear stress. Suspended load is further divided into coarse sediment and fine sediment. The concentration of either kind of sediment is obtained by solving the diffusion-convection equations. Fine sediment moves as flocs in the water column, the deposition amount is evaluated with empirical formulae for cohesive sediment. As coarse sediment is non-cohesive, the movement may be estimated with the classical methods for suspended load. The exchange of fine sediment with the local riverbed will alter the bed cohesiveness, which may affect the sediment erosion process. This phenomenon is modeled by quantifying the bed cohesiveness as a linear function related to the clay content in the bed surface. Algorithms for the bed sorting processes are presented as well. Due to the bed sorting, the bed cohesiveness changes with time.

In Chapter 5, some applications are presented by using the morphological model, which combines the hydrodynamic module with the sediment transport module. The bed evolution process of the Yodo River system in West Japan is simulated with the proposed model for sediment mixtures including both coarse and fine sediment. A conventional model excluding the effect of fine sediment is also implemented for comparing the performance of the two models. The proposed model gives quite close result to the field measurement and significantly humbles the conventional one. Then, the model is applied to predict the bed deformation in a model flume with a series of non-submerged spur dykes. The bed morphology at the equilibrium state is found to be reasonably reproduced by the 3D model, including both the local scour around the spur dykes and the main channel degradation. The flow structure is also well captured. Finally, the local scour around a spur dyke with over-topping flow is computed with the proposed morphological model. Since the geometry in this case is relatively complex, a model based on an unstructured mesh may be the best solution. The flow pattern and the local scour after 1 hour are simulated with the proposed model. The model result is in good agreement with the limited experimental data. From the numerical simulation, many details on the flow structure have been observed, which have not been measured in the current experiment but have been found in similar experiments carried out by other researchers.

In Chapter 6, the results obtained in this study are summarized and concluded and suggestions for future researches are also presented.

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Chapter 2

Experimental studies

2.1 Introduction

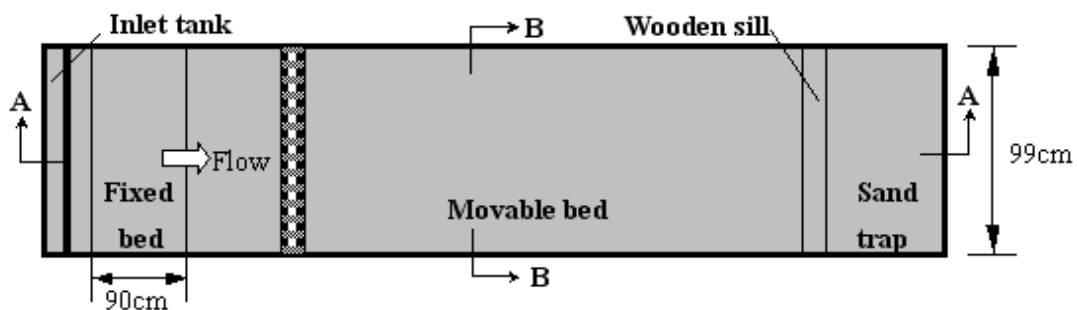
Experimental methods are effective ways for the understanding of the physics of the flow and sediment transport in channels with spur dykes. The geometry flexibility and controllable experiment parameters make laboratory measurements of significant meaning in the past several decades and of continuous interest till now, for example, Ahmad, 1953; Nakagawa and Suzuki, 1974; Kwan, 1984; Rahman, 1998; Ishigaki and Baba, 2004 and Sheppard et al., 2005. Nevertheless, experimental studies to date have been predominantly carried out with an individual spur dyke or spur-dyke-like structure. The spur-dyke-like structure here may include the bridge piers, the abutments and some other local obstructions. All these structures concentrate the river flow and can therefore cause local scour.

The previous experimental studies provide a substantial collection of data and help to further the understanding of the flow and sediment transport around such kind of structures. However, construction of a single spur dyke is generally not preferred in the river engineering practice. Prototype spur dykes are arranged in sequences in order to increase the efficiency and enlarge the improved stretch of the river system. Grouped spur dykes exhibit some new characteristics to the flow and river system which deserve special attention. For instance, as the presence of the spur dyke has a great effect on the flow pattern in the river downstream, the spur dykes organized further downstream will experience these flow disturbances. The change in the flow pattern may lead to the change of the scour pattern, which may result in a further change in the flow pattern as these two aspects are strongly coupled. This process continues until a new balance is reached. What will happen during this process and how can this process be effectively controlled or utilized? Engineers should have a clear insight into these problems. Unfortunately, experimental data concerning the bed variation around a series of spur dykes is still very few in the state-of-the-art of the literatures, available researches include, e.g. Ohmoto et al., 1998; Bahar and Fukuoka, 1999; Egashira et al., 2002; Muto et al., 2003; Kitamura, 2003. In view of these arguments, two experiments using a series of impermeable and permeable spur dykes were conducted in this study.

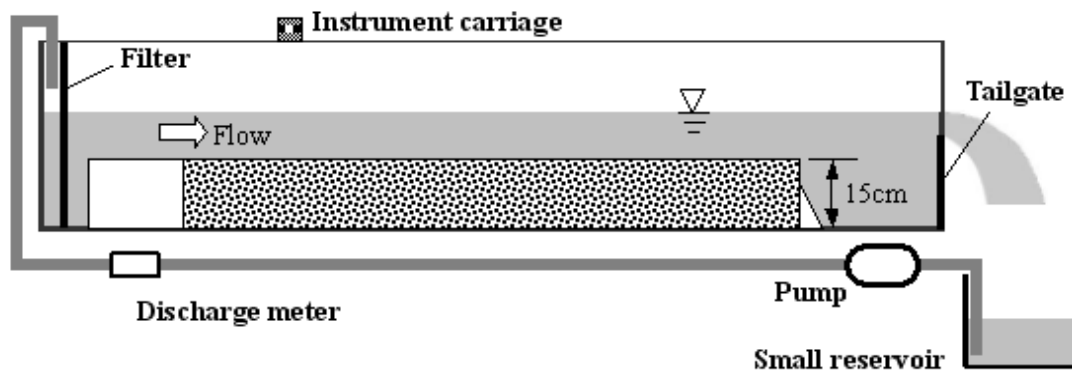
2.2 Experiment conditions and procedures

2.2.1 Laboratory flume

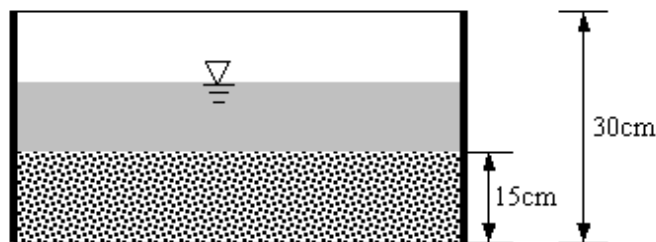
The experiments were conducted in a glass flume located in the Ujigawa Hydraulics Laboratory, Kyoto University. The flume was 20m long, 1m wide and 30cm deep. After setting a Styrofoam lining of 0.5cm thickness along either side of the bank, the effective channel width in the current experiments was 99cm. The bottom of the flume was adjustable so that nearly uniform flow conditions might be achieved.



Top View of the Experimental Flume



Longitudinal Cross-section A-A



Transverse Cross-section B-B

Fig.2.1 Sketch of the Experimental Flume

The sketch of the experimental flume was depicted in Fig. 2.1. The flume was divided into an upstream section, a test section and a downstream section. In the upstream area, there was a fixed bed made of a wooden plank. It was situated 15cm above the flume bottom and extended 0.9m downstream from the inlet tank. The test reach was composed of a movable bed with relatively fine and uniform sands to a depth of 15cm. A wooden sill was used to lock up the movable bed at the end of this reach. It was followed by a sand trap in the downstream section. At the outflow point of the channel, a tailgate has been set to control the water level. The water passed through a sediment filter before it flowed to a small reservoir from which the water was pumped back to the laboratory system. The pump was connected to a computer system and the flow discharge might be controlled manually or by the computer automatically.

2.2.2 Model spur dykes

Along both sides of the flume, ten pairs of impermeable and permeable spur dykes have been equipped in the corresponding experimental run. The impermeable spur dykes were made of wooden cuboids. And the permeable spur dykes consisted of round sticks and were designed to have a permeability of 50%. (see Photo 2.1 for details) The impermeable spur dyke was 1.5cm thick and 14.6cm long, which was supported at the bottom with a 2cm-thick foundation. Twelve piles were used for one permeable spur dyke and all these piles were fixed to a 2cm-thick foundation at the bottom. Each pile had a diameter of 0.6cm and the space between two consecutive piles was also 0.6cm. All these spur dykes were organized perpendicular to the channel banks.



Photo 2.1 Permeable (Left) and Impermeable (Right) Spur Dykes Used in the Experiments

2.2.3 Sediment properties

The sediment used for the experiments had a mean diameter of $196\ \mu\text{m}$ and a geometric standard deviation of 1.496, which were based on a sieve analysis. The sieve analysis results were shown in Table 2.1 and Fig. 2.2.

Table 2.1 Sieve Analysis Results of the Sediment in the Experiments

Sieve (μm)	31.2	38.1	46.4	56.6	69.0	84.1	102.5	125.0	152.3
Cumulative (%)	0	0.02	0.16	0.73	2.31	5.62	10.98	17.71	29.06
Sieve (μm)	185.7	226.4	275.9	336.4	410.1	499.9	609.4	724.9	
Cumulative (%)	47.73	72.06	89.11	94.14	97.79	99.47	99.94	100	

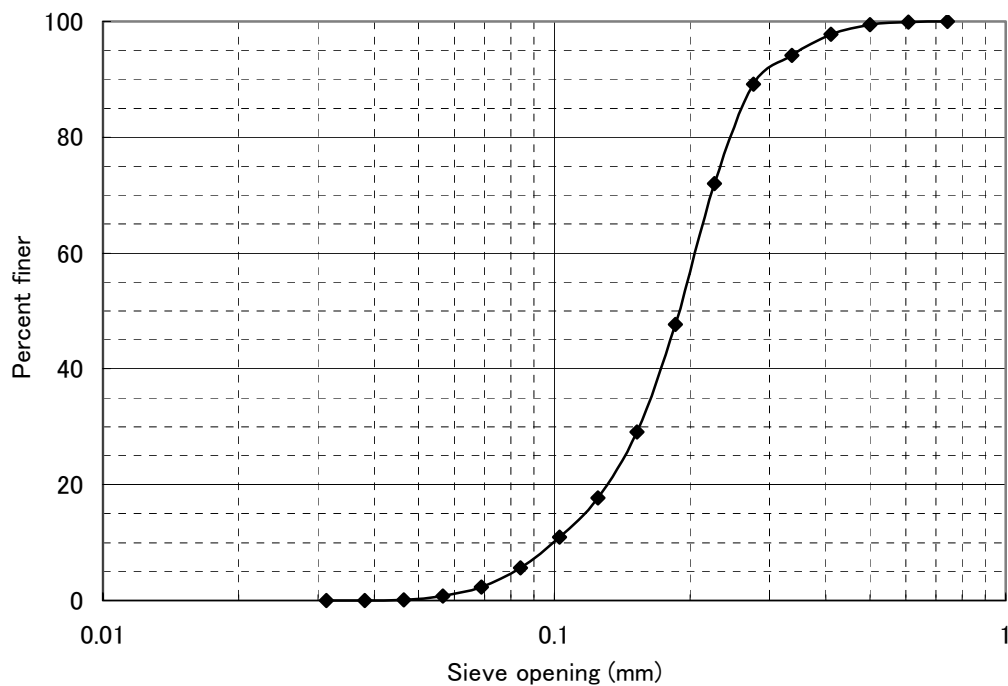


Fig. 2.2 Cumulative Semi-logarithmic Probability Size-frequency Graph

The angle of the sediment repose was measured to be 34.8° by taking the sample from the final riverbed after all the experiments were completed.

The sediment density was 2.56g/cm^3 , and the critical shear velocity $u_{*c} = 1.475\text{cm/s}$ by using the Iwagaki Formula (1956). With the Rubey's Formula (1933), the sediment falling velocity was estimated to be $w_s = 2.463\text{cm/s}$.

2.2.4 Measurement apparatus

The 3D flow velocity, the water surface variation and the bed deformation were measured at the equilibrium state. The velocity components were collected with an I-shape and an L-shape (the type of the probe) electromagnetic velocity meters (model ACM250-A, Alec Electronics Co., LTD) under dynamic flow conditions. The average of 600 samples processed by a computerized data acquisition system (see Fig. 2.3) at a frequency of 10Hz was taken as the time-averaged value of each measured quantity at each point. A point gauge was utilized to measure the water level. The flow structure on the free surface was visualized before stopping the pump by employing a mixture of sawdust and aluminum powders as tracers. And finally a laser-type transducer (see Fig. 2.4) was used to determine the bed deformation after the flume was completely drained out. All these measurement devices were mounted on an instrument carriage that traveled on the rails over the channel.

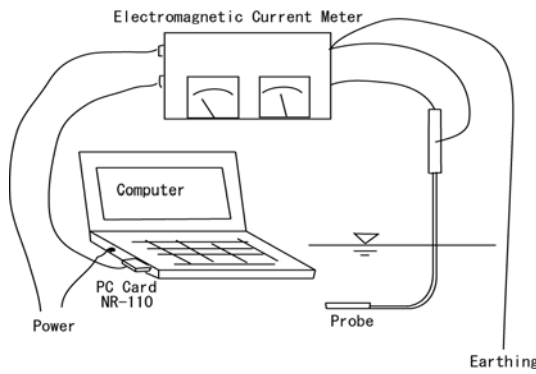


Fig. 2.3 Measurement of the Velocity

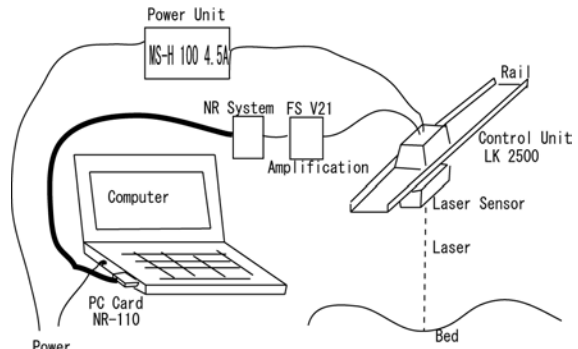


Fig. 2.4 Measurement of the Bed Deformation

2.2.5 Experiment procedures

The two experiments in this study were important components of a series of experiments related to spur-dyke-like structures carried out at Kyoto University (e.g. Khaleduzzaman, 2004, Takeuchi, 2004 and Ito, 2005). The sediment was reused in these experiments. After one case was finished, the sediment in the sand trap was moved to the test reach and new sediment from the supply container was also used to supplement the unavoidable sediment loss during the experiment. Before each experimental run, the sediment bed surface was leveled with a scraper blade mounted on a carriage riding on the rails over the channel. After that, the flume was slowly filled with water from the downstream side. This might avoid the undesirable scour by the action of the sheet flow at an inadequate water depth. When the desired water depth was

achieved by adjusting the height of the tailgate at the end of the flume, the pump was started.

The experiments were ended when the variations concerning the bed configuration were not significant anymore. It took about 893 hours for the impermeable case and 454 hours for the permeable case, respectively.

2.2.6 Experiment conditions

The center of the first spur dyke was 5.5m away from the inlet boundary, which was theoretically sufficient to achieve a fully developed turbulent flow at the position around the first embayment (i.e. the area formed by the two consecutive spur dykes). In either case, the spur dykes were non-submerged. Moreover, the scour holes were allowed to develop under the clear-water condition, i.e. the approach flow velocity was less than the critical flow velocity for the sediment entrainment.

Table 2.2 Details of the Hydraulic Conditions

Discharge Q (l/s)	Mean velocity u (cm/s)	Flow depth h (cm)	Shear velocity u_* (cm/s)	Shear velocity ratio (u_*/u_{*c})	Reynolds number	Froude number
10.52	18.98	5.60	1.35	0.915	10,682	0.26



Photo 2.2 Experiment Setup (Impermeable Spur Dykes, Left; Permeable Spur Dykes, Right)

The two experiments were conducted under the same hydraulic conditions. Details were summarized in Table 2.2. The experiment setup was shown in Photo 2.2. On the left, the bed

was just leveled after the installation of the impermeable spur dykes. On the right, the desired flow depth was just achieved in the permeable experiment by slowly supplying water from the downstream. The coordinate system for the data collection and analysis was shown in Fig. 2.5.

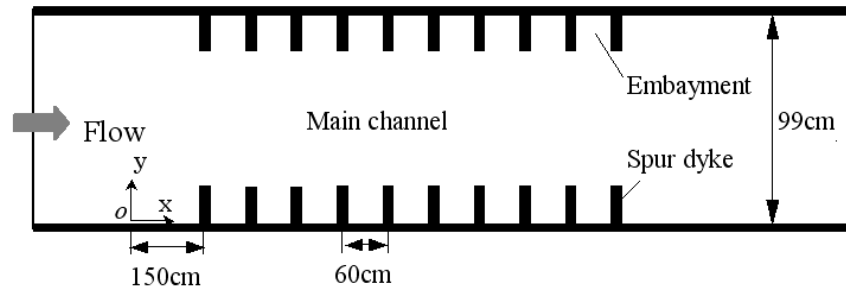


Fig. 2.5 Coordinate System Used in the Measurements

2.3 Results and Analyses

2.3.1 Qualitative observations

(1) Development of local scour

The impermeable spur dykes created great flow disturbances. Due to the artificial narrowing, the velocity in the main channel was significantly intensified. At the heads of the spur dykes, the flow diverted, which led to severe local scour in front of the structures. In particular around the first pair of the spur dykes, the scour holes were developed soon after the pump started. In several minutes, these two scour holes met together at almost the center of the channel and became deeper and deeper. Local scour holes also occurred at the toes of the other spur dykes, but their developing speed was quite slow. Sediment in the embayment areas was found to be eroded near the spur dykes, which might indicate that there were strong return currents. As time passed, the sediment in the proximity of the first pair of the spur dykes was completely eroded. After that the deepest scour holes continued to develop on the glass bed.

In the permeable case, scour holes were also observed at the head of the first spur dyke (i.e. around the piles at the head) on either side. However these scour holes developed very slowly and they seemed to extend to the downstream rather than to the main channel. The water surface gradient was found to be very steep between the upstream and downstream of the piles at the beginning. With the removal of sediment around the piles, this gradient became mild. The deepest scour holes were found at the interface between the main channel and the embayment

area, which was quite different from the impermeable case.

(2) Visualized flow pattern at equilibrium states

The surface flow structure at equilibrium conditions was visualized with tracers. In the case of either impermeable spur dykes or permeable spur dykes, there seemed to be three different zones in the spur dyke stretch. Namely, a main channel zone, an embayment zone and a mixing zone in-between them. However, the details of the flow pattern were different in the two cases.

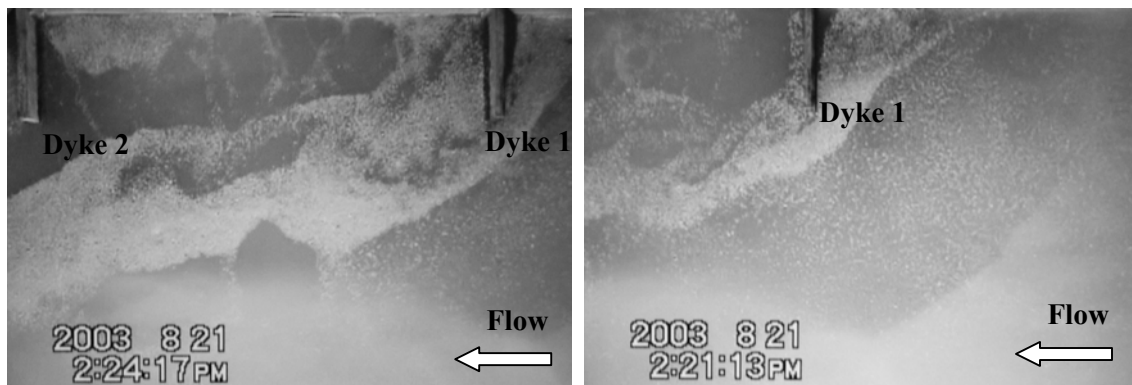


Photo 2.3 Surface Flow Visualization of the Impermeable Case

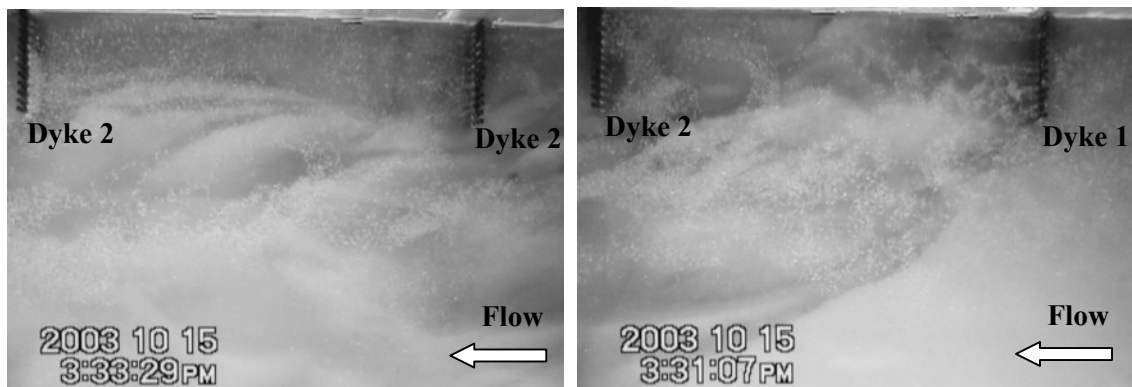


Photo 2.4 Surface Flow Visualization of the Permeable Case

Photo 2.3 showed the instantaneous movement of the tracers in the proximities of the first embayment (Left) and the first spur dyke (Right) on the right side of the channel for impermeable case. There generated a strong clock-wise circulation when the flow approached the first spur dyke. In the first embayment, the flow velocity was decreased. Near the corner, some horizontal circulations could be observed. Different from many fixed bed experimental results (e.g. Uijtewaal et al., 2001 and Kurzke et al., 2002), the tracers at the center area of the embayment seemed to divert and transport in three directions: downstream, upstream and

towards the main channel. This indicated that the flow pattern was much complex and strongly three-dimensional. The degraded bed might be responsible for this phenomenon. Most of the tracers were finally collected in the mixing zone and transported downstream by a series of vortices which were generated immediately after the flow detached from the first spur dyke.

In Photo 2.4, the flow velocity in the vicinities of the first (Right) and the second (Left) embayments were visualized for the permeable case. After the flow passed the first spur dyke, the longitudinal velocity was still predominant. Nevertheless, the movement of the tracers became slower in the embayment, while in the main channel area, faster. The mixing zone also transported a series of vortices, but these vortices were much weaker than the impermeable case. The velocity in the second embayment was found to be much smaller than that in the first one. And the main channel velocity was significantly increased correspondingly. This indicated that the flow velocity was gradually reduced in the embayment from upstream to downstream.

2.3.2 Bed deformation and surface variation

The final bed contour for both impermeable case and permeable case was shown in Fig.2.6. More intuitive images were given in Photo 2.5.

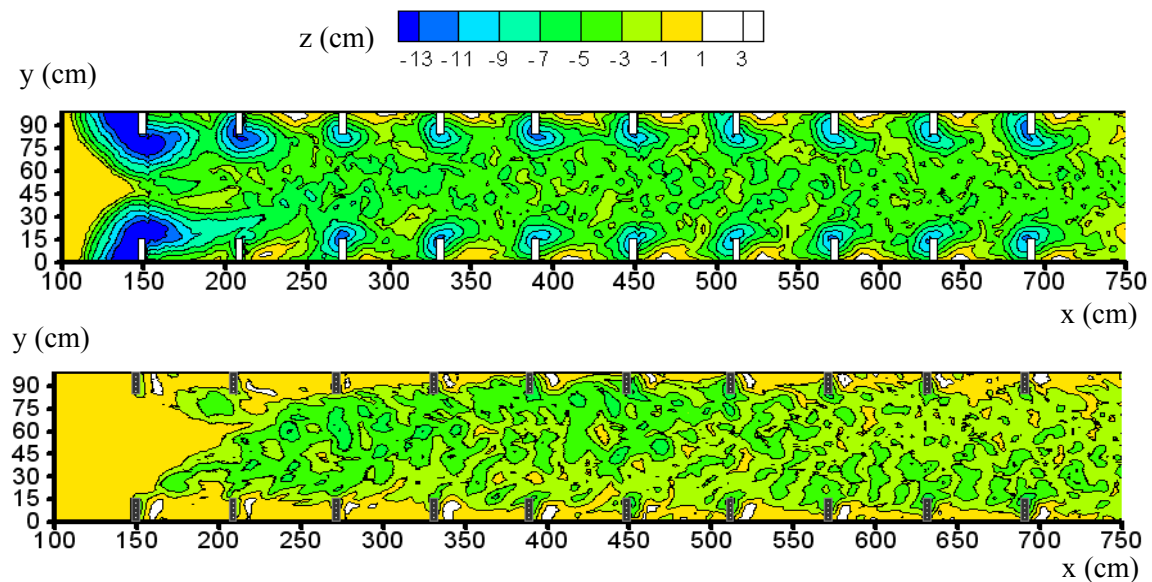


Fig. 2.6 Final Bed Contour (Impermeable, Top; Permeable, Bottom)

In the impermeable case, severe local scour occurred at the toes of all the spur dykes, in particular the first pair. Around the first spur dyke on either side, all the sediment has been eroded and the root of the spur dyke was exposed to water. It was evident that the deepest scour hole depth might exceed 15.0cm if the sand bed was a little thicker. The local scour holes were

also observed to extend to the river banks. It might be attributed to the existing return currents towards the bank after the flow diverted from the heads of the spur dykes. As the consecutive spur dykes were not close enough, these currents attacked the banks. In engineering design and construction, these problems should be paid special attention. If the countermeasures were not effective enough, the structure itself might be undermined besides the bank collapse. Unfortunately, it had to be mentioned that the cost for the additional treatments was generally comparable to that for the structures. Compared with the approaching flow area where the bed level had almost no change, the main channel in the spur dyke stretch was soundly degraded. Very small deposition area was found along the bank in-between the spur dykes. The maximum deposition depth was about 4.8cm.

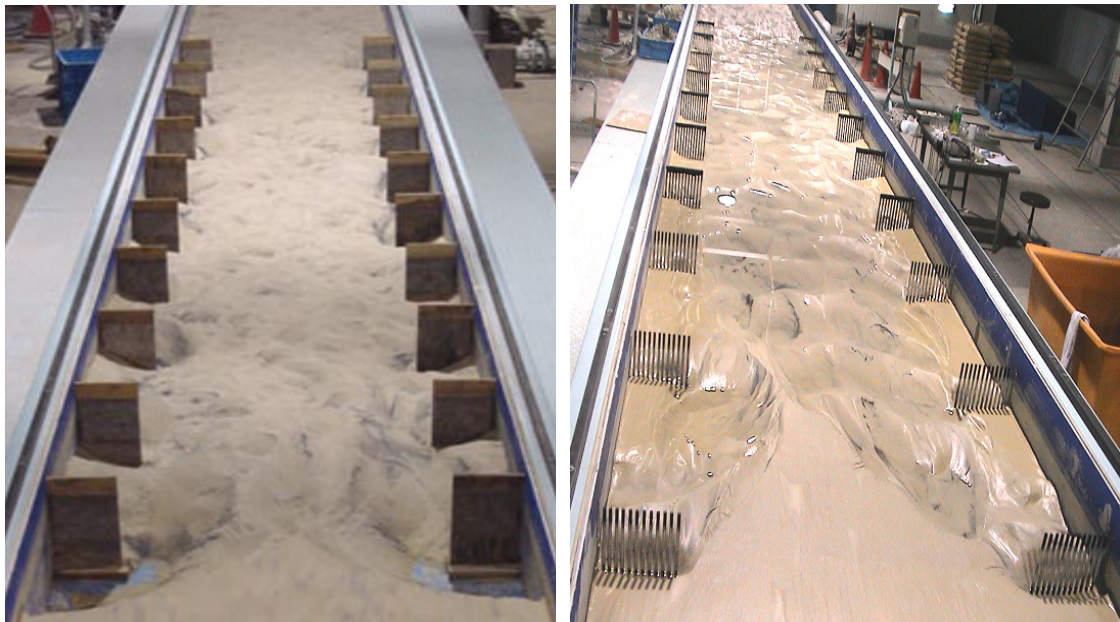


Photo 2.5 Final Bed Morphology (Impermeable, Left; Permeable, Right)

The bed morphology in the permeable case was quite different. The local scour holes around the spur dykes were shallow and not very obvious. The scour along the axes of all the spur dykes had a shape of V. In the reach of the first embayment, great bed deformation was almost confined to the mixing zone area. Downstream of the second pair of the spur dykes, bed degradation was almost limited to the main channel area. This might be due to the fact that after the flow passed the second spur dyke, the velocity in the main channel was dramatically intensified, which has been observed in the surface visualization. It was also noted that although the area very close to the spur dykes was eroded, deposition was found behind the spur dykes. This indicated that the flow passing the spur dykes removed the sediment around the piles and deposited it a little downstream of the spur dyke due to the reduction of velocity. However the

magnitude of both the main channel degradation and embayment deposition was a bit smaller than the impermeable case. The deepest erosion depth was 7.3cm (i.e. less than half of the impermeable case) and the maximum deposition depth was around 3.9cm.

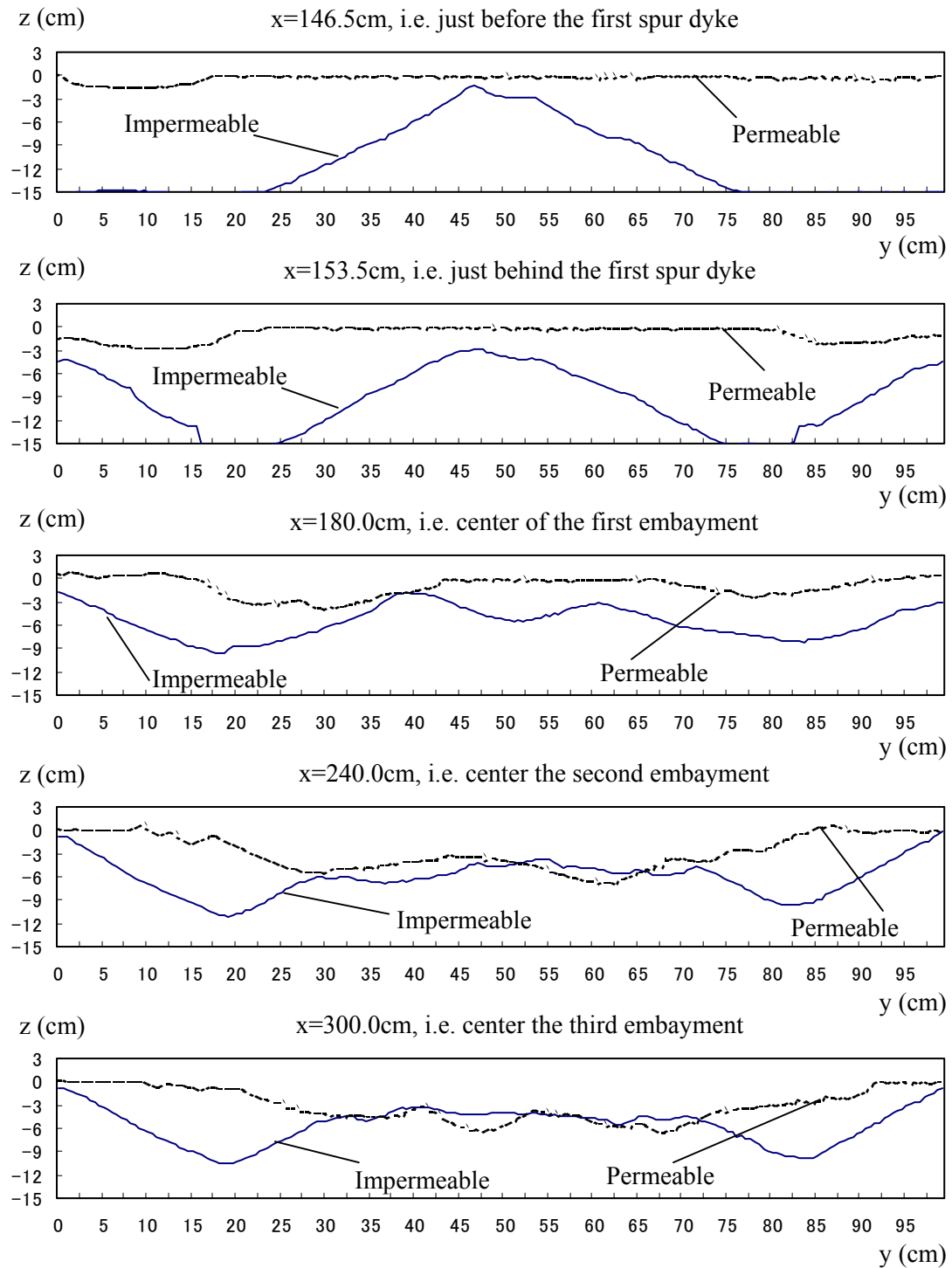


Fig. 2.7 Transverse Bed Profiles at Representative Cross-sections

The bed profiles at some representative transverse cross-sections were given in Fig. 2.7. The impermeable and permeable cases were plotted in the same figure for the convenience of comparison. Due to the severe local scour, the riverbed around the impermeable spur dykes was extremely degraded. While around the permeable spur dykes, the bed elevation was almost maintained. In both cases, the obvious main channel degradation occurred after the flow passed the second spur dyke. It was also interesting to find that the main channel bed degradation induced by the impermeable spur dykes was almost the same as that caused by the permeable ones from the second embayment ($x=240\text{cm}$). These bed profiles gave a clear explanation for the general use of spur dykes, i.e. protection of the river banks and improvement of the channel navigability. However, some special attention should be paid to the impermeable spur dykes. If one took a look at the bed profiles just in front of and behind of the first impermeable spur dyke, one might find that the bed near the channel bank was also significantly eroded. Hence, suitable countermeasures were necessary for bank protection as well.

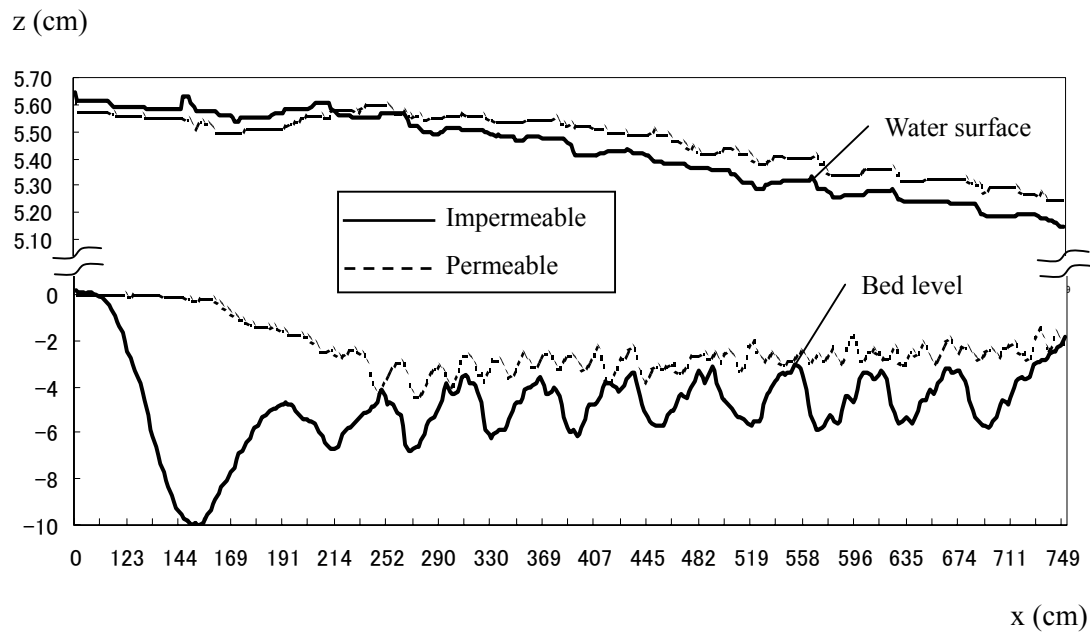


Fig. 2.8 Longitudinal Variation of Mean Bed Level and Water Level in the Channel

Fig. 2.8 demonstrated the longitudinal variation of the mean bed elevation and water level in the channel at the equilibrium state. In both cases, the water surface slope was mild before the second spur dyke. After that, the slope became steeper. Moreover, the water surface slope in the impermeable case was found to be a bit steeper than the permeable case.

Regarding the bed variation, one could take a look at the section along the first spur dyke (i.e. $x=150.0\text{cm}$). It seemed to be quite different from the remaining ones. In the impermeable case,

the mean bed degradation at $x=150.0\text{cm}$ was much larger than the other sections due to the contribution of the most significant local scouring around the first spur dyke. Nevertheless the mean bed level was almost no change at the same location in the permeable case. From the second spur dyke (i.e. $x=210.0\text{cm}$), the pattern of the bed deformation became similar and stable.

Both the surface variation and the bed profile changed at the second spur dyke. It indicated that the most upstream spur dyke (i.e. the first spur dyke) and the following embayment played a special role in a group of spur dykes and deserved *ad hoc* attention. Due to the existence of the first spur dyke, the flow was disturbed either in the direction or the magnitude or both. From a short distance upstream to the embayment just behind the first spur dyke, there provided a space for the flow adjustment. From this viewpoint, the proximity of the first spur dyke served as a transition zone between the approach flow stretch and the grouped spur dyke stretch. A suitable design of this zone will maximize the function of a group of spur dykes. This might include the shape, size and alignment of the first spur dyke, together with the distance of the embayment.

More detailed information on the surface variation was shown in Fig. 2.9 and Fig. 2.10. These were the longitudinal profiles of the surface level at some representative sections.

It was very obvious in Fig. 2.9 that the water level just in front of the spur dykes was much higher than that immediately behind the spur dykes ($y=3\text{cm}$ and $y=10\text{cm}$). The maximum difference was 2.7mm around the first spur dykes at Section $y=10\text{cm}$. From the upstream to the downstream, the water level formed some kind of stairs. If one took a look at one step of these stairs, one might be surprised that the water level seemed to have a tendency to increase. However, if several steps (embayments) were taken into account, one might find that the water level was gradually decreased step by step.

At the interface of the embayments and the main channel ($y=16\text{cm}$), the stairs seemed to disappear. But the differences between the upstream and the downstream side of the spur dykes could be still observed if one took a careful look. When the cross-section was far from the spur dykes ($y=36\text{cm}$ and $y=49.5\text{cm}$), the location of the spur dykes did not directly influence the water level profile any more. In Fig. 2.9, the water level at Section $y=36\text{cm}$ was found to be quite similar to that at Section $y=49.5\text{cm}$. However, there was a water surface slope between Section $y=16\text{cm}$ and Section $y=36\text{cm}$.

Significant water level gradients were observed only around the first spur dyke in the permeable case (Fig. 2.10). The differences were 1.8mm at Section $y=3\text{cm}$, 2.3mm at Section $y=12\text{cm}$ and 1.6mm at Section $y=16\text{cm}$, respectively. These were due to the bow wave formed just in front of the piles. As the velocity in the embayment area was soundly reduced after the flow passed the first spur dyke, and the surface increment in front of the other spur dykes became very small.

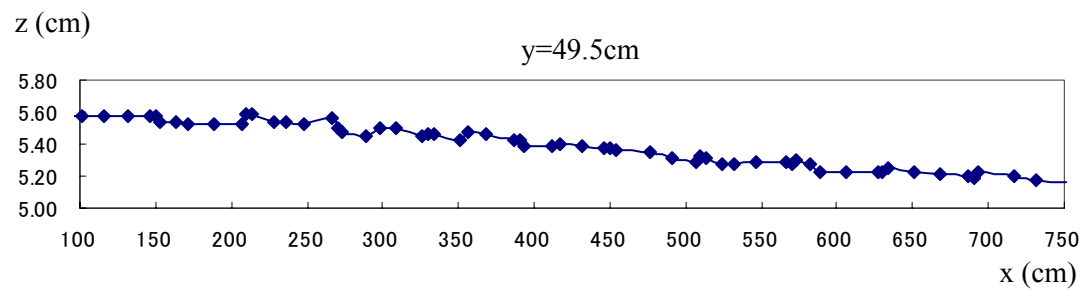
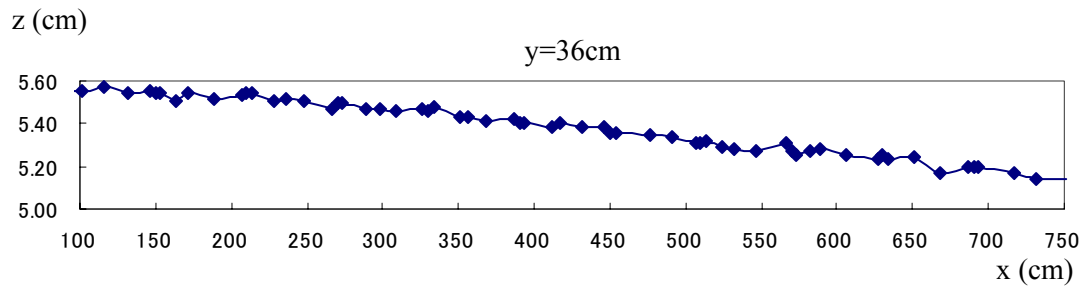
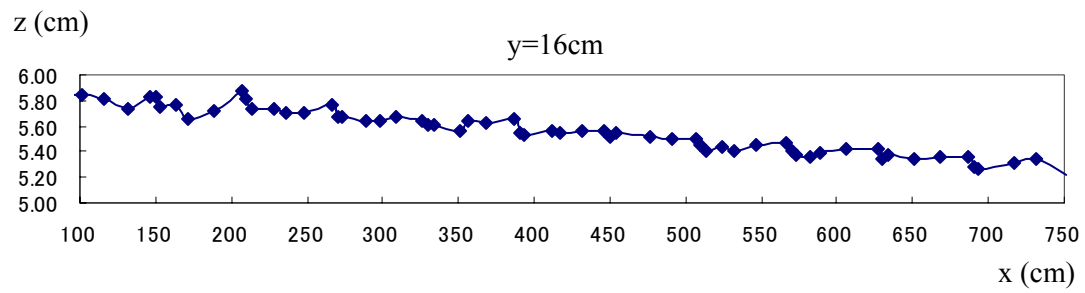
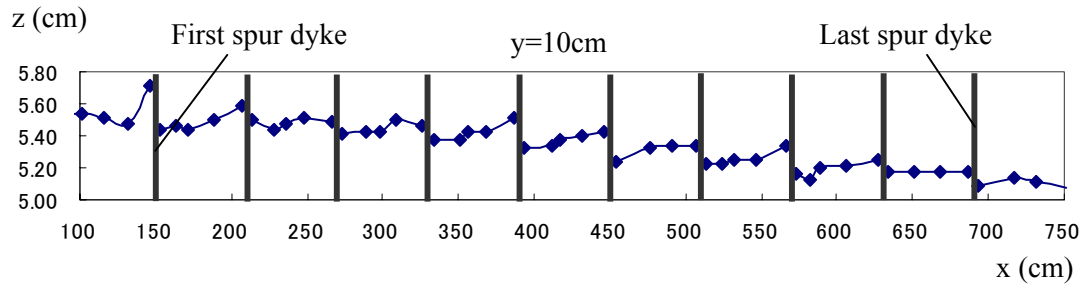
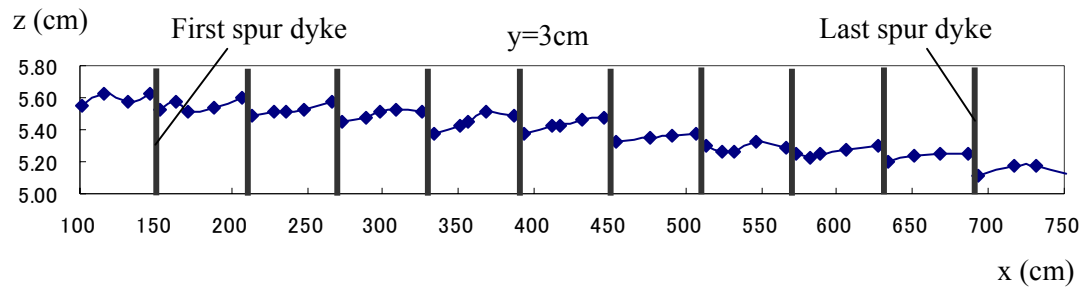


Fig. 2.9 Surface Variation in the Longitudinal Direction (Impermeable Case)

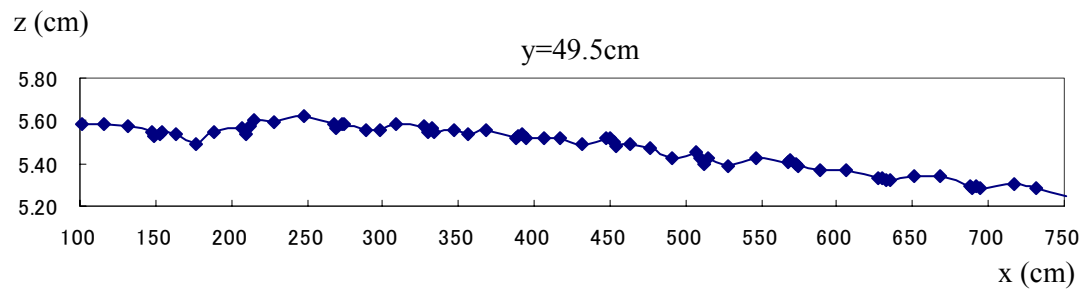
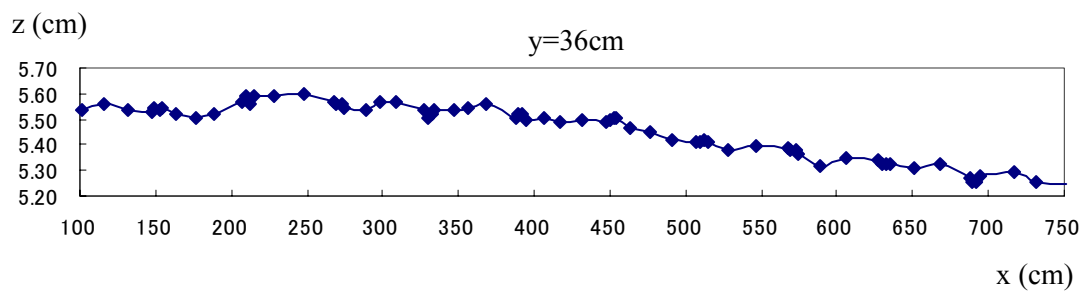
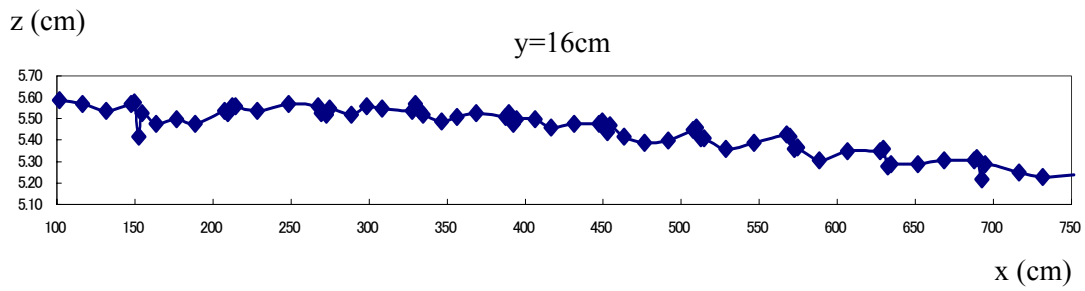
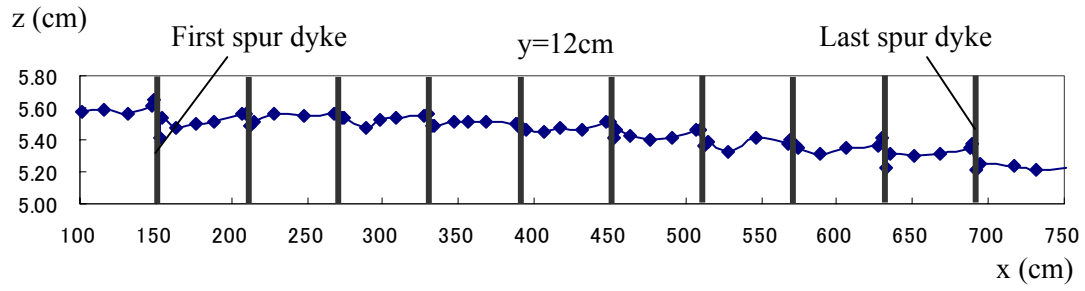
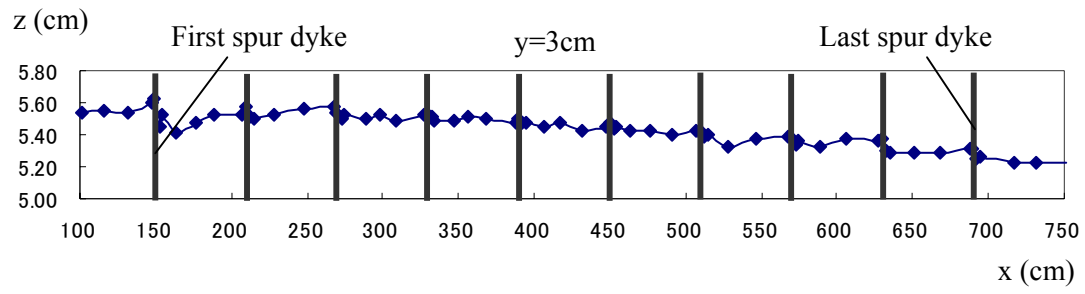


Fig. 2.10 Surface Variation in the Longitudinal Direction (Permeable Case)

2.3.3 Velocity field

From the surface visualization, one could get some ideas about the complexity of the local flow around spur dykes. In this section, a further insight into the 3D characteristics of the velocity field was described with more quantitative information.

(1) Impermeable case

In the vertical direction, the 3D velocity components were measured in two layers. *Ad hoc* measurements were carried out in some of the scour holes in order to acquire the details of the local flow. As the experimental flume was geometrically symmetrical, only half of the study domain was selected during the analyses.

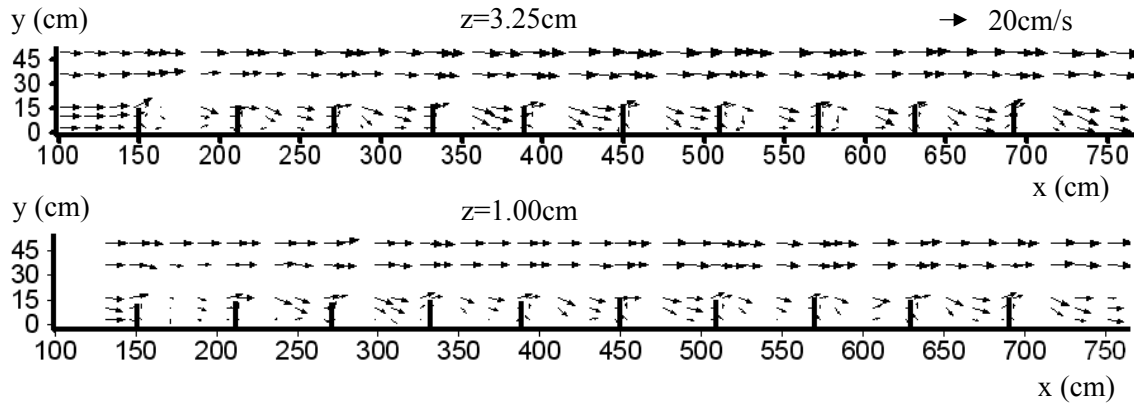
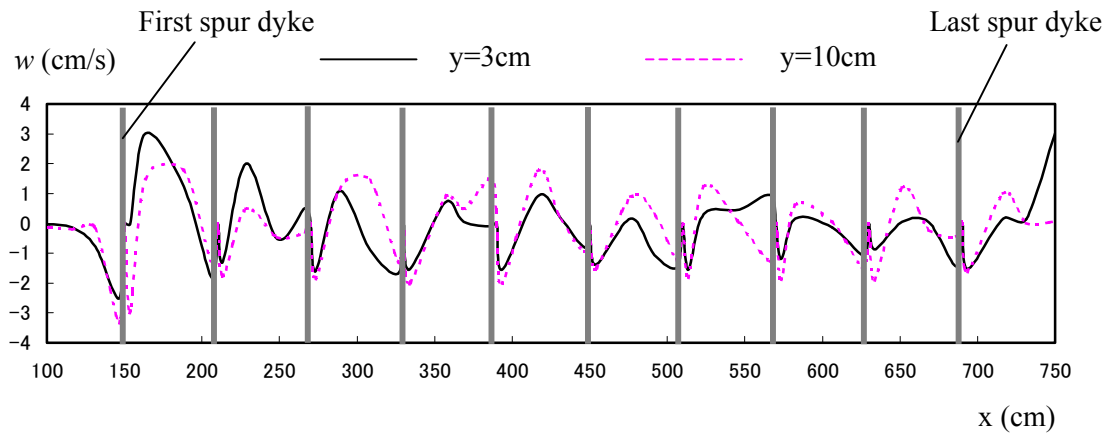


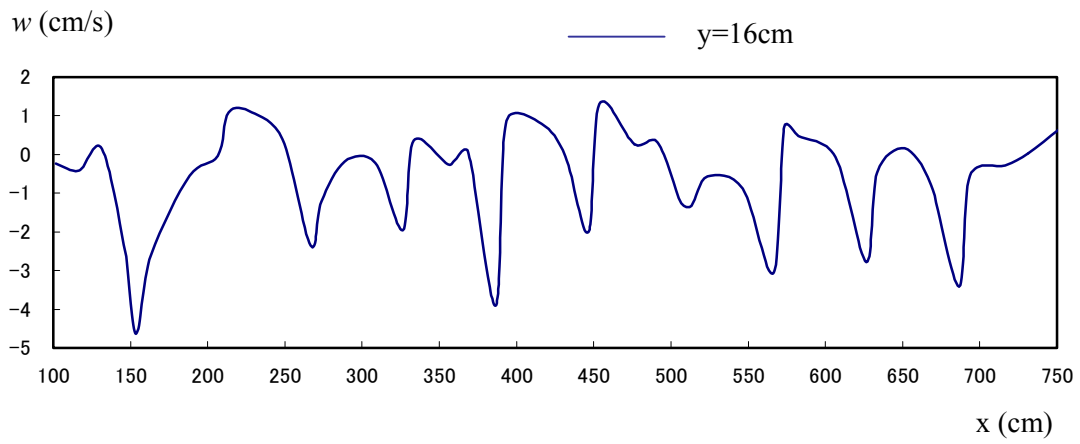
Fig. 2.11 Velocity Vectors (u , v) at Two Different Planes

The longitudinal and transverse velocity vectors were plotted in Fig. 2.11 at different layers with the same scale. The approach flow had a larger velocity in the upper layer. However, it was very obvious that the flow pattern in the two layers was generally similar.

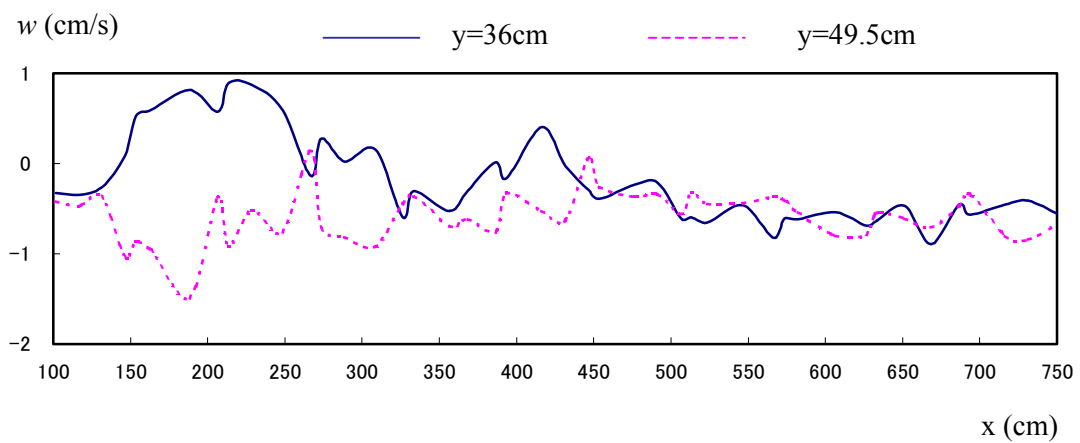
The flow approached the first spur dyke and got separated at the head of the structure. This process was repeated when the flow passed by the other spur dykes downstream. Nevertheless, the diversion angle at the first spur dyke was much larger than the others. In each embayment formed by the two consecutive spur dykes, a circulating flow could be observed. The center of the circulation was not at the geometrical center of the embayment. It was much closer to the upstream spur dyke which formed one of the embayment boundaries. This proved the existence of the wake vortex system behind the spur dykes. If the measured data was more clustered, more circulations might be observed, some of which has been found in the surface visualization. The corresponding vertical velocity profiles were given in the following figures (Fig. 2.12 and Fig. 2.13).



(a) Embayment Area

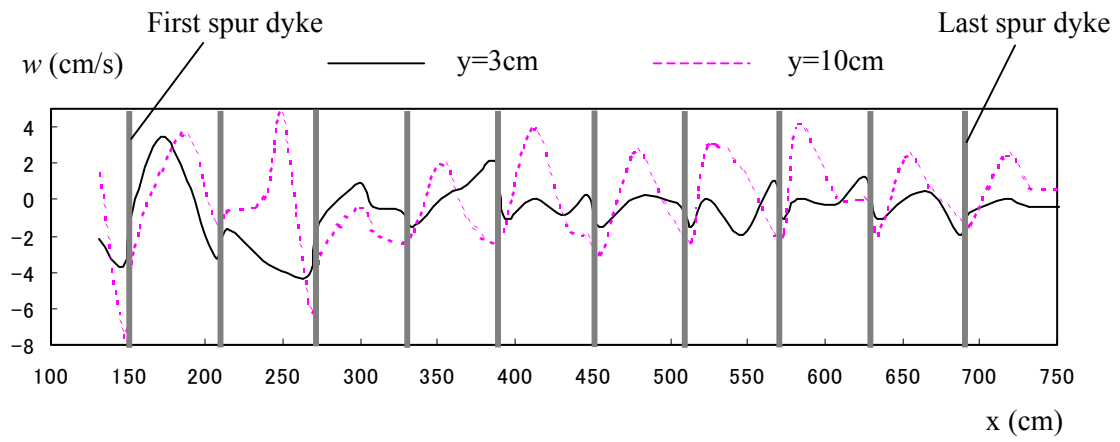


(b) Mixing Zone

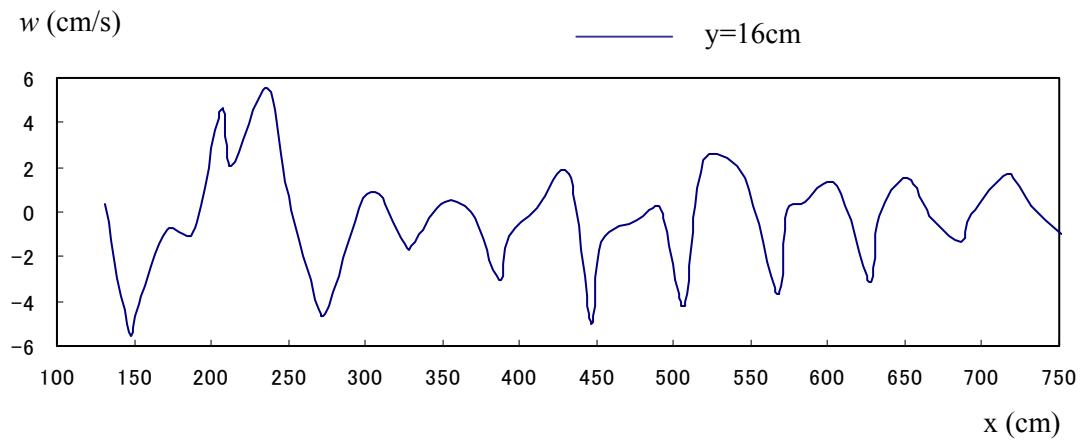


(c) Main Channel Area

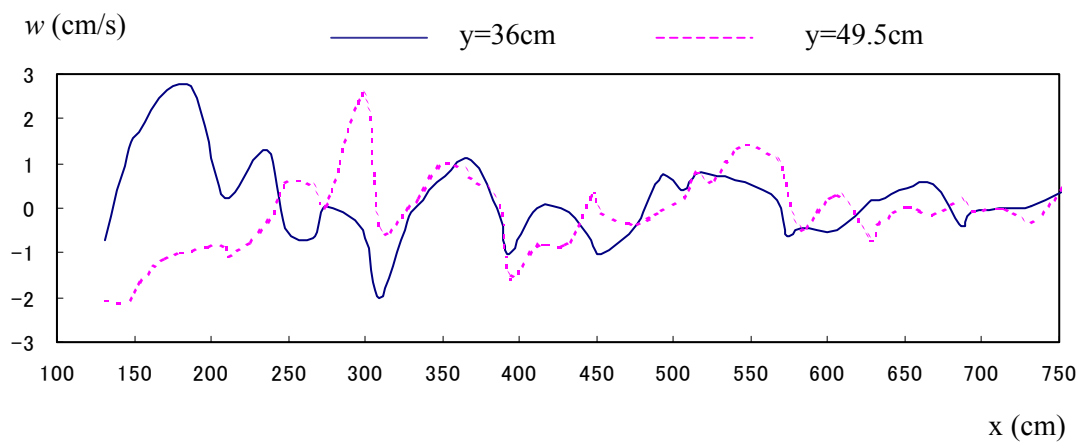
Fig. 2.12 Longitudinal Distribution of Vertical Velocity Profile w at $z=3.25\text{cm}$



(a) Embayment Area



(b) Mixing Zone



(c) Main Channel Area

Fig. 2.13 Longitudinal Distribution of Vertical Velocity Profile w at $z=1.00\text{cm}$

For clarity, the vertical velocity profiles were plotted in three groups according to the location of the cross-section. The measured data at $y=3\text{cm}$ and $y=10\text{cm}$ was grouped because both locate in the embayment area as shown in Fig. 2.12 (a) and Fig. 2.13 (a). It might be observed that the vertical velocity was generally negative (downward) in front of the spur dyke and became positive (upward) just behind the spur dyke. At the first spur dyke, the downward velocity was quite large. This velocity was increased from the upper layer ($z=3.25\text{cm}$) to the lower layer ($z=1.00\text{cm}$), which attacked the riverbed and became one the major agent for local scour.

The vertical velocity at $y=16$ (in the mixing zone) was very complex as shown in Fig. 2.12 (b) and Fig. 2.13 (b). In the vicinity of the first spur dyke, the velocity was downward. And it got to be upward around the second spur dyke. The flow pattern around the remaining spur dykes was a little similar. Besides the effect of the spur dykes, this kind of velocity distribution was also closely related to the complex bed geometries. In the main channel area, two representative sections $y=36\text{cm}$ and $y=49.5\text{cm}$ were plotted in Fig. 2.12 (c) and Fig. 2.13 (c). The magnitude of the vertical velocity became very small in this area. As the bed change in these two sections was not very much, it might be concluded that the vertical velocity near the bed was also small. Compared with the longitudinal velocity, the contribution to the bed deformation by the vertical velocity might be negligible.

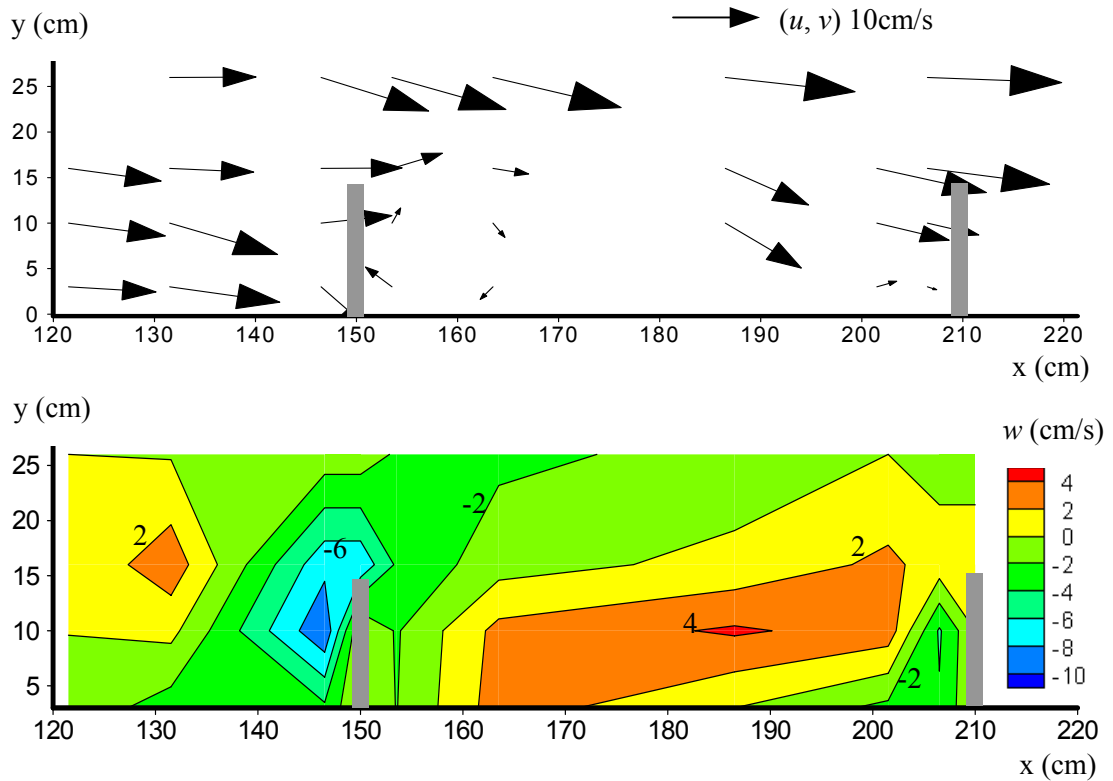


Fig. 2.14 Local Flow Around the First Spur Dyke at $z=-0.4\text{cm}$ (Vectors u, v , Top; w , Bottom)

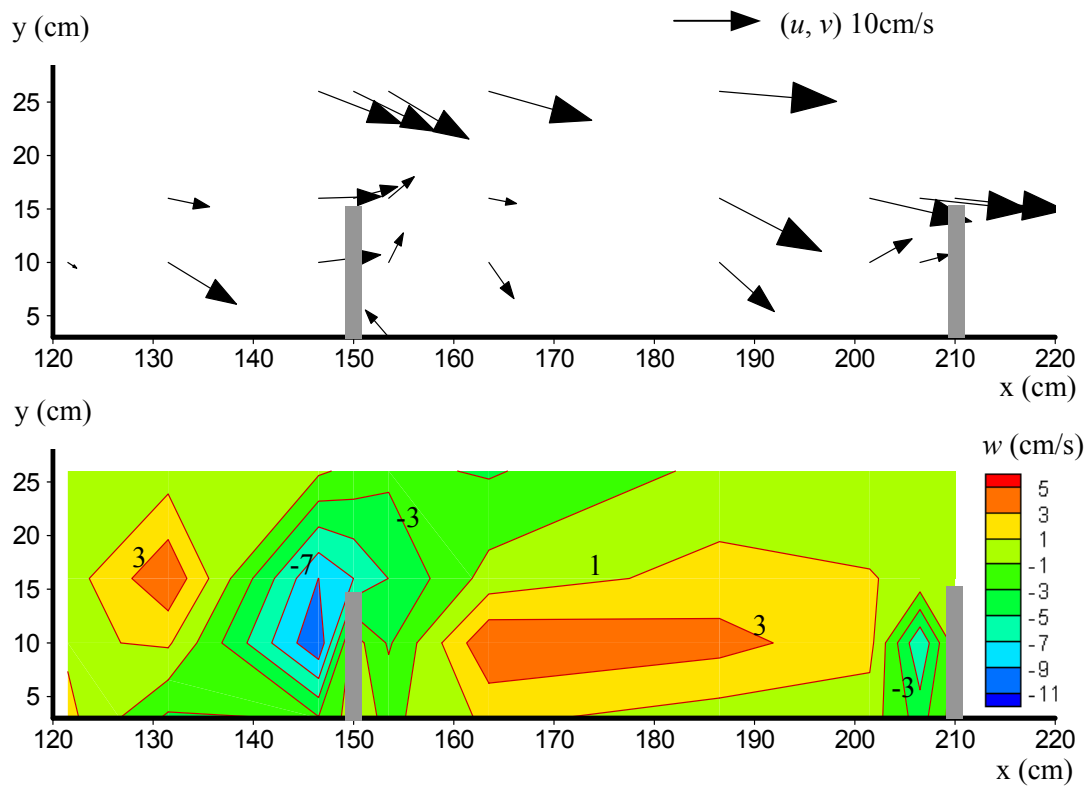


Fig. 2.15 Local Flow Around the First Spur Dyke at $z = -2.4$ cm (Vectors u , v , Top; w , Bottom)

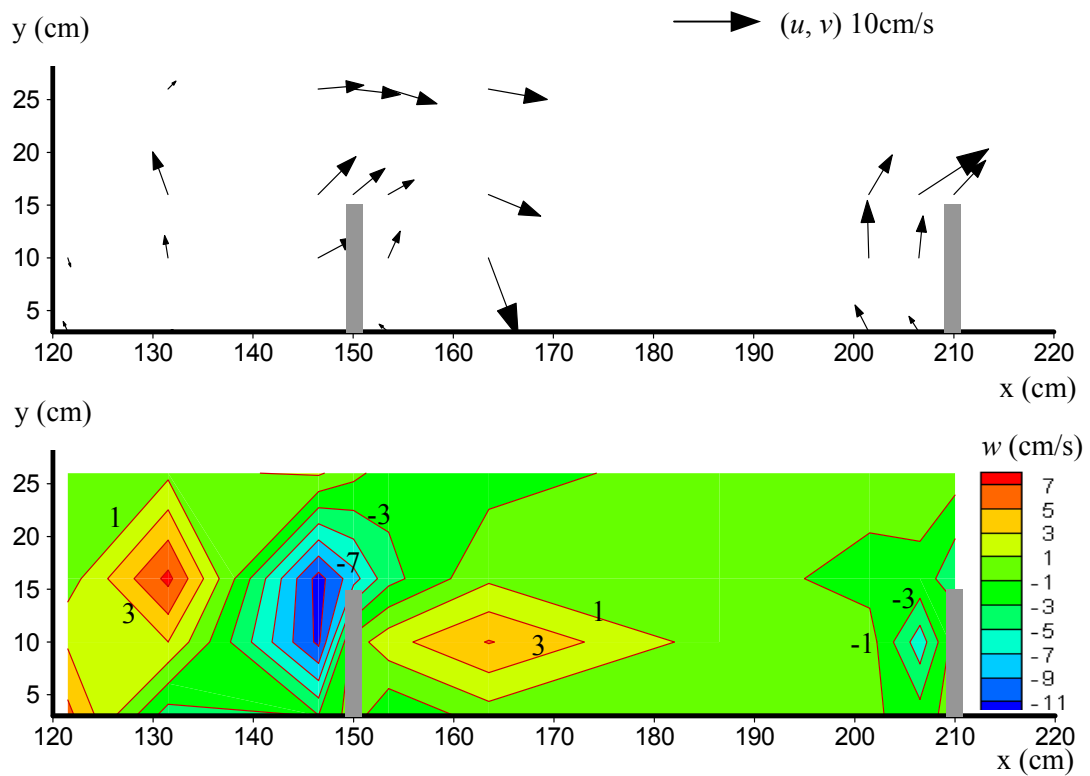


Fig. 2.16 Local Flow Around the First Spur Dyke at $z = -5.4$ cm (Vectors u , v , Top; w , Bottom)

From Fig. 2.14 to Fig. 2.16, the local flow in the proximity of the first spur dyke was depicted. In each figure, the longitudinal and transverse velocities were plotted as vectors and the isolines were shown for the vertical velocity. There were three layers with different elevations, i.e. $z=-0.4\text{cm}$ (Fig. 2.14), $z=-2.4\text{cm}$ (Fig. 2.15) and $z=-5.4\text{cm}$ (Fig. 2.16). A value of zero was given to the point which had a elevation higher than that of the corresponding layer.

The 3D characteristics of the local flow were evidently observed from these figures. The magnitude of the three velocity components was comparable at the head of the first spur dyke. In some areas, the vertical velocity was even larger than the other two. From the upper layer to the lower layer, the vertical velocity was significantly increased just in front of the first spur dyke. This further gave the transport route of the strong downward flow. The wake vortex system observed in Fig. 2.11 was again appeared behind the first spur dykes. This kind of vortex system acted like a vacuum cleaner to suck the bed sediment into the main flow. The return currents in the first embayment were a bit strong, which might be the major cause for the erosion near the bank. At the downstream corner area of the first embayment, a small vortex was also observed in all these layers. But this vortex was very weak.

The primary vortex (i.e. the horseshoe vortex) was also observed during this experiment. Fig. 2.17 showed the structure of this vortex projected on a vertical plane just in front of the first spur dyke.

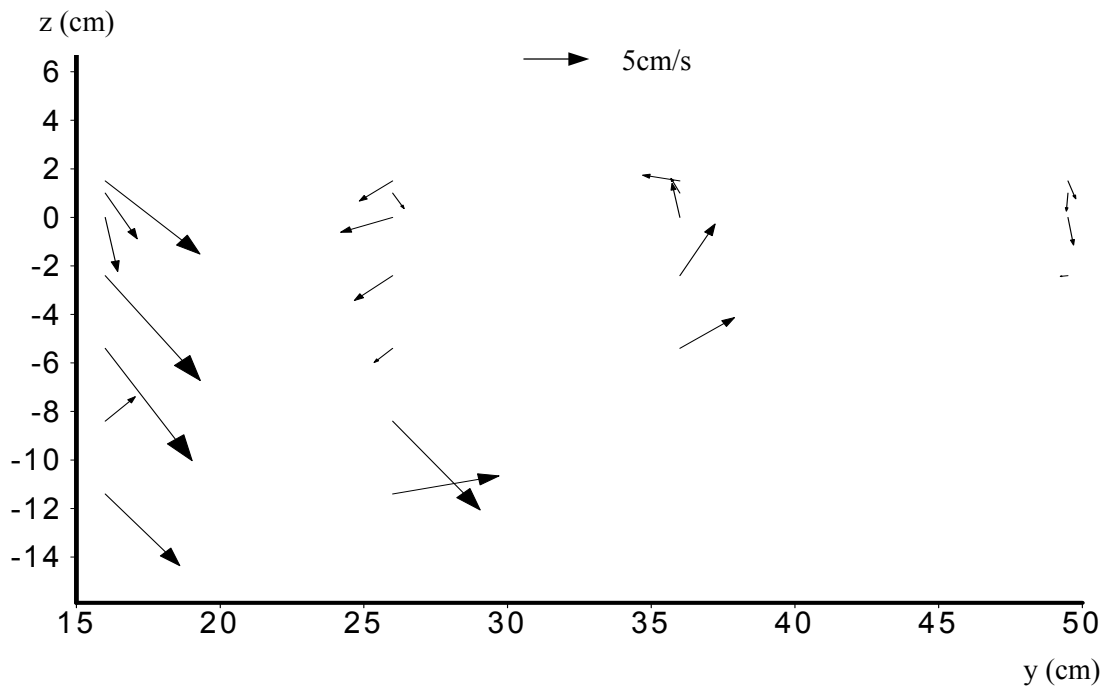


Fig. 2.17 Velocity Vectors (v , w) at a Transverse Cross-section in front of the First Spur Dyke (Section $x=150\text{cm}$)

It might be seen from Fig. 2.17 that the vortex was very strong. No wonder the sediment in this area has been completely eroded.

(2) Permeable case

In the permeable case, the velocity components were also measured in two layers with different elevation: $z=3.25\text{cm}$ and $z=2.00\text{cm}$. The velocity vectors (u, v) were plotted in Fig. 2.18 and Fig. 2.19. The vertical velocity profile was shown in Fig. 2.20 and Fig. 21.

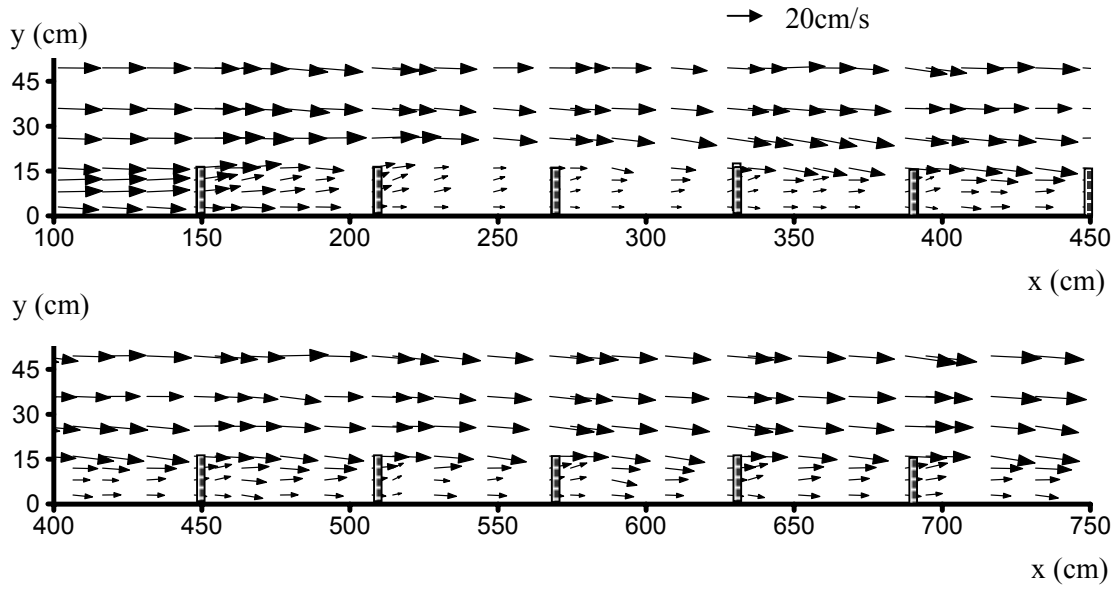


Fig. 2.18 Velocity Vectors (u, v) at $z=3.25\text{cm}$

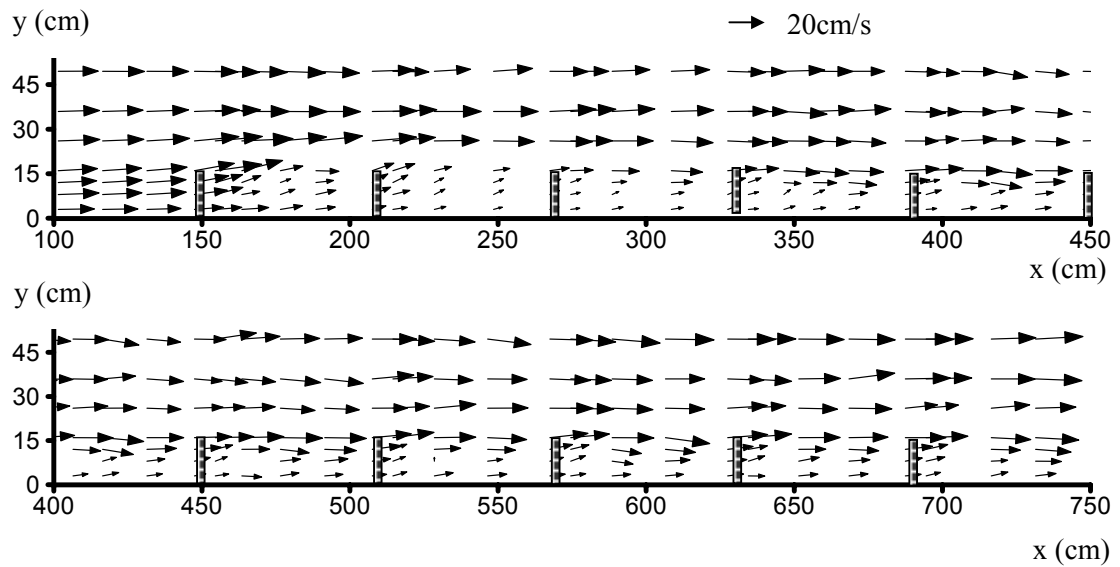
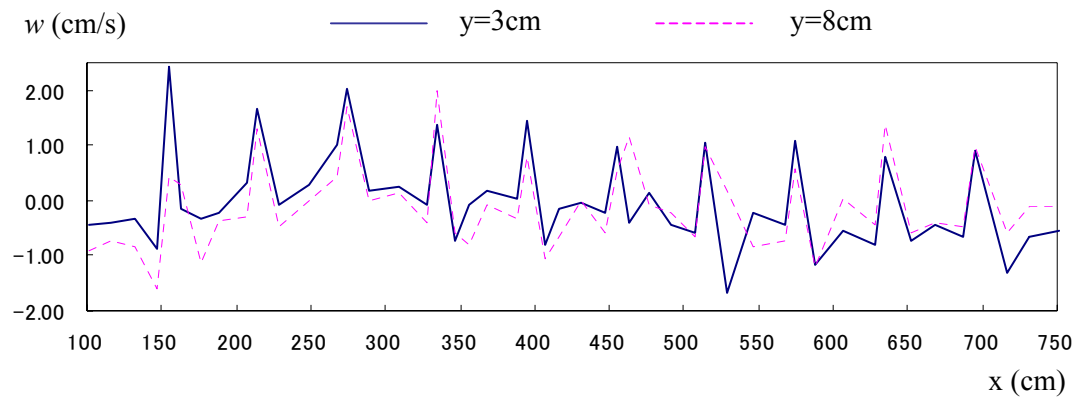
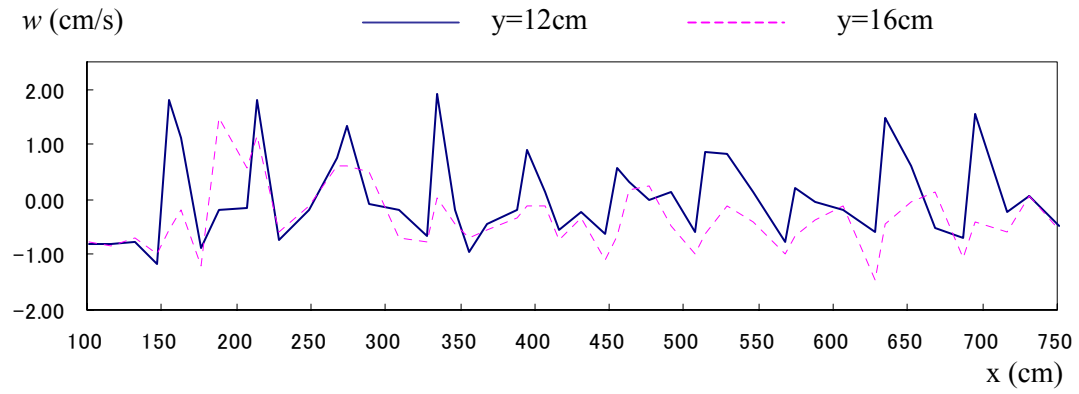


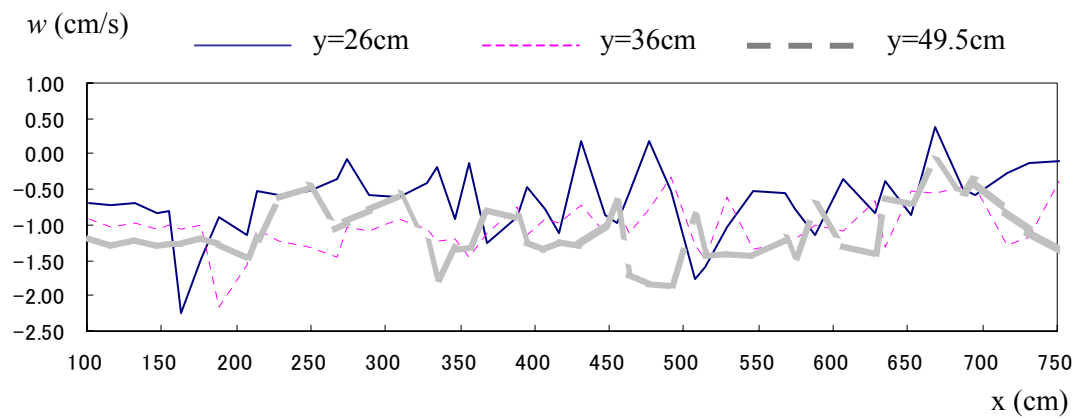
Fig. 2.19 Velocity Vectors (u, v) at $z=2.00\text{cm}$



(a) Embayment Area

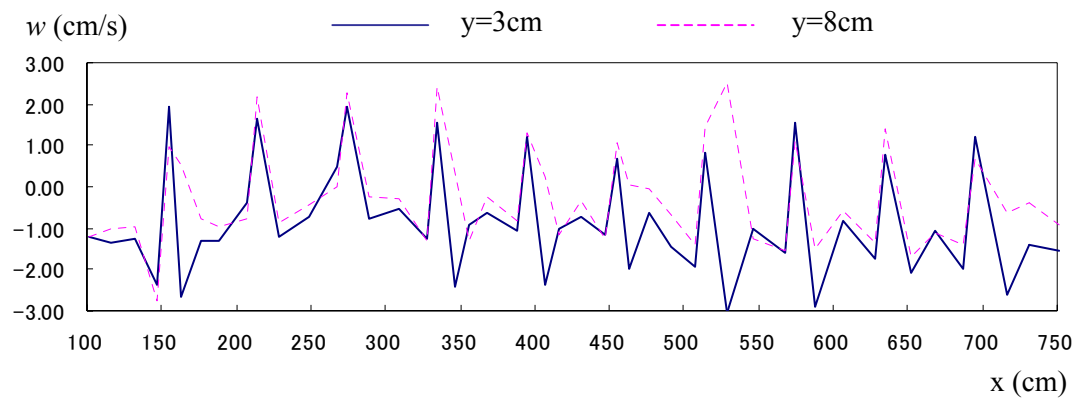


(b) Mixing Zone

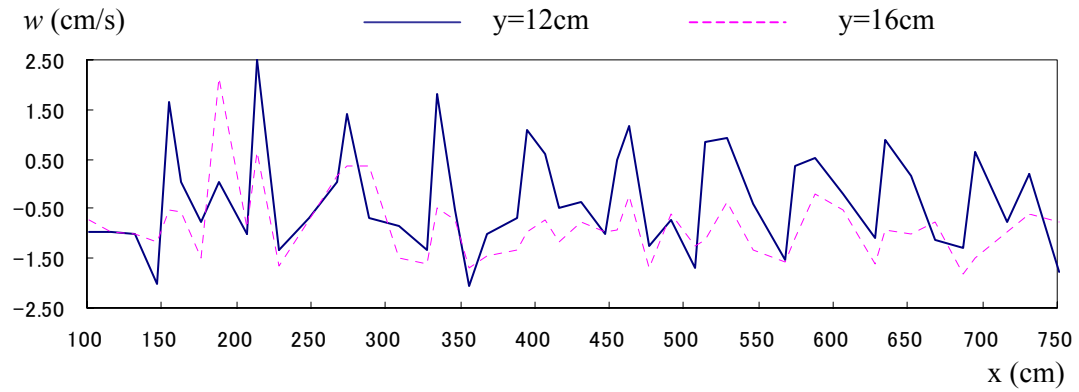


(c) Main Channel Area

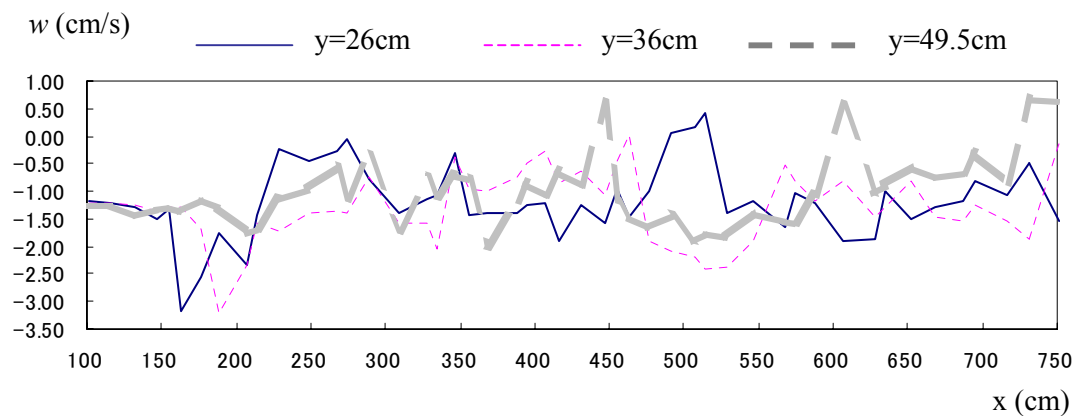
Fig. 2.20 Longitudinal Distribution of Vertical Velocity Profile w at $z=3.25\text{cm}$



(a) Embayment Area



(b) Mixing Zone



(c) Main Channel Area

Fig. 2.21 Longitudinal Distribution of Vertical Velocity Profile w at $z=2.00\text{cm}$

From Fig. 2.18 and Fig. 2.19, such conclusion could be drawn that the permeable spur dykes did not significantly change the flow direction. However, a series of permeable spur dykes led to great velocity reduction in the embayment area and soundly velocity increment in the main channel. It was also noted that the velocity reduction was gradually achieved as has mentioned in the previous surface flow visualization. After the flow passed the third spur dyke, the flow velocity in the embayment was only about 20% of the original one. As a result, most of the embayment areas almost maintained the original riverbed elevation when the equilibrium condition was reached. On the other hand, the increased flow velocity in the main channel resulted in great bed degradation. Although the piles of the permeable spur dykes worked as local obstructions, the local scour holes were limited to a small area in the proximity of the piles. Permeable spur dykes did not lead to great flow disturbances, the function of which was a little similar to some kind of roughness element. And the bed morphology was also simpler than the impermeable case. If one had a look at Fig. 2.20 and Fig. 2.21, one might find that the vertical velocity was generally very small. This meant that the longitudinal velocity was still predominant in channels with permeable spur dykes.

2.4 Summary

The results of an experimental study of the flow structure and bed deformation around a series of non-submerged impermeable and permeable spur dykes are presented. Their influence on the flow and bed under the same hydraulic conditions are compared and analyzed. Both kinds of spur dykes are able to increase the main channel velocity and decrease the velocity in the embayment area. As a result, either of them is possible to find an application in the campaign of river training and river restoration.

Compared with a single spur dyke whose influence on the flow and channel bed have been summarized in Chapter 1, a group of spur dykes shows different characteristics. With a single spur dyke, the local effect on the flow is the most important. However, a series of spur dykes are not only some kind of local obstructions but also an artificial narrowing of the channel width. These two aspects lead to the final flow pattern and the bed morphology. Local scour prediction formulae are questionable if they are used to predict the scour due to a group of spur dykes without any correction. The complex interaction between the flow, sediment transport and bed evolution necessitates more advanced numerical methods.

But for the most upstream spur dyke, the effect of the local obstruction is still dominated. The most upstream spur dyke in a group is the most important one during the engineering design. The most upstream spur dyke may behave like an individual structure if it is organized far from the succeeding one in a group. In the current impermeable experiment, the distance

between the most upstream spur dyke and the immediately following one seems to be a little large. (The aspect ratio, i.e. the ratio of the embayment length to the spur dyke length, is about 4) As a result, obvious return currents caused dangerous erosion near the bank. The behavior of the upstream spur dykes will affect the downstream ones by disturbing the flow pattern and breaking the sediment equilibrium. This effect will be very strong for the spur dyke immediately following the most upstream one. Nevertheless, the flow and the bed variation will tend to be stable after some adjustments of several spur dykes. In the current experiments, a relatively stable state is achieved after the flow passes three spur dykes in both impermeable and permeable cases. In this stable area, the intensified velocity in the main channel governs the main channel degradation, while the local scour around spur dykes is a result of the effect of both the local obstruction and the channel narrowing.

From the standpoint of river training, the permeable spur dykes have some advantages over the impermeable ones. The impermeable spur dykes are passive hydraulic structures which are followed by the extreme local scour at the toes of the structures and the potential erosion of the river bank due to return currents. Suitable protection countermeasures are usually mandatory in the design stage. And these *ad hoc* treatments may result in a significant increment of the project investment. The relatively long construction time as well as the great consumption of construction materials is also a defect compared with the permeable spur dykes. Subject to the cost constraints, the permeable spur dykes will lead to an economically attractive solution for river training. On the other hand, diversity of scour holes and stable pools are generally desirable in river restoration. By enlarging the scour hole in a stream restoration project, Shields et al. (1995) found a great increasing of fish numbers, size, species and the area of the aquatic habitat. From this viewpoint, the final bed morphology and velocity distribution resulted from the impermeable spur dykes should be preferred to create a suitable environment for the spawning, feeding and living of the biological species.

The drawbacks of these two kinds of spur dykes can be partly overcome by optimizing the shape of the structure, selecting the aspect ratio, adjusting the protruding angle, changing the permeability of the structure etc. (e.g. the experimental study of Kuhnle, et al., 2002). However, in order to achieve the maximum beneficial effects to the riverine ecosystem and still afford an effective control of the river, the combination of the impermeable and permeable spur dykes seems to be a more promising alternative. This has been confirmed by Khaleduzzaman (2004), Takeuchi (2004) and Ito (2005) who tested different kinds of combined structures such as top blocked permeable spur dykes and bandal-like structures with experimental methods. However, in their test cases, the combined structure becomes so complex that it poses new challenges for construction and other unexpectable problems. It is also possible to use both impermeable and permeable spur dykes in one group. For example, the most upstream one is a permeable spur

dyke while the remaining ones are impermeable. The applicability of this kind of combined methods deserves further investigations. Moreover, this chapter has limited the discussion to non-submerged spur dykes. In actual cases, the spur dykes tend to suffer from the over-topping flow. The interaction between the flow and sediment transport has been reported to be more sophisticated (see e.g. Muto et al., 2003 and Kitamura, 2003). In this thesis, a calculation for a submerged spur dyke will also be given later.

References

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Chapter 3

Hydrodynamic models

3.1 Introduction

Experimental methods (including both flume experiments and physical scale model experiments) have achieved a great development in the past several decades. Nevertheless, flume experiments are generally carried out in laboratories under idealized conditions as has mentioned before. For instance, steady flows, uniform sediments, simplified geometries, etc. Therefore their applications to field situations may still be problematic and may produce questionable results (Yen et al., 2001). Physical scale model experiments can overcome some of the shortcomings, but they usually involve large expenditure and are time-consuming in the model construction and experimentation. As a result, they are usually used in some important projects, for instance, the sediment experiments of the neighborhood of the TGP (Three Gorges Project) (Tsinghua University, 1996). Furthermore, the accuracy is also limited due to the scale distortion which is almost unavoidable whenever the sediment transport is involved (Chang, 1988). In view of these respects, a promising alternative is to take recourse to the numerical methods, which are attracting more and more attention with the progress in both the fluid dynamics and computer techniques. A numerical model usually consists of a water routine and a sediment routine. The two routines work together in a coupled or decoupled way and are capable of predicting the flow field and the sediment behavior. The hydrodynamic models employed in the water routine will be presented in this chapter and the sediment transport models in the sediment routine are discussed in the next chapter.

As is known, the selection and application of hydrodynamic models are strongly related to the problem studied, e.g. the scale of the problem, the available data and the required accuracy. From the viewpoint of the dimensionality, there are options of one-dimensional (1D), two-dimensional (2D) and three-dimensional (3D) models. 1D models are widely used in practices and may be applied to large river systems. They are able to provide mean quantities of the cross-sections with a relatively short computational time. Moreover, the information from the 1D models may also serve effectively as the initial input and physical constraints for more elaborate models. 2D models can be further divided into 2D depth-averaged models (2DH) and

2D simulations in the vertical plane (2DV). The former type is of particular interest in situations where the flow field shows no significant variations in the vertical direction. While for flows uniform in the lateral direction but with significant changes along the vertical direction, the latter one is more popular (van Rijn, 1993). 3D models are very powerful, but unfortunately they are also the most costly ones in terms of computational resources and time. They are not practicable for long river stretches so far and are more suitable for the prediction of local phenomena.

In view of these arguments, models of different levels are commonly combined to solve the problems in field situations. In this study, both 1D and 3D models are developed considering the characteristics of the flow and sediment transport in channels with spur dykes. The 1D model can be used for the long-term prediction of the river system, while the local flows around the spur dykes are resolved with the 3D model. As the 1D flow model has been well developed and widely applied in hydraulic engineering, only a brief introduction will be given in this chapter. Detailed information may be found in many related textbooks (e.g. Chang, 1988 and Jain, 2001). The emphasis here is put on the newly developed 3D model based on an unstructured mesh.

3.2 One-dimensional flow modeling

3.2.1 Governing equations

The continuity and momentum equations governing the 1D open channel flow are presented as below.

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q \quad (3.1)$$

$$\frac{1}{gA} \frac{\partial Q}{\partial t} + \frac{1}{gA} \frac{\partial}{\partial x} \left(\frac{Q^2}{A} \right) + \frac{\partial}{\partial x} (z_b + h) + I_e = \frac{Q}{gA^2} q \quad (3.2)$$

where A = the cross-sectional area; t = time; Q = discharge; x = the longitudinal coordinate component; q = the lateral inflow; g = the gravitational acceleration; z_b = the elevation of the channel bed; h = the water depth and I_e = the energy slope.

In the long-term prediction of the river system, the flow is generally assumed to be steady, and the calculation may be simplified to compute the water-surface profiles at successive time steps. The other hydraulic parameters such as the velocity and the hydraulic radius are obtained from the corresponding water-surface profile. The governing equations are then reduced to

$$J = -\frac{dH}{dx} = -\frac{d}{dx} \left(Z + \alpha \frac{u^2}{2g} \right) \quad (3.3)$$

where J = the energy gradient (i.e. the head loss per unit channel length); H = the total energy head; Z = the water surface elevation; u = the mean velocity in the cross-section and α = the energy coefficient.

3.2.2 Solution methods

Appropriate boundary conditions should be prescribed to close the governing equations. Practically, the hydrograph is usually known at the upstream boundary, and at the downstream boundary the stage-discharge relation or the bed variation is specified. The FDM (Finite difference method) is usually used to discretize the governing equations. It is easy to find that Eq. 3.3 for each cross-section has a finite-difference form as

$$\bar{J} \Delta x = \left(Z_i + \alpha_i \frac{u_i^2}{2g} \right) - \left(Z_{i+1} + \alpha_{i+1} \frac{u_{i+1}^2}{2g} \right) \quad (3.4)$$

where subscript i = index of the cross-section counted from upstream to downstream. Δx = distance between sections i and $i+1$; $\bar{J}$ = averaged energy gradient between sections i and $i+1$.

The solution procedure follows the commonly used standard-step method, which is a trial-and-error approach. In the computation, hydraulic parameters such as the flow area, the wetted perimeter and the roughness are expressed in terms of the water stage for each cross section. The water surface computation is forward section to section in different directions depending on the flow condition (subcritical or supercritical).

3.3 Three-dimensional flow modeling

3.3.1 Introduction

(1) 3D turbulence models

Although there is usually an intuitive feeling that a turbulence model in essence is to find way to solve the Navier-Stokes equations, it poses lots of interesting and challenging problems. Till now, mathematical modeling of the turbulent flow is one of the thorniest fields in the hydraulic engineering. As the Navier-Stokes equations provide enough equations for the unknowns,

directly resolving the whole spectrum of turbulent scales is hence theoretically possible. The method solving the Navier-Stokes equations directly is termed DNS (Direct numerical simulation). However DNS leads to an extreme consumption of computer resources and is unlikely to be attainable for engineering use recently or even in the foreseeable future. Two alternatives then occurred to transform the Navier-Stokes equations in such a way that the small-scale turbulent fluctuations do not have to be directly simulated: time-averaged (also known as Reynolds-averaged) and space-filtered. Both methods introduce additional unknown terms that need to be modeled in order to achieve a closure. The former transformation results in RANS (Reynolds-averaged Navier-Stokes equations) models and the latter leads to LES (Large eddy simulation). LES uses very fine grid to resolve the larger eddies, and employs turbulence models only for the smaller scales. The rationale behind LES is: (a) the large eddies are the most effective transporters of the conserved properties and are mostly dictated by the geometries and boundary conditions of the flow involved; and (b) small eddies are usually much weaker and tend to be more isotropic, consequently it is relatively easier to construct a universal model. Literatures related to LES are undergoing considerable expansion nowadays. A recently published monograph by John (2004) provides a comprehensive study of three LES models: the Smagorinsky model, the Taylor LES model and the rational LES model. However, the computation burden is still a bottleneck for the use of LES in the engineering practice.

An engineer is usually concerned with the time-averaged properties of turbulences (Launder and Spalding, 1972). This suggests ways of accounting for the turbulences on the mean flow behavior by solving the RANS, which represents transport equations only for the mean flow quantities, with all the scales of the turbulence being modeled. Furthermore, RANS models are relatively simple to achieve and may greatly reduce the computational effort compared with DNS and LES. As a result, RANS models are considered to be the standard approach for industrial CFD applications. RANS models may be again divided into many sub-categories such as zero-equation models, one-equation models, two-equation models and ARS (Algebraic Reynolds stress simulation) models. Amongst these models, the $k - \varepsilon$ model, together with its variants, is the most well-developed and successful one. This kind of models has been confirmed to be capable of predicting a fairly large variety of hydraulic flow phenomena with the same empirical input and can be applied with some confidence (Rodi, 1980).

Three-dimensional prediction of flows in channels with spur dykes or spur-dyke-like structures has been carried out by some research groups. These researches are differed in the selection of turbulence models. Some calculated with LES, e.g. Zhou et al., 2000 and Kawaguchi et al., 2004. Some preferred RANS models, e.g. Ouillon and Dartus, 1997; Peng and Kawahara, 1998; Richardson and Panchang, 1998; Kimura et al., 2002 and Chrisohoides et al., 2003. The others used commercial software (generally the $k - \varepsilon$ model and its variants are

encapsulated) such as FLUENT, e.g. Ali and Karim, 2002 and Salaheldin et al., 2004. All these computations are documented to be able to capture the fundamental aspects of the 3D flow (in particular the local structure induced vortex systems) with a reasonable accuracy. And the limitations of the adopted models were also argued, together with the experimental conditions used in the verifications. Moreover, all these literatures listed here adopted structured mesh methods, which will be discussed later. These efforts may provide the successors with two kinds of information. Firstly, the understanding of the flow field around the local structures will be furthered. And secondly, more insight will be gained into the performance of different turbulence models in different flow conditions.

(2) Mesh system

Besides the turbulence model, the mesh system is also one of the most important subjects in a numerical simulation. An elaborate model may give quite bad result on a poorly organized mesh. The two most commonly adopted mesh systems in CFD community are generally referred to as structured mesh and unstructured mesh. The signature feature of structured mesh is the implicit grid structure, which may alleviate the need to store the mesh connectivity and permit the construction of rapid solution algorithms (Mavriplis, 1996). In order to take full advantages of structured mesh, many researchers have been devoting themselves to improving existing algorithms and developing new schemes, e.g. Jongen, 1997; Kimura and Hosoda, 2003; Li and Fleming, 2003; Johnston and Liu, 2004 and Brüger et al., 2005. However, despite its widespread use, the structured mesh methods are generally limited to solution domains of a relatively simple shape. It is very difficult and sometimes impossible to generate structured mesh for complex geometries, especially in 3D space. Moreover, structured mesh does not permit mesh adaptation and is very difficult to treat moving boundaries. On the other hand, flows in complex geometries with changeable boundaries are usually encountered and of great interest in the engineering practices nowadays. For example, sediment-laden flows pass the natural rivers with extremely irregular cross sections or artificial channels with small but important hydraulic structures. An accurate resolution of the domain geometries and movable boundaries necessitates the employment of unstructured mesh methods. The flow in channels with spur dykes is a typical problem that should be solved with an unstructured mesh.

Although the past several decades have witnessed the significant development in unstructured mesh methods, most of the published literatures dealt with flow phenomena in other fields such as aerodynamics, e.g. Mavriplis, 1997; Nakahashi et al., 1999; Hassan et al., 2000; Nakahashi et al., 2003 and Basara, 2004. Hydraulic engineering poses many special problems requiring solution methods probably not included in other fields. But the related research is quite few to the author's knowledge and is generally confined to 2D flows. For

instance, Kim et al. (1997) reported a RANS solver based on a 2D unstructured mesh, and the solver was verified to be capable of predicting some wall-bounded turbulent flows with a good accuracy. Nallapati and Perot (2000) developed a staggered unstructured mesh method for the 2D calculation of free surface flows and it was applied successfully to some benchmark problems. Recently, Olsen (2003) constructed a 3D model to calculate the formation of a meandering channel. The turbulent flow was predicted with the $k-\varepsilon$ model based on an unstructured mesh. Unfortunately, very few information has been shown on the quantitative comparison between the computational result and the physical model measurement in the paper. In the conclusion, Olsen indicated the necessity for further improvements and verifications. Haque et al. (2005) also carried out a 3D numerical simulation with a RANS solver provided by the commercial software FLUENT. The hybrid mesh adopted in the study was argued to be able to resolve the detailed hydraulic structures with a relatively small total number of cells. They compared the temperature distribution within a dam and its forebay situated on the Colombia River and found a reasonable similarity between the calculation and measurement. But the velocity field was not validated due to the shortage of field data.

Till now, the author does not find any successful report from other groups on the 3D modeling of turbulent flows induced by spur dykes or spur-dyke-like structures with unstructured mesh methods. It is then convinced that the current numerical model is of significant value. In this section, a 3D turbulence model based on an unstructured mesh will be presented. This model solves the RANS with a standard $k-\varepsilon$ model and some non-linear $k-\varepsilon$ models (optional) for the turbulence closure. The wall function approach is employed to resolve the near-wall boundary area. With this model, arbitrary polyhedral mesh up to six faces may be used for the calculation to resolve the complex geometries. In order to simulate the sophisticated interaction between the flow and bed evolution, the model is designed to be applicable for not only fixed meshes but also movable meshes from the beginning. At the current stage, the mesh is permitted to move in the vertical direction so as to capture the moving boundaries such as the riverbed.

This section is organized as follows. In the succeeding Sub-section 3.3.2, a set of governing PDEs (Partial differential equations) are firstly introduced. After that comes the detailed information on the discretization methods for the governing equations in Sub-section 3.3.3, which include: (a) FVM formulation; (b) Unstructured mesh; (c) Control volume decomposition; (d) Spatial discretization; (e) Mesh skewness; (f) Temporal integral; (g) Pressure-velocity coupling and (h) Mesh movement. The techniques distinguished from those of structured mesh methods are accentuated. In Sub-section 3.3.4 and Sub-section 3.3.5, the boundary conditions and the solution algorithms for sparse equation systems are presented one after another. In Sub-section 3.3.6, some numerical examples are tested and computational

results are compared with existing laboratorial measurements.

3.3.2 Governing equations

Turbulence models based on the RANS transform the Navier-Stokes equations in such a way that the transport equations are used for the time-averaged flow quantities only, while the time-averaged statistical turbulent fluctuations are modeled from the information of the mean flow. The representative models of this kind come from the $k - \varepsilon$ model family based on the eddy viscosity hypothesis.

(1) Mean flow field

The unsteady 3D RANS equations and the continuity equation expressed in a Cartesian coordinate system with the Einstein summation convention are as follows.

Momentum equations:

$$\frac{\partial u_i}{\partial t} + u_j \frac{\partial u_i}{\partial x_j} = f_i - \frac{1}{\rho} \frac{\partial p}{\partial x_i} + \nu \frac{\partial^2 u_i}{\partial x_j \partial x_j} + \frac{1}{\rho} \frac{\partial \tau_{ij}}{\partial x_j} \quad (3.5)$$

Continuity equation:

$$\frac{\partial u_j}{\partial x_j} = 0 \quad (3.6)$$

where u_i = time-averaged velocity; x_i = Cartesian coordinate component; ρ = density of the fluid; f_i = body force; p = time-averaged pressure; ν = molecular kinematic viscosity of the fluid; $\tau_{ij} = -\rho \overline{u'_i u'_j}$, are the Reynolds stress tensors, and u'_i is the fluctuating velocity component. As is readily seen, the above equations are not closed because of the unknown Reynolds stress tensors.

(2) Turbulence closure

In the standard $k - \varepsilon$ model, the Reynolds tensors are acquired through the linear constitutive equation.

$$-\overline{u'_i u'_j} = 2\nu_t S_{ij} - \frac{2}{3} k \delta_{ij} \quad (3.7)$$

where k = turbulence kinetic energy; δ_{ij} = the Kronecker delta; ν_t = eddy viscosity and S_{ij} = the

strain-rate tensor, the latter three are expressed by

$$\delta_{ij} = \begin{cases} 1 & \text{if } i = j \\ 0 & \text{if } i \neq j \end{cases} \quad (3.8)$$

$$\nu_t = C_\mu \frac{k}{\varepsilon} \quad (3.9)$$

$$S_{ij} = \frac{1}{2} \left(\frac{\partial u_i}{\partial x_j} + \frac{\partial u_j}{\partial x_i} \right) \quad (3.10)$$

in which C_μ is a coefficient, and is usually set to be a constant and equal to 0.09, ε is the dissipation rate of the turbulence kinetic energy k . Two transport equations as described below are employed to estimate k and ε , respectively.

$$\frac{\partial k}{\partial t} + u_j \frac{\partial k}{\partial x_j} = \frac{\partial}{\partial x_j} \left[\left(\nu + \frac{\nu_t}{\sigma_k} \right) \frac{\partial k}{\partial x_j} \right] \quad (3.11)$$

$$\frac{\partial \varepsilon}{\partial t} + u_j \frac{\partial \varepsilon}{\partial x_j} = \frac{\partial}{\partial x_j} \left[\left(\nu + \frac{\nu_t}{\sigma_\varepsilon} \right) \frac{\partial \varepsilon}{\partial x_j} \right] + (C_{1\varepsilon} G - C_{2\varepsilon} \varepsilon) \frac{\varepsilon}{k} \quad (3.12)$$

where G = the rate-of-production of the turbulence kinetic energy k , is defined as

$$G = -\overline{u_i u_j} \frac{\partial u_i}{\partial x_j} \quad (3.13)$$

The model constants suggested by Rodi (1980) generally take the universal values as below.

$$\sigma_k = 1.0 \quad \sigma_\varepsilon = 1.3 \quad C_{1\varepsilon} = 1.44 \quad C_{2\varepsilon} = 1.92 \quad (3.14)$$

The standard $k - \varepsilon$ model has achieved a widespread use and is of continuous interest in the engineering practice. It appears to be the simplest model amongst those which are suitable for flows with complex geometries (Rodi, 1980). There are a lot of experiences on its behavior, and it may be used with confidence in a great number of engineering flows.

However, as pointed out by Speziale (1991), the standard $k - \varepsilon$ model suffers from the following major deficiencies: (1) the inability to properly account for the streamline curvature, rotational strains and other body-force effects; and (2) the neglect of the non-local and historical effects of the Reynolds stress anisotropies. These may be partially cured by introducing a non-linear constitutive relation between the turbulence stresses and the mean strain rate satisfying certain tensorial properties. Efforts and achievements have been made by many

research groups such as Rubinstein and Barton, 1990; Gatski and Speziale, 1993; Shih et al, 1993 and Kimura and Hosoda, 2003. A general form for a quadratic constitutive equation can be summarized as

$$\begin{aligned}\overline{u'_i u'_j} = & \frac{2}{3} k \delta_{ij} - 2\nu_t S_{ij} \\ & + a_1 \frac{k^3}{\varepsilon^2} \left(\frac{\partial u_i}{\partial x_l} \frac{\partial u_l}{\partial x_j} + \frac{\partial u_j}{\partial x_l} \frac{\partial u_l}{\partial x_i} - \frac{2}{3} \frac{\partial u_k}{\partial x_m} \frac{\partial u_m}{\partial x_k} \delta_{ij} \right) \\ & + a_2 \frac{k^3}{\varepsilon^2} \left(\frac{\partial u_i}{\partial x_l} \frac{\partial u_j}{\partial x_l} - \frac{1}{3} \frac{\partial u_m}{\partial x_k} \frac{\partial u_m}{\partial x_k} \delta_{ij} \right) \\ & + a_3 \frac{k^3}{\varepsilon^2} \left(\frac{\partial u_l}{\partial x_i} \frac{\partial u_l}{\partial x_j} - \frac{1}{3} \frac{\partial u_m}{\partial x_k} \frac{\partial u_m}{\partial x_k} \delta_{ij} \right)\end{aligned}\quad (3.15)$$

where a_1 , a_2 and a_3 are coefficients, which may have different values due to different models. For example, the model proposed by Kimura and Hosoda (2003) adopts

$$C_\mu = \min \left(0.09, \frac{0.3}{1 + 0.09 M_{s\Omega}^2} \right) \quad (3.16)$$

$$a_1 = 0, \quad a_2 = \frac{0.4 C_\mu}{1 + 0.01 M_{s\Omega}^2}, \quad a_3 = \frac{-0.13 C_\mu}{1 + 0.01 M_{s\Omega}^2} \quad (3.17)$$

in which

$$M_{s\Omega} = \max(S_{k\varepsilon}, \Omega) \quad (3.18)$$

and

$$S_{k\varepsilon} = \frac{k}{\varepsilon} \sqrt{2 S_{ij} S_{ij}} \quad (3.19)$$

$$\Omega = \frac{k}{\varepsilon} \sqrt{2 \Omega_{ij} \Omega_{ij}} \quad (3.20)$$

where $S_{k\varepsilon}$ = the strain parameter and Ω = the rotation parameter.

This model has been tuned by experimental studies and consideration of the constraints of realizability such as non-negativity of the normal Reynolds stresses and Schwarz' inequality between turbulent velocity correlations. The applicability of this model has also been confirmed

by applying to some flows in laboratory flumes based on structured meshes, e.g. Kimura et al., 2002 and Kimura and Hosoda, 2003. The coefficients for some other models used in this study are summarized in Table 3.1.

Table 3.1 Coefficients for Some Non-linear Models Used in this Study

	C_μ	a_1	a_2	a_3
Yoshizawa, 1984	0.09	-0.167	0.057	-0.0067
Speziale, 1987	0.09	0.014	0.041	-0.014
Rubinstein and Barton, 1990	0.0845	0.104	0.034	-0.014
Shih, Zhu and Lumely, 1993	$\frac{0.67}{1.25 + S_{k\varepsilon}}$	$\frac{-4}{1000 + S_{k\varepsilon}^3}$	$\frac{13}{1000 + S_{k\varepsilon}^3}$	$\frac{-2}{1000 + S_{k\varepsilon}^3}$
Gatski and Speziale, 1993	$0.68C_{S\Omega}$	$0.03C_{S\Omega}$	$0.093C_{S\Omega}$	$-0.034C_{S\Omega}$
Remarks	$C_{S\Omega} = \frac{1 + 0.0038S_{k\varepsilon}^2}{3 + 0.0038S_{k\varepsilon}^2 + 0.0008S_{k\varepsilon}^2\Omega^2 + 0.2\Omega^2}$			

The non-linear $k-\varepsilon$ models can reflect more physics and are more elaborate from the viewpoint of mathematics. However, the non-linear terms may introduce some numerical problems during implementation especially in the unstructured mesh system, which have been observed in the numerical experiments.

In this study, the standard $k-\varepsilon$ model is still preferred. But the non-linear $k-\varepsilon$ models are also implemented in some simple cases in order to understand the performance of the standard $k-\varepsilon$ model and acquire more detailed information on the physics of the flow.

Substituting the Reynolds stress expression (i.e. Eq. 3.15) into the momentum equations (i.e. Eq. 3.5), the following equation is obtained.

$$\frac{\partial u_i}{\partial t} + u_j \frac{\partial u_i}{\partial x_j} = f_i - \frac{\partial}{\partial x_i} \left(\frac{p}{\rho} + \frac{2}{3}k \right) + (v + v_t) \frac{\partial^2 u_i}{\partial x_j \partial x_j} + v_t \frac{\partial}{\partial x_j} \left(\frac{\partial u_j}{\partial x_i} \right) + \frac{\partial N_{ij}}{\partial x_j} \quad (3.21)$$

where N_{ij} = the non-linear terms introduced in the Reynolds stress expression, which is a polynomial of the derivatives of the velocity components. When the standard $k-\varepsilon$ model is used, this term becomes zero.

In the above equation, the terms in the Reynolds stresses have been divided into three parts, the diffusive-like term $v_t \partial^2 u_i / \partial x_j^2$ is treated implicitly as a diffusive term, the term $2/3k\delta_{ij}$ is incorporated into the pressure term and the remainder is transferred to the source terms. If the

pressure gradient is also considered as a source term, it is seen that the final momentum equations (i.e. Eq. 3.21) have the same form as that of the transport equations of k and ε (i.e. Eq. 3.11 and Eq. 3.12). This indicates that the same numerical methods may be used for the governing PDEs.

3.3.3 Discretization methods

(1) FVM formulation

As is known, the FVM is the most widely used numerical methods in CFD for engineering applications. In an FVM procedure, the study domain is divided into a number of continuous polyhedral CVs (Control volumes). If one integrates the governing PDEs over a CV, the final governing equations with the FVM formulation have the general form as follows.

$$\frac{\partial}{\partial t} \int_V dV + \int_S \mathbf{u} \cdot \mathbf{n} dS = 0 \quad (3.22)$$

$$\frac{\partial}{\partial t} \int_V \phi dV + \int_S \phi \mathbf{u} \cdot \mathbf{n} dS = \int_S \Gamma \nabla \phi \cdot \mathbf{n} dS + \int_V b dV \quad (3.23)$$

where V = the volume of the CV; S = the CV face with a unit normal vector $\mathbf{n}$ directing outwards; ϕ = general conserved quantity representing either scalars or vector and tensor field components; $\mathbf{u}$ = the fluid velocity vector whose Cartesian components are u_i or (u, v, w) ; Γ = diffusion coefficient and b = the volumetric source of the quantity ϕ . Eq. 3.22 expresses the mass conservation and Eq. 3.23 stands for the conservation of other quantities such as the momentum. It is noted that on the left side of Eq. 3.23 there are transient terms and convective terms which are balanced by diffusive terms and source terms on the right. The equation system is mesh-independent and is valid for arbitrary polyhedral CVs. As a result, an FVM procedure is able to take full advantages of an unstructured mesh system. Moreover, the summation of the equations for all CVs leads to the global conservation equation because the inner CV faces will cancel out. This is also one of the most attractive points to use FVM in the engineering practice as the conservativeness is usually a great consideration.

(2) Unstructured mesh

The application of unstructured mesh is undergoing considerable expansion recently. It is mostly attributed to some of its inherent advantages. As is well-known, unstructured mesh offers significant flexibility in treating complicated geometries and provides a convenient means of pursuing mesh adaptation. Both of which are important considerations for the engineering practice to date.

In an unstructured mesh method, the mesh connectivity is not implicitly known. Since the CVs may be numbered in any order and have any number of neighbors, an extra space in the memory should be maintained for that kind of information. This may sometimes result in a considerable consumption of computer resources.

As is known, any kind of polyhedral mesh may be used in the numerical simulation theoretically, but the generally adopted unstructured mesh in engineering practices is confined to tetrahedra, pyramids, prisms and hexahedra (see Fig. 3.1). A partial explanation lies in the difficulty of generating polyhedral mesh with more than six faces. Furthermore, there is no evidence that the simulation accuracy can be obviously improved by increasing the complexity of the mesh system. Amongst all the above mesh types, the hexahedron may be accounted as the general case, and the quadrilateral is the general case for the CV face correspondingly. It indicates that a hybrid mesh system in practice is possible to be considered as a hexahedral mesh provided that the polyhedra with less than six faces are assigned some nominal ones to satisfy the convention. This provides a way to simplify the data structure in storage.

In order to define the connectivity of the CVs, there are a lot of alternatives. The most general data structure contains all the information of the CV explicitly. That is: a CV is defined by its six faces, the faces by the lists of edges and the edges by their corresponding vertices. However this method is not preferred in order to reduce the number of arrays needed for the definition of mesh connectivity. A simplified data structure is introduced here. This method is also employed in some CFD codes and recommended by Ferziger and Perić (2002).

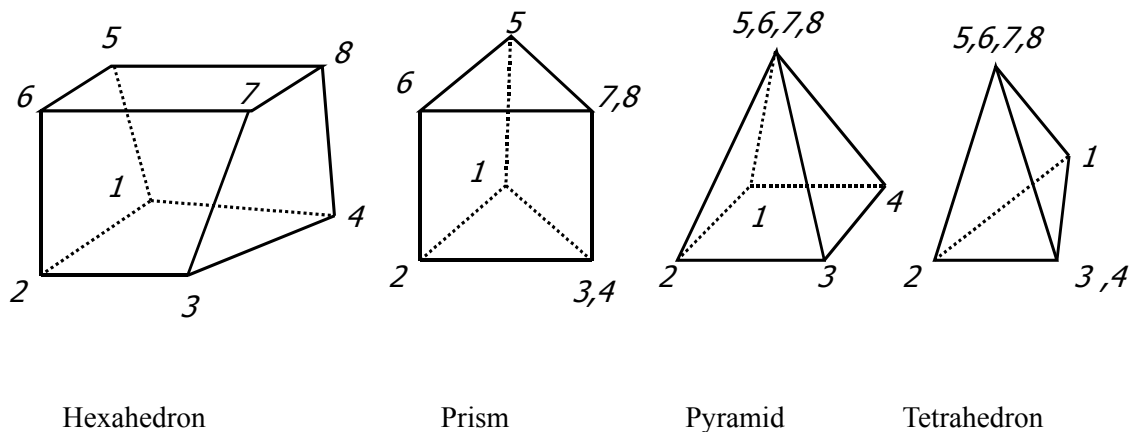


Fig. 3.1 CV and Its Vertices

As indicated in Fig. 3.1, a CV may be defined only by a list of its eight vertices in a counter-clockwise order. With such definition, the faces enclosing the CV and the edges forming

the faces are also uniquely identified. They may be implicitly organized in an ordered way without occupying any computer memory. Moreover, since the arbitrary polyhedral mesh is stored as a hexahedral mesh, the number of the neighboring CVs is fixed to be six. Hence the neighboring CVs which share the common faces with the current CV may also be indexed in the same ordered way as that for the faces. For nominal hexahedra, due to the existence of nominal faces, some of the vertices will be repeated during storing as also shown in Fig. 3.1. Using this kind of treatment, the input of a mesh system includes only a list of nodal coordinates, a list of vertices of the CVs and a list of the neighboring CVs. A typical CV and its six neighbors are depicted in Fig. 3.2.

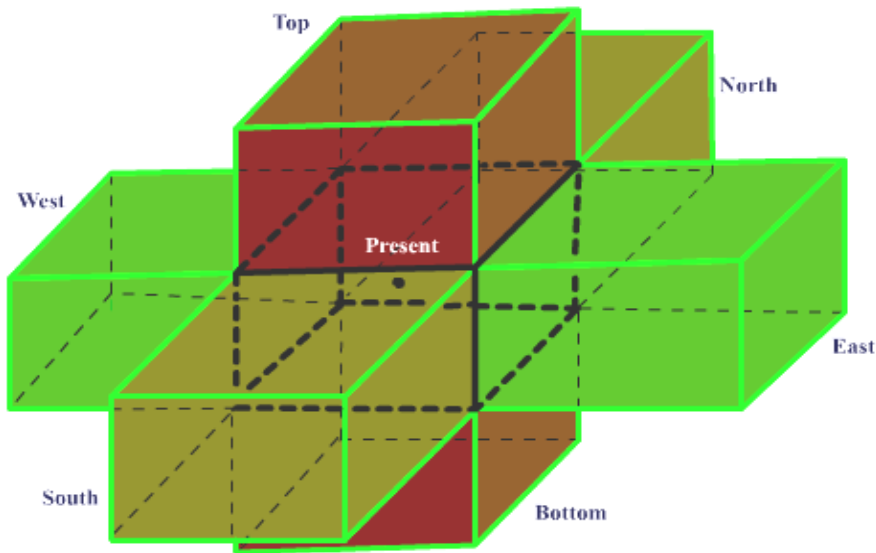


Fig. 3.2 A Typical CV (Center) and Its Six Neighbors

The FVM also provides options for the variable arrangement on the mesh system, which is generally categorized into two kinds: staggered mesh and collocated mesh (see Fig. 3.3 for the typical arrangements of the velocity field and pressure in 2D space).

A staggered arrangement of variables on a structured mesh has achieved great success, since this kind of arrangement can lead to a strong coupling between the velocities and pressure gradient. The coupling of velocity and pressure has been a troublesome problem for a long time until the invention of the Rhie-Chow interpolation methods in 1983. However, a staggered arrangement becomes very difficult when the CVs of arbitrary shape are allowed, which is generally encountered in an unstructured mesh. Furthermore, different variables in a staggered arrangement are calculated on different mesh systems. This does not bring much trouble in a

structured mesh because all the data is ordered using a lexicographic ordering. Nevertheless, a significant increment of computer storage is unavoidable in an unstructured mesh because each mesh system has to be explicitly defined. In view of these arguments, a collocated arrangement is adopted in this study. With this arrangement, all variables share the same CV and are defined at the center of the CV. As a result, only one mesh system is needed in the calculation, which will lead to a tremendous saving of computer resources.

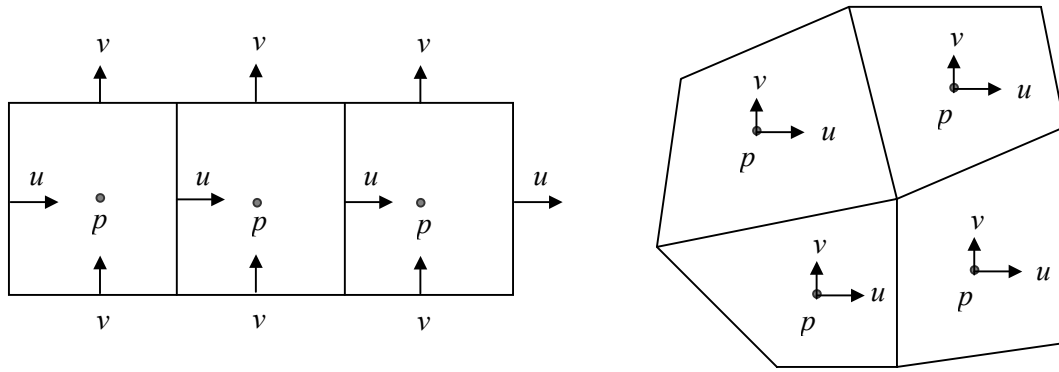


Fig. 3.3 Staggered Mesh (Left) and Collocated Mesh (Right) in 2D Space

In order to solve the variables at the center of the CVs, the governing equations should be discretized into a series of algebraic equations. It is very clear from Eq. 3.22 and Eq. 3.23 that there are different classes of problems during the fully discretization process. These are: (a) CV decomposition; (b) Integrals over time, volume and surface; (c) Interpolation from the center to the surface of the CVs; and (d) Gradients of conserved quantities. As all these problems may introduce errors to the simulation, approximation methods should be carefully selected and they are discussed below.

(3) CV decomposition

With the aforementioned data structure of the mesh system, it is no need to evaluate the geometrical elements for arbitrary polyhedra. The calculation is limited to quadrilaterals in 2D space and hexahedra in 3D space. This makes it possible to design more effective algorithms and sub-routines for the CV decomposition.

As the area and the center of a triangle is very easy to acquire. One can subdivide a quadrilateral face into two triangles. The vector area of the quadrilateral is approximated by the summation of those of the two sub-triangles. For instance, the vector area of the quadrilateral (1,2,3,4) in Fig. 3.4 is computed from

$$\mathbf{r}_i = x_i \mathbf{i} + y_i \mathbf{j} + z_i \mathbf{k} \quad (3.24)$$

$$\mathbf{S}_{1234} = \mathbf{S}_{123} + \mathbf{S}_{134} = \frac{1}{2} [(\mathbf{r}_2 - \mathbf{r}_1) \times (\mathbf{r}_3 - \mathbf{r}_1) + (\mathbf{r}_3 - \mathbf{r}_1) \times (\mathbf{r}_4 - \mathbf{r}_1)] \quad (3.25)$$

where $\mathbf{r}_i$ = the i th vertex of the CV; $\mathbf{i}, \mathbf{j}, \mathbf{k}$ = unit vectors in the x, y, z coordinate directions; $\mathbf{S}_{123}$, $\mathbf{S}_{134}$ = vector areas of the sub-triangles Δ_{123} and Δ_{134} , respectively and $\mathbf{S}_{1234}$ = vector area of the quadrilateral surface (1,2,3,4).

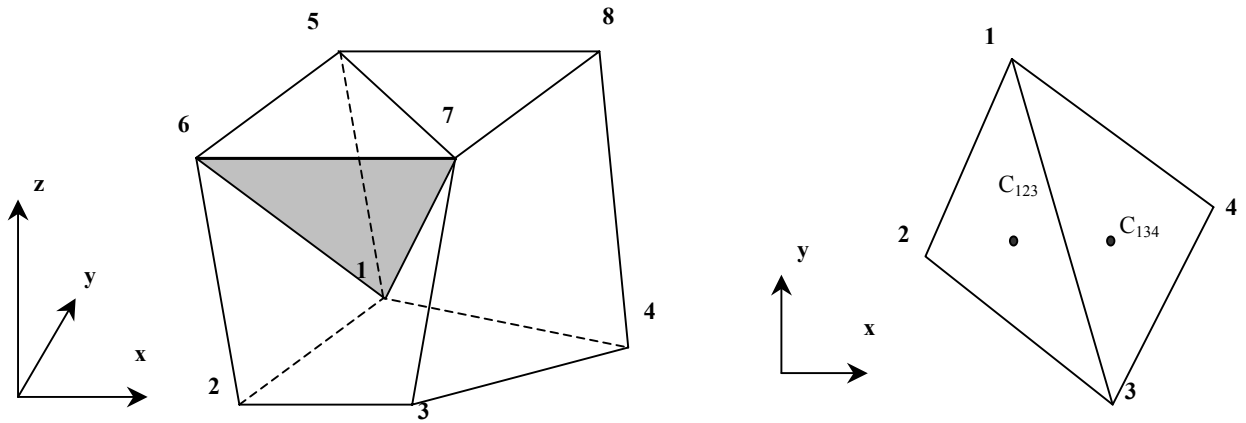


Fig. 3.4 CV and Its Vertices

The center of the surface is the mean value of those of the sub-triangles weighted by the corresponding area, i.e.

$$\mathbf{c}_{1234} = \frac{S_{123}\mathbf{c}_{123} + S_{134}\mathbf{c}_{134}}{S_{1234}} \quad (3.26)$$

where $\mathbf{c}$ = the center of a surface and S = the area of a surface.

Extending the above logic from 2D to 3D, a hexahedron is divided into 6 tetrahedra. The evaluation of the volume is thus acquired by summation of the volumes of all the sub-tetrahedra corresponding to their partitioning. And the volume of a tetrahedron is a dot product of two vectors, for example the volume of tetrahedron (1,5,6,7) is

$$V_{1567} = \frac{1}{3}(\mathbf{r}_5 - \mathbf{r}_1) \cdot \mathbf{S}_{567} \quad (3.27)$$

where V_{1567} = the volume of tetrahedron (1,5,6,7). The centroid of the CV is the mean value of those of the sub-tetrahedra weighted by the corresponding volume.

$$\mathbf{c}_V = \frac{\sum V_i \mathbf{c}_i}{V} \quad (3.28)$$

where $\mathbf{c}_V$ = the centroid of the CV; V_i = the volume of sub-tetrahedron i (i ranges from 1 to 6) and $\mathbf{c}_i$ = the centroid of sub-tetrahedron i .

According the above analysis, the sub-routine for the calculation of the geometry elements can be easily coded.

(3) Spatial discretization

Using a second-order mid-point rule, the semi-discretized formulation is easily obtained in space term by term, this yields.

$$\frac{\partial}{\partial t} \int_V \phi dV + \sum_f \phi_f (\mathbf{u}_{f\perp} \mathbf{S}_f) = \sum_f \Gamma_f \left. \frac{\partial \phi}{\partial \mathbf{n}} \right|_f \mathbf{S}_f + b_P - s_P \phi_P \quad (3.29)$$

where subscript P = the present CV; subscript f = the face of the CV; subscript $\perp$ = the component of the quantity normal to the surface; $\mathbf{S}_f$ = the area of the CV surface; b_P = the part of the source term containing all the contributions excluding unknown variables and $-s_P \phi_P$ = the part of the source term including the unknown variables which can be treated implicitly.

It is readily seen that Eq. 3.29 is not an explicit expression of the variables defined at the centers of the CVs. Values at other locations have to be obtained by some kind of interpolation methods. The diffusive term contains the gradient of a quantity, which necessitates some numerical differentiation techniques.

An arithmetic interpolation method is commonly used to evaluate the surface values. For a quantity ϕ on the surface, the value is estimated from the calculated values at the center of the CV on either side, i.e.

$$\phi_f = \alpha_f \phi_P + (1 - \alpha_f) \phi_A \quad (3.30)$$

$$\text{in which, } \alpha_f = \frac{d_{Af}}{d_{Af} + d_{fP}} \quad (3.31)$$

where the subscript A = the adjacent CV, d_{fP} and d_{Af} are the distances from the surface to the present CV and to the adjacent CV, respectively.

From the viewpoint of stability and ease of programming, use of higher order schemes are avoided in this study. The power law scheme is employed during the spatial discretization. This scheme is relatively easy and has been confirmed to be applicable in 3D calculations (Wilson et al., 2003; Olsen, 2004). With the power law scheme, Eq. 3.29 can be finally assembled as

$$\sum_f \left\{ \left[D_f A(|P_f|) + \max(-F_f, 0) \right] (\phi_P - \phi_A) + F_f \phi_P \right\} = b_P - s_P \phi_P \quad (3.32)$$

where the strength of the convection F_f , diffusion conductance D_f and the ratio of them are given as

$$F_f = u_{f\perp} S_f, \quad D_f = \frac{\Gamma_f S_f}{d_{AP}}, \quad P_f = \frac{F_f}{D_f} \quad (3.33)$$

and

$$A(|P_f|) = \max \left[0, \left(1 - 0.1 |P_f| \right)^5 \right] \quad (3.34)$$

The interpolation of the diffusive coefficient on the surface and the surface fluxes deserve special attention. For the diffusive coefficient, the harmonic mean may reflect more physics and reasonableness in particular near the boundary. This results in

$$\Gamma_f = \frac{\Gamma_A \Gamma_P}{\alpha_f \Gamma_P + (1 - \alpha_f) \Gamma_A} \quad (3.35)$$

A simple arithmetic mean for the surface fluxes may lead to checkerboard variable distribution, which has caused the slow acceptance of the use of collocated mesh. This problem can be cured by the interpolation method proposed by Rhie and Chow (1983). The method introduces an additional term related to the pressure gradient when calculating the fluxes on the surface.

As is known, the unknown quantities of the present CV can be finally expressed by all of its neighboring CVs after discretization. For instance, the momentum equations for u at present CV and one of its adjacent CVs are written as

$$a_P u_P = \sum_{nb} a_{nb} u_{nb} \Big|_P - \int_V \frac{\partial p}{\partial x} dV \Big|_P + b_P \quad (3.36)$$

$$a_A u_A = \sum_{nb} a_{nb} u_{nb} \Big|_A - \int_V \frac{\partial p}{\partial x} dV \Big|_A + b_A \quad (3.37)$$

where a = coefficient for the unknowns at the center of the approximated CV and nb = the

neighboring CV.

From the conservation principle of the FVM formulation, the velocity at the common face of the two neighboring CVs must also have a discretized momentum equation of the similar form as that of Eq.3.36 and Eq. 3.37, i.e.

$$a_f u_f = \sum_{nb} a_{nb} u_{nb} \Big|_f - \int_V \frac{\partial p}{\partial x} dV \Big|_f + b_f \quad (3.38)$$

Approximating the solution u_f of Eq.3.38, the information from Eq.3.36 and Eq. 3.37 can be used. By employing some linear interpolation and simplification, the following equation is obtained.

$$u_f = \bar{u}_f + \frac{1}{a_f} \left(\overline{\int_V \frac{\partial p}{\partial x} dV} \Big|_f - \int_V \frac{\partial p}{\partial x} dV \Big|_f \right) \quad (3.39)$$

where

$$\begin{aligned} \bar{u}_f &= \alpha_f u_P + (1 - \alpha_f) u_A \\ a_f &= \alpha_f a_P + (1 - \alpha_f) a_A \\ \overline{\int_V \frac{\partial p}{\partial x} dV} \Big|_f &= \alpha_f \int_V \frac{\partial p}{\partial x} dV \Big|_P + (1 - \alpha_f) \int_V \frac{\partial p}{\partial x} dV \Big|_A \\ \int_V \frac{\partial p}{\partial x} dV \Big|_f &= S_{fx} (p_A - p_P) \end{aligned} \quad (3.40)$$

in which S_{fx} = projected area of the surface to the yz plane (perpendicular to the x axis). With this interpolation method to evaluate the surface flux (i.e. Eq. 3.39), the checkerboard phenomenon of the variables may be effectively avoided. The extension to other velocity components is straightforward.

(4) Mesh skewness

In the above discretization process, such assumption has been made that the line connecting the neighboring CV centers is almost orthogonal to the cell face and passes through the cell-face center. Under this presumption, the numerical simulation can give almost the same accuracy as that achieved by the mathematical derivation. However it is not always the case. When unstructured mesh is used, the mesh skewness is almost unavoidable.

Mesh skewness is generally classified into two kinds: non-conjunctionality and non-orthogonality. The former means that the intersection is not the midway of the surface and the latter stands for a poor perpendicularity. Ferziger and Perić (2002) have suggested some ways to maintain the discretization accuracy in case of mesh skewness.

If a non-conjunctional mesh occurs, the value at the intersection (f' in Fig.3.5) is firstly evaluated by interpolation with the methods described before, and then a correction term is introduced. The value at the center of the surface (f in Fig.3.5) is evaluated from

$$\phi_f = \phi_{f'} + (\nabla \phi)_{f'} \cdot (\mathbf{r}_f - \mathbf{r}_{f'}) \quad (3.41)$$

where f' = the intersection of the surface and the line connecting the two neighboring CVs, and the gradient of ϕ at f' is obtained by interpolating the cell-center gradients at either side of the face.

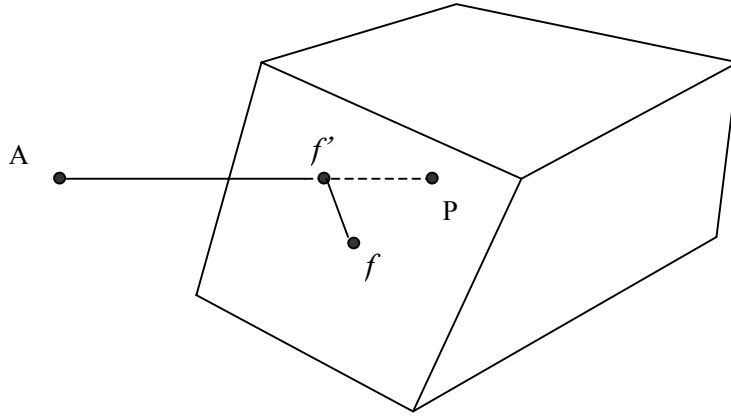


Fig. 3.5 Treatment of Mesh Non-conjunctionality

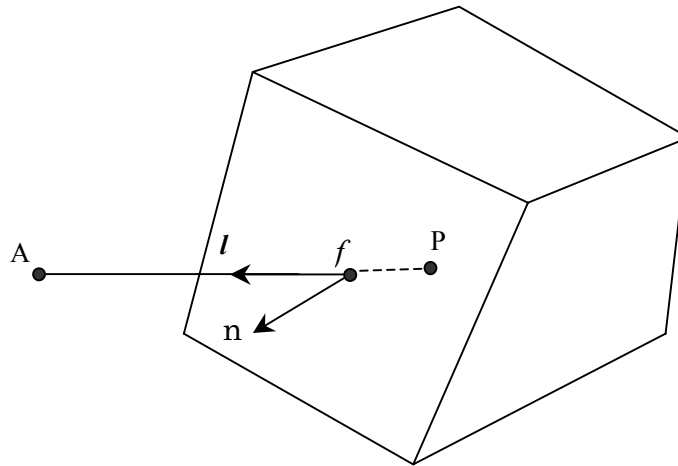


Fig. 3.6 Treatment of Mesh Non-orthogonality

For a non-orthogonal mesh as shown in Fig. 3.6, the normal gradient of the quantity ϕ at the surface in the diffusive term has to be corrected. Dividing the diffusive term into a normal diffusion and a cross diffusion, the following approximation is suggested.

$$\left. \frac{\partial \phi}{\partial n} \right|_f = \frac{\phi_A - \phi_P}{d_{AP}} + (\nabla \phi)_f^{\text{exp}} \cdot (\mathbf{n} - \mathbf{l}) \quad (3.42)$$

$$\text{where } \mathbf{l} = \frac{\mathbf{r}_A - \mathbf{r}_P}{|\mathbf{r}_A - \mathbf{r}_P|} \quad (3.43)$$

where $\mathbf{l}$ = the unit vector in the direction of the line connecting the center of the current mesh and its neighbor.

The first term on the right hand side is treated implicitly while the second term is a deferred correction which is calculated using the interpolated cell center gradients explicitly. (This is the reason why the superscript *exp* is used) It is obvious that in the final discretized algebraic equation, the second term on the right hand side becomes a source term. And if non-orthogonality is not very severe, the vectors $\mathbf{n}$ and $\mathbf{l}$ are co-linear, this source term will disappear.

(5) Temporal integral

At the end of the spatial discretization, one can get the following equation,

$$\frac{\partial}{\partial t} \int_V \phi dV = F \quad (3.44)$$

$$\text{where } F = \sum_{nb} a_{nb} \phi_{nb} + b_P - a_P \phi_P \quad (3.45)$$

Taking the second order implicit Crank-Nicolson scheme as follows

$$\frac{\phi^{m+1} - \phi^m}{\Delta t} V = \frac{F^{m+1} + F^m}{2} \quad (3.46)$$

where the superscript m and $m+1$ stand for the previous and the current time step. One can arrive at the final algebraic equation set.

$$a_P \phi_P^{m+1} = \sum_{nb} a_{nb} \phi_{nb}^{m+1} + b_P \quad (3.47)$$

in which

$$\begin{aligned}
a_p^{m+1} &= \sum_{nb} a_{nb}^{m+1} + \sum_f F_f + s_p^{m+1} \\
a_p &= a_p^{m+1} + \frac{2V}{\Delta t} \\
b_p &= b_p^{m+1} + \frac{2V}{\Delta t} \phi_p^m + \left(\sum_{nb} a_{nb} \phi_{nb} + b_p - a_p \phi_p \right)^m
\end{aligned} \tag{3.48}$$

It is seen from Eq. 3.47 and Eq. 3.48 that after the time integral, a time-related term is introduced to the coefficient of a_p comparing with the steady case. The source term b_p also has some changes. Besides a temporal term, there is another contribution from the previous time step.

(6) Pressure-velocity coupling

The continuity equation does not include the pressure information explicitly, but provides constraints for the velocity field. In order to couple the pressure and velocity, the solution procedure follows the widely used SIMPLE (Semi-implicit method for pressure-linked equations) algorithm. The main concept of this method is to guess a pressure distribution firstly and then get a pressure correction with the continuity equation. The procedure for an unstructured mesh is summarized here.

In the derivation, such notations are used. The guessed pressure distribution and the velocity field not satisfying the continuity equation are denoted with an index *, the corrections of the variables are denoted with an index ', and the variables without any superscript stand for the corrected values. Hence,

$$\begin{aligned}
u_i &= u_i^* + u_i' \\
p &= p^* + p'
\end{aligned} \tag{3.49}$$

Extracting the pressure term from the source term in the discretized momentum equation, the velocity component in the x direction u^* satisfies

$$a_p u_p^* = \sum_{nb} a_{nb} u_{nb}^* - \sum_f S_{fx} p_f^* + b_p \tag{3.50}$$

On the other hand, the same discretized momentum equation based on the corrected velocity component u has the following form

$$a_p u_p = \sum_{nb} a_{nb} u_{nb} - \sum_f S_{fx} p_f + b_p \tag{3.51}$$

If Eq. 3.50 is subtracted from Eq. 3.51, and Eq. 3.49 is taken into account, the velocity

correction for u is obtained.

$$u'_p = \frac{\sum a_{nb} u'_{nb}}{a_p} - \frac{\sum S_{fx} p'_f}{a_p} \quad (3.52)$$

In a SIMPLE method, the first term on the right hand side is omitted. Then the velocity correction in the CV center has a simple relation with the pressure correction. That is

$$u'_p = - \frac{\sum S_{fx} p'_f}{a_p} \quad (3.53)$$

For the velocity correction on the surface, the same relationship is assumed to be valid instead of interpolating the value from the CV centers. It means

$$u'_f = - \frac{1}{a_p} S_{fx} (p'_A - p'_p) \quad (3.54)$$

The correction of the other velocity components may be derived in the same way. The corrected velocity field is then introduced to ensure the continuity equation. An equation set for the pressure correction will be obtained.

$$\sum_f \left[(u_f^* S_{fx} + v_f^* S_{fy} + w_f^* S_{fz}) + \left(\frac{S_{fx}^2}{a_{px}} + \frac{S_{fy}^2}{a_{py}} + \frac{S_{fz}^2}{a_{pz}} \right) (p'_p - p'_A) \right] = 0 \quad (3.55)$$

where a_{px} , a_{py} and a_{pz} = the coefficients for the calculation of variables u , v , and w , respectively. Eq. 3.54 can be further written as

$$a_p^p p'_p = \sum_{nb} a_{nb}^p p'_{nb} + b_p^p \quad (3.56)$$

in which

$$\begin{aligned} a_{nb}^p &= \frac{S_{fx}^2}{a_{px}} + \frac{S_{fy}^2}{a_{py}} + \frac{S_{fz}^2}{a_{pz}} \\ a_p^p &= \sum_{nb} a_{nb}^p \\ b_p^p &= - \sum_f (u_f^* S_{fx} + v_f^* S_{fy} + w_f^* S_{fz}) \end{aligned} \quad (3.57)$$

This equation set is closed and has almost the same form as that of the governing equations. And the same discretization methods may be used. The solution of the equation set yields the pressure correction and hence the velocity correction. The velocity field is then renewed with Eq. 3.49 and a new iteration starts until the convergence is achieved.

(7) Mesh movement

In order to simulate the flow in channels with spur dykes, one should consider not only the complex geometries of the flow domain but also the interaction between the flow and the bed evolution. The latter leads to the movement of the computational domain. Consequently, a moving mesh should be introduced during the simulation.

The final governing equations considering the mesh movement with the FVM formulation are generally written as (Ferziger and Perić, 2002)

$$\frac{\partial}{\partial t} \int_V dV + \int_S (\mathbf{u} - \mathbf{u}_m) \cdot \mathbf{n} dS = 0 \quad (3.58)$$

$$\frac{\partial}{\partial t} \int_V \phi dV + \int_S \phi (\mathbf{u} - \mathbf{u}_m) \cdot \mathbf{n} dS = \int_S \Gamma \nabla \phi \cdot \mathbf{n} dS + \int_V b dV \quad (3.59)$$

where $\mathbf{u}_m$ = the velocity vector of CV face movement whose Cartesian components are u_{mi} or (u_m, v_m, w_m) . Compared these two equations with Eq. 3.22 and Eq. 3.23, only a velocity for the mesh movement has been added to the convective terms. It means that the governing equations for movable CVs may be derived from those of fixed CVs by just replacing the velocity vector $\mathbf{u}$ in the convective terms with the relative velocity vector $\mathbf{u} - \mathbf{u}_m$. The problem becomes how to evaluate the velocity of mesh movement.

Since the mesh movement has been included in the governing equations, the solution of the equation system is related to the CV itself rather than the location of the CV center in the Cartesian coordinate. As a result, it is no need to redistribute the solution from the current time step to the next time step even if a new mesh system is generated due to the change of domain geometries. On the other hand, the mesh movement may introduce artificial mass sources in the discretized equations, which have a potential to accumulate and spoil the simulation. Therefore, the conservation of the space should also be satisfied when the CV changes its shape and/or position with time. That is, the summation of the volume fluxes through CV faces due to the mesh movement must equal the rate of the volume change. A mathematic interpretation is

$$\frac{\partial}{\partial t} \int_V dV - \int_S \mathbf{u}_m \cdot \mathbf{n} dS = 0 \quad (3.60)$$

If Eq. 3.60 is substituted into Eq. 3.58, one may find that Eq. 3.58 has a much simpler form as follows

$$\int_S \mathbf{u} \cdot \mathbf{n} dS = 0 \quad (3.61)$$

From the viewpoint of computational conveniences, the mesh system is allowed to move only in the vertical direction for the time being. This makes it possible to use some simple approach

to evaluate the volume fluxes and that the space conservation law Eq. 3.60 is automatically guaranteed at the same time. In this study, the velocity of the movement of a CV face is determined from the difference in the mesh locations at two consecutive time steps m and $m+1$ (see Fig. 3.7), i.e.

$$\mathbf{u}_m = \frac{\mathbf{r}_c^{m+1} - \mathbf{r}_c^m}{\Delta t} \quad (3.62)$$

where r_c = center of the CV face, Δt = time.

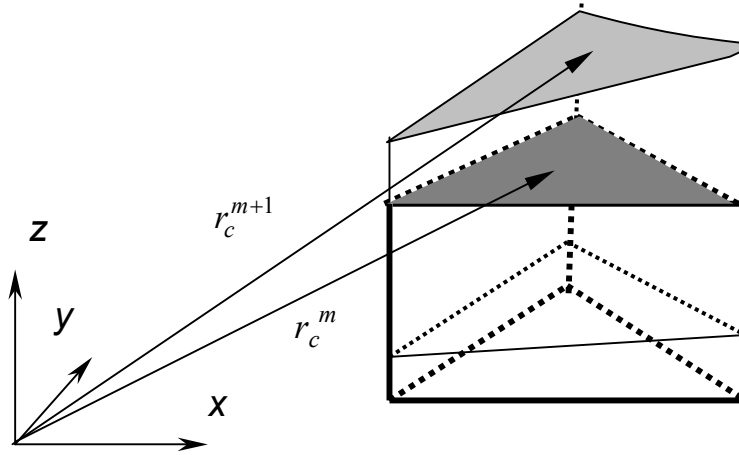


Fig. 3.7 Movement of a CV face at two consecutive time steps

3.3.4 Boundary conditions

The implementation of boundary conditions requires special attention. Since the boundaries do not provide additional equations, introducing of additional unknowns is not desirable. The commonly encountered boundary conditions in engineering practice include the inlet, the outlet, the impermeable wall, the free surface and the symmetrical plane. For the time being, the free surface is considered as a symmetrical plane. This presumption can greatly simplify the solution process and is acceptable for many hydraulic problems.

(1) Inlet

The inlet boundary is considered as a Dirichlet boundary, and all the quantities have to be prescribed. In the discretized equation for the CV near the inlet boundary, as the boundary value

is given directly, the contribution from the boundary turns into a source term and is no need to be calculated implicitly. In the pressure-correction equation, as the velocity field is given, the velocity correction is zero. And the Neumann boundary of zero gradients is suitable for the pressure.

(2) Outlet

At the outlet boundary, the flow information is usually very few. In order to avoid the propagation of errors, an alternative is to place the outlet boundary as far downstream of the study domain as possible. Then a Neumann boundary with zero gradients can be assumed. In order to ensure the global conservation of mass, the following technique is employed. Firstly, an initial estimation of the velocity at the outlet is acquired by extrapolation from the near boundary CVs. And then the velocity is corrected by making the outlet mass flux the same as the inlet mass flux. A mathematic interpretation is

$$u_{fi} = u_{fi}^{old} \frac{\sum_{inlet} u_{f\perp} S_f}{\sum_{outlet} u_{f\perp} S_f} \quad (3.63)$$

where u_{fi} = the velocity component at the outlet boundary (i ranges from 1 to 3) and the superscript *old* means the value is an initial estimation as mentioned above. As this also provides a way to correct the velocity field at the outlet, the velocity is no need to be corrected again in the SIMPLE procedure.

(3) Impermeable wall

The no-slip condition is the appropriate condition for velocity components at both the riverbed and the side- walls. However the wall function approach is preferred here to avoid the possible integration through the viscous sub-layer and implement the wall roughness more flexibly. In the wall function approach, the near wall CV velocity is assumed to be parallel to the wall and denoted by $u_{//}$ (see Fig. 3.8). Although it is not always the case, the treatment can be simplified without significant influence on the result.

As is known, with the definition of the dimensionless distance y^+ and dimensionless velocity u^+ as follows

$$y^+ = \frac{u_* y_{\perp}}{\nu} \quad (3.64)$$

$$u^+ = \frac{u_{//}}{u_*} \quad (3.65)$$

where u_* = the friction velocity near the bed and $y_{\perp}$ = the normal distance from the center of the near wall CV to the wall surface, the universal wall function can be expressed by

$$u^+ = \frac{1}{\kappa} \ln(E_r y^+) \quad (3.66)$$

where κ = the van Karman constant (= 0.41) and E_r = the roughness parameter of the wall. Assuming that the flow is in local equilibrium, i.e. the production and dissipation rate of the turbulence are nearly equal, one can obtain

$$u_* = C_{\mu}^{1/4} k_P^{1/2} \quad (3.67)$$

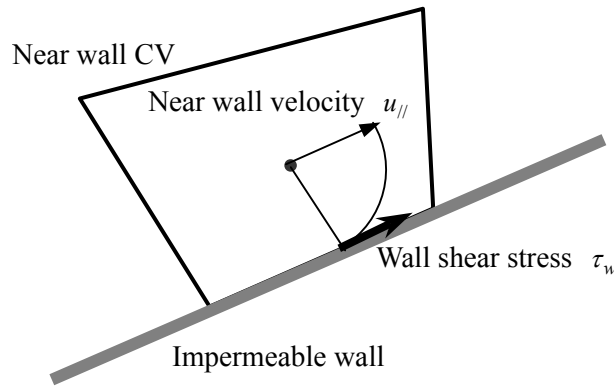


Fig. 3.8 Near Wall Treatment

Then the wall shear stress is written as

$$\tau_w = \rho u_*^2 = \frac{\rho C_{\mu}^{1/4} k_P^{1/2} u_{//}}{u^+} \quad (3.68)$$

In the momentum equations, the link with the wall is suppressed by setting it to zero and adding the wall force from Eq.3.68 as a source term. The normal derivative of k at the wall boundary CV is set to be zero in the $k - \varepsilon$ equation, and the production in the wall region is computed from

$$G_p = \frac{\tau_w}{\rho} \frac{\partial u_{//}}{\partial n} = \frac{\tau_w}{\rho} \frac{u_{//}}{y_{\perp}} \quad (3.69)$$

ε in the near wall CV is directly set to

$$\varepsilon_p = \frac{u_*^3}{\kappa y_\perp} = \frac{C_\mu^{3/4} k_p^{3/2}}{\kappa y_\perp} \quad (3.70)$$

The roughness parameter in Eq. 3.66 provides a simple way to take the bed roughness into account. This is very important to calculate the flow and sediment transport with changeable bed. Suggested by some researchers (e.g. Wu et al., 2000 and Salaheldin, 2004), this parameter may be evaluated as below.

$$E_r = \exp[\kappa(B_0 - \Delta B)] \quad (3.71)$$

where $B_0=5.2$, is a constant; ΔB = roughness function defining the shift of the intercept due to the roughness effect, it is evaluated from

$$\Delta B = \begin{cases} 0 & k_s^+ < 2.25 \\ B_b \sin[0.4285(\ln k_s^+ - 0.811)] & 2.25 \leq k_s^+ < 90 \\ B_b & k_s^+ \geq 90 \end{cases} \quad (3.72)$$

in which

$$B_b = B_0 - 8.5 + \frac{\ln k_s^+}{\kappa} \quad (3.73)$$

$$k_s^+ = \frac{u_* k_s}{\nu} \quad (3.74)$$

and k_s is the equivalent roughness height, which is a quantitative estimation of the influence of the bed roughness elements such as the sand grains and sand waves and other bed forms. For a hydraulic smooth bed, $k_s=0$, and for a rough bed, many empirical relations are available in the literatures according to the bed conditions (e.g. Garde and Ranga Raju, 1985; Chang, 1988; van Rijn, 1984; van Rijn, 1993).

On a flat bed, van Rijn suggested a value of $k_s = 3d_{90}$ (here d_{90} is the sediment size with 90% finer) based on about 100 flume and field data. On a riverbed with bed form, the following expression is used

$$k_s = 3d_{90} + 1.1\Delta(1 - e^{-25\Psi}) \quad (3.75)$$

where $\Psi = \Delta / \Lambda$, is the bed-form steepness, and Δ and Λ are the height and length of the bed form. The bed-form length $\Lambda = 7.3h$ (here h is the water depth) and the bed-form steepness is calculated from

$$\Psi = \frac{\Delta}{\Lambda} = 0.015 \left(\frac{d_{50}}{h} \right)^{0.3} (1 - e^{-0.5T})(25 - T) \quad (3.76)$$

where T = the bed shear stress parameter, is defined as

$$T = \frac{\tau_g - \tau_c}{\tau_c} \quad (3.77)$$

where τ_c is the critical shear stress for sediment motion which will be introduced in the next chapter, and τ_g is the grain-related shear stress which is evaluated with the following expression

$$\tau_g = \rho g \left(\frac{\bar{u}}{C'} \right)^2 \quad (3.78)$$

in which, $\bar{u}$ is the depth averaged velocity, and C' is the grain-related Chezy coefficient. The latter is calculated from

$$C' = 18 \log \left(\frac{12h}{3d_{90}} \right) \quad (3.79)$$

It is noted that the bed shear stress parameter is also used by van Rijn as one of the most important parameters to classify the bed forms generated in sand-bed rivers (van Rijn, 1984 and van Rijn, 1993).

3.3.5 Solution methods

(1) Under-relaxation

The final AES (Algebraic equation system) is under-relaxed before submitted to the linear equation solver. The widely used method proposed by Patankar (1980) is adopted here.

Despite of the discretization methods, the final AES resulted from the discretization process for the iteration step n may be written as

$$a_P \phi_P^n = \sum_{nb} a_{nb} \phi_P^n + b_P \quad (3.80)$$

Since the equations are linearized and decoupled, the direct use of the solution of Eq. 3.80 for the next iteration step may cause numerical instability during the iteration process. Hence, a relaxation coefficient is usually introduced as follows

$$\phi^n = \phi^{n-1} + \alpha_\phi (\phi - \phi^{n-1}) \quad (3.81)$$

where ϕ^n = calculated variables at the current iteration step; ϕ^{n-1} = calculated variables at the previous iteration step; ϕ = solution of Eq. 3.80 and α_ϕ = relaxation coefficient. If Eq. 3.80 is introduced to Eq. 3.81, i.e. the variable ϕ in Eq. 3.81 is replaced by

$$\phi = \frac{\sum_{nb} a_{nb} \phi_{nb}^n + b_p}{a_p} \quad (3.82)$$

one can obtain

$$\frac{a_p}{\alpha_\phi} \phi_p^n = \sum_{nb} a_{nb} \phi_{nb}^n + b_p + \frac{1 - \alpha_\phi}{\alpha_\phi} a_p \phi_p^{n-1} \quad (3.83)$$

This equation may be further written as

$$a_p^* \phi_p^n = \sum_{nb} a_{nb} \phi_{nb}^n + b_p^* \quad (3.84)$$

in which

$$a_p^* = \frac{a_p}{\alpha_\phi} \quad (3.85)$$

$$b_p^* = b_p + \frac{1 - \alpha_\phi}{\alpha_\phi} a_p \phi_p^{n-1} \quad (3.86)$$

It may be seen that Eq. 3.84 has the same form as that of Eq. 3.80. However, the diagonal dominance of the coefficient matrix a has been increased. This may lead to more efficient solution processes. So in the program, Eq. 3.84 is solved in the iteration process instead of Eq. 3.80.

(2) Equation solver

Solution of Eq. 3.84 is the most time consuming part of the computation. Suitable solvers should be sought in order to achieve an efficient simulation. Considering the sparseness and non-symmetry characteristics of the coefficient matrices, Krylov subspace iterative methods are preferred.

The most popular and representative methods may include GMRES (Generalized minimal residual method), Bi-CGSTAB (Bi-conjugate gradient stabilized method) and TFQMR (Transpose-free variant of quasi-minimal residual method) (Zhang, 2000). In order to get

practically useful, the iterative solver usually works with a suitable preconditioner. In this study, two solvers are integrated in the program: a Bi-CGSTAB solver and a preconditioned GMRES solver together with an ILUTP (Incomplete LU factorization with threshold and pivoting) preconditioner. These solvers have been confirmed to be effective in CFD computations (e.g. Deng et al., 1996 and John, 2004). The detailed information about the two solvers may be found in the publications by van der Vorst, 1992; Sleijpen and Fokkema, 1993 and Saad, 2002.

(3) Convergence criteria

There are two levels of iterations: the inner iterations and the outer iterations. Within one inner iteration, the linear equation system for one quantity (e.g. the velocity component u) is solved with the aforementioned equation solvers. While the outer iterations are used to deal with the non-linearity and coupling of the equation systems. The selection of the criteria to stop the iterations in different levels is very important for both accuracy and efficiency.

The reduction of the residual is used as the convergence criteria in this thesis. In the iterations, the equation system is considered to be converged if the residual norm reduced to some fraction of its original value. A mathematical interpretation is

$$\frac{\|r^n\|}{\|r^0\|} \leq \varepsilon_t \quad (3.87)$$

where r^n = the residual of step n ; r^0 = the initial residual and ε_t = the prescribed tolerance for stopping the iteration. In the outer iteration, this tolerance may be fixed and assigned to a very small value. For the inner iteration, the tolerance is not a fixed value, it changes with the calculated residual level of the outer iteration.

As the equation system is linearized and de-coupled, it is no need to solve the equation system very accurately, i.e. a relatively large tolerance for the inner iterations at the beginning may be suggested. Experiences show that a reduction of the residual level by 10% to 1% is suitable for a quicker convergence of the outer iterations. After that, the calculated residual norm prior to the first inner iteration provides a reference to check the convergence of the inner iterations. In other words, the tolerance of the inner iteration will decrease with the reduction of the calculated outer iteration residual level. If the residual norm decreased by several orders of the magnitude, the tolerance for the inner iterations should also be decreased to a much smaller value, say, 10^{-8} , depending on the problem involved.

The above treatments can guarantee that the residual levels of the inner and outer iterations are in a suitable balance. It has been found that the risk of numerical divergence could be

effectively reduced and the computation has been fastened.

(4) Solution procedure

The calculation sequences for the flow following the SIMPLE procedure may be summarized as below.

Starting from an initial riverbed geometry, all variables are assigned initial values at $t = t_0$. Time is advanced and calculation starts.

- (a) Solve the momentum equations for each velocity component, in which the other velocity components, the pressure, the eddy viscosity, the turbulence kinetic energy and its dissipation rate are treated as known. Solving of each equation system is an inner iteration.
- (b) The resulted velocity field is used to calculate the mass fluxes through CV faces. Solve the pressure correction equation and improve the velocity field.
- (c) Solve the transport equations for the turbulence kinetic energy and its dissipation rate, respectively. Update the eddy viscosity.
- (d) The above procedure is called an outer iteration. Repeat the above procedures until the residual level becomes sufficiently small or the prescribed maximum iteration step number is reached. The computed values at the current time step are used as the initial values for the new time step.
- (e) Calculate sediment transport rate and bed variation. Check the local bed slopes and adjust the bed elevation until the angles of all the local beds are not greater than the angle of sediment repose. These will be discussed in the next chapter.
- (f) Generate a new mesh according to the bed geometry. Time is then forward.
- (g) Check the maximum bed variation
 - (i) If the maximum bed change is larger than a specific number depending on the problem involved and the accuracy required (for example, 8% of the water depth), return to (a) and repeat the preceding calculation.
 - (ii) If the maximum bed change is relatively small, the flow field is assumed to be unchanged, only the sediment routine starting from step (e) will be repeated.
 - (iii) If the maximum bed change is very small (for example, less than 10^{-4} cm/s), an equilibrium condition may be assumed.
- (h) Stop the computation if the equilibrium condition is reached or a specified time is covered.

The above procedure is designed for a movable bed calculation. In the case of a fixed bed calculation, step (e), step (f) and step (g) are skipped.

3.3.6 Model verification

In order to verify the applicability of the proposed methodology, the model is applied to predict some flow phenomena in laboratory flumes under fixed bed conditions. The verification of the moving mesh strategy will be made after the construction of the sediment transport models.

(1) Flow in a rectangular channel

The flow in a rectangular channel is one of the simplest hydraulic phenomena in nature. Since the nature of the flow is simple, any valid model should be able to reproduce the flow fundamentals. This flow phenomenon may serve as a suitable benchmark problem to test the performance of the new model. The flow structure and its mechanism have been analyzed by Nezu and Nakagawa, 1993; Ishigaki, 1993 and Nezu, 2005. In this verification, the model is applied to one of the laboratory experiments carried out by Imamoto et al (1987). The experiment was conducted in a straight glass flume. The flume is 8m long, 20cm wide and 23cm deep. An LDV (Laser Doppler velocimeter) system was used to measure the fully developed turbulent flow velocities in a transverse cross-section. Detailed measurements were reported by Imamoto and Ishigaki (1985) and Imamoto et al. (1987). The experimental conditions are shown in Table 3.2. The aspect ratio is $B/h = 5$. According to the classification of Nezu (2005), this aspect ratio is almost a critical value for a wide channel and a narrow channel.

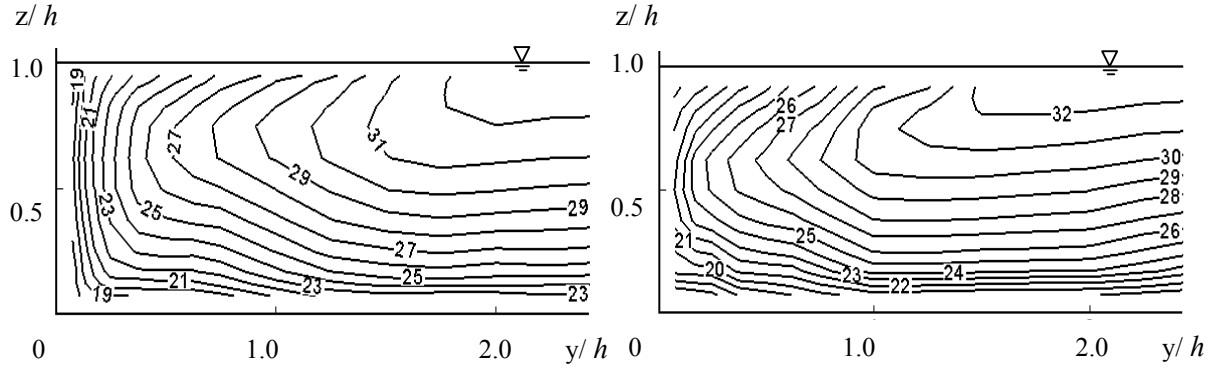
Table 3.2 Experiment conditions

Width B (cm)	Depth h (cm)	Discharge Q (l/s)	Slope I_e	Reynolds number	Froude number
20.0	4.00	2.055	1/1,400	7,700	0.48

As the experimental flume is geometrically symmetrical, only half of the study domain is selected as the computational domain by employing a symmetrical plane boundary. A longitudinal distance of 140cm ($=35h$) is assumed to be long enough to eliminate the influence of the inlet and outlet boundaries and has been chosen for the simulation after some trial computations. A logarithmic velocity profile is prescribed in the vertical direction at the inlet boundary. The turbulent quantities k and ε are specified with Eq. 3.88 and Eq. 3.89, corresponding to a viscosity ratio of 10.0 (i.e. $\nu_t = 10\nu$) and taking the turbulence intensity $I = 8\%$.

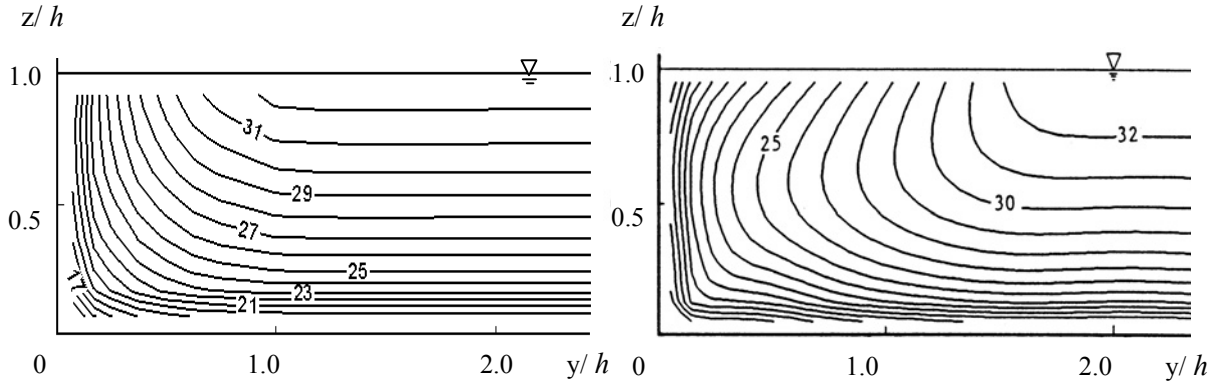
$$k = 1.5(uI)^2 \quad (3.88)$$

$$\varepsilon = C_\mu \frac{k^2}{v_t} \quad (3.89)$$



(a) Experimental result

(c) Non-linear model, structured mesh



(b) Linear model, unstructured mesh

(d) ARS model, structured mesh

Fig. 3.9 Comparison of the Primary Velocity u (cm/s)

Both the standard $k-\varepsilon$ model and some non-linear $k-\varepsilon$ models are implemented on a hexahedral mesh with a total number of 10,626. The non-linear $k-\varepsilon$ model proposed by Kimura and Hosoda (2003) seems to perform best amongst all the non-linear models integrated in the program. The result of this model is thus selected as the representative of the non-linear models. Ishigaki (1993) also simulated the same experiment with two ARS models incorporated

with the wall function approach based on a structured mesh method. The result of one of the models proposed by Naot and Rodi (1982) is also shown here as a representative of the structured mesh methods. In this verification, the coordinate components x , y and z stand for the longitudinal direction, the transverse direction and the vertical direction, respectively. And the corresponding velocities are u , v and w , respectively.

The isovels of the primary velocity u is shown in Fig. 3.9 including the experimental measurement and the computational results of the standard $k-\varepsilon$ model, the non-linear $k-\varepsilon$ model and an ARS model based on a structured mesh. The interval of the isovels is 1 cm/s. The experimental result shows that this channel has some characteristics of typical wide channels. There is no obvious velocity-dip phenomenon at the centerline of the channel (i.e. $y/h = 2.5$). Idealized 2D time-averaged flow is attained at the center of the flume. However, the velocity is significantly distorted away from the centerline. The velocity on the free surface is smaller than that just below the surface in particular around $y/h = 1$. This is due to large-scale secondary currents just below the free surface which are caused by the turbulence inhomogeneity and anisotropy.

The performances of different turbulence models are very easily observed from the comparison. A standard $k-\varepsilon$ model is able to capture the basic properties of the flow. But it does not include any information of the turbulence-driven secondary currents, which in return affects the prediction of the flow pattern especially near the free surface and the corner area. On the other hand, both the non-linear model and the ARS model reproduce the primary velocity field quite well because they contain more information such as the turbulence anisotropy. It is very difficult to distinguish the winner between the non-linear $k-\varepsilon$ model and the ARS model in this test case although the latter one is theoretically more accurate than the former one. However, it does demonstrate that the unstructured mesh proposed in this study is applicable and the accuracy is comparable to a structured one. As the computational conditions (including both the mesh number and the computer used) for the structured mesh and unstructured mesh are quite different, the quantitative comparison of the computation time is not fulfilled. But it is observed that the standard $k-\varepsilon$ model is much faster than the non-linear $k-\varepsilon$ model with the same unstructured mesh. For a steady state calculation, iteration step for the standard $k-\varepsilon$ model is 104; while for the non-linear one, it takes 354 iterations.

In Fig. 3.10, the comparison of the turbulence kinetic energy is shown. The absolute value of k has been normalized by the squared friction velocity u_*^2 . The interval of the iso-lines is 0.2. It seems that the non-linear model based on the unstructured mesh gives the closest results to the LDV measurement. The standard model captures only the basic information. And the ARS model gives more reasonable result near the corner and the free surface. However, in all the computational cases, the turbulence kinetic energy is slightly over-estimated either near the

side-wall or along the flume bottom. Since all these models use the wall function approach to resolve the near-wall area, it is believed that the wall function approach is responsible for these differences if the error of the experiment measurement is negligible. In order to get more information about the predicted turbulences near the wall, the distribution of the Reynolds stresses resulted from the non-linear $k-\varepsilon$ model is plotted in Fig. 3.11 compared with the experimental measurement.

Each figure shows the distribution of the Reynolds stresses from the side-wall to the center of the channel at a specific elevation. The absolute values have been normalized by the squared friction velocity u_*^2 again.

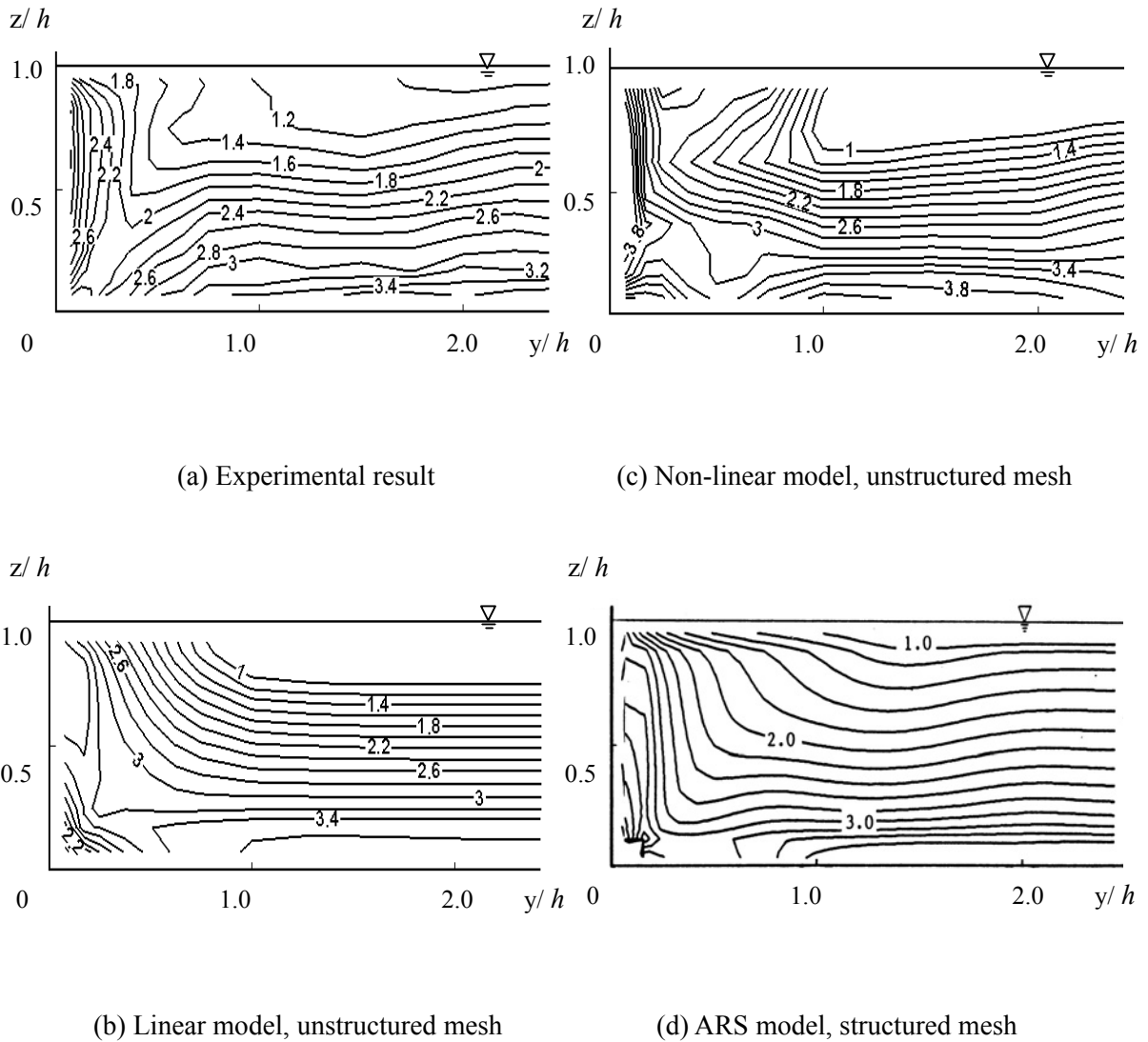
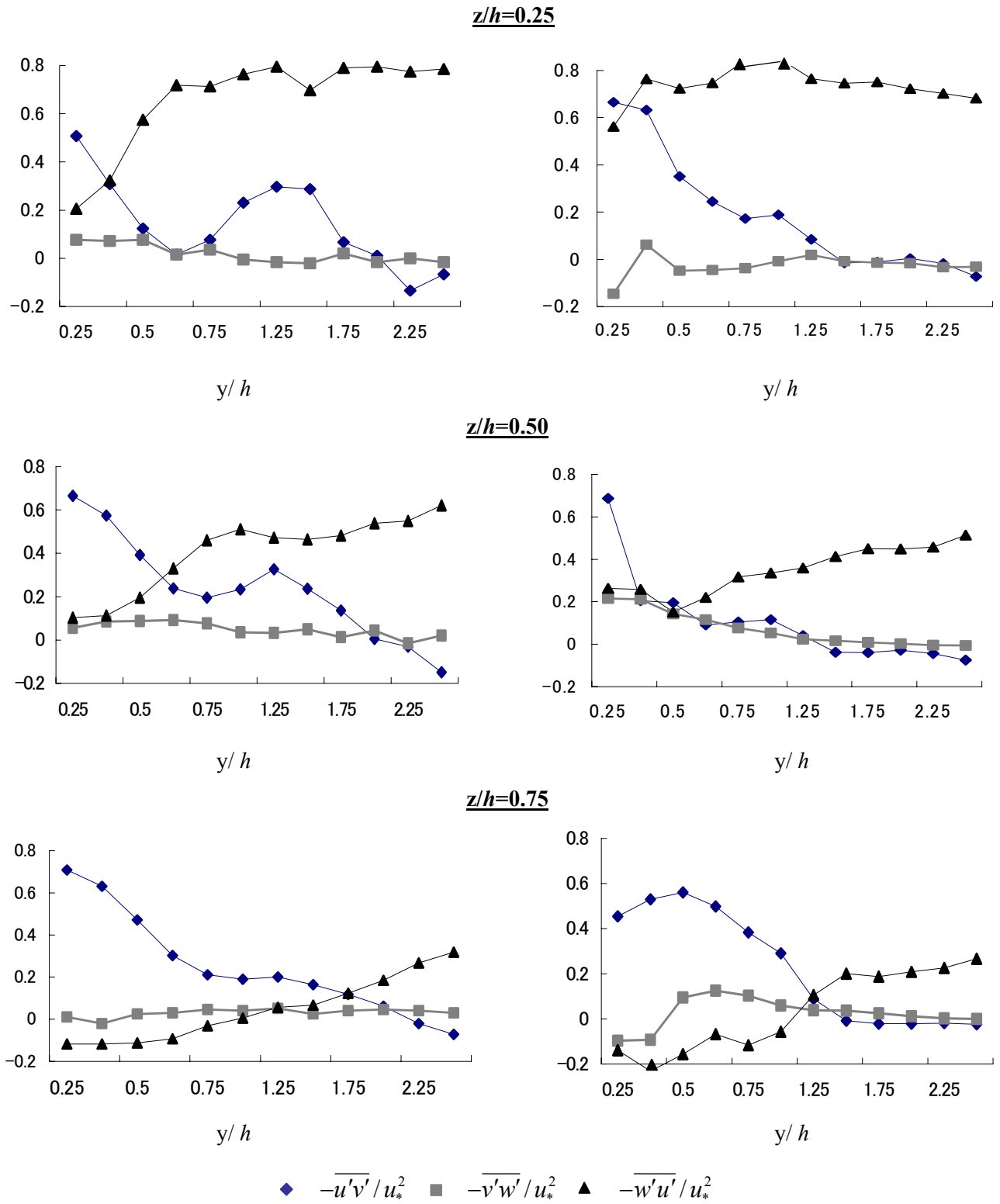


Fig. 3.10 Comparison of Turbulence Kinetic Energy Normalized by Squared Friction Velocity k/u_*^2



**Fig. 3.11 Comparison of the Lateral Reynolds Stress Distribution
(Experiment: Left; Computation: Right)**

The Reynolds stress components have been reasonably reproduced in the area where y/h is relatively large. Nevertheless, an over-estimation of the Reynolds stresses is observed near the boundary area where y/h is small. These observations coincide with what have been found in the comparison of the turbulence kinetic energy.

From this verification, one may come to the following conclusions:

- (a) Although the flow in a straight rectangular channel is simple, it includes turbulence-driven secondary motions, which have an effect on the primary velocity profile. A standard $k-\varepsilon$ model is not able to resolve this effect due to its inherent defects (e.g. the isotropic eddy viscosity hypotheses). A non-linear $k-\varepsilon$ model and an ARS model are suitable alternatives if a higher accuracy is required.
- (b) The unstructured mesh method proposed in this study is valid and the result is comparable to a structured mesh method. Since most flows of engineering interest are involved with complex geometries, the unstructured mesh method is more promising for industrial use.
- (c) A non-linear $k-\varepsilon$ model is more costly than a standard one. (About 3 times compared with the standard model in terms of iteration steps in this test case) If the turbulence model is a module in a morphological model and will be repeatedly called to calculate the flow field with time, the computation time should be taken into account.
- (d) An elaborate turbulence model may significantly improve the prediction result in the main flow. But the prediction of the near-wall area strongly relies on the bridge between the turbulence area and the impermeable wall. As the sediment transport is very sensitive to the near-wall flow, special attention should be paid.

(2) Flow around an embayment

The flow in an embayment is one of the most important hydraulic phenomena in the river engineering practice. Due to the complex flow structure, the balance of the sediment transport is broken. It results in a great number of problems, for instance, the local scour, the change of environmental parameters, etc. By using two LDAs (Laser Doppler anemometers) and an EMC (Electromagnetic current meter), Muto et al. (2000) experimentally investigated the flow exchanges between the main channel and an embayment area. In this section, the methodology is applied to this kind of flows and the result is compared with the measurements.

The experiments were conducted in a compound flume. The flume consists of a main channel which has a width of $B=16.0\text{cm}$ and a flood plain with a width $b=16.0\text{cm}$. Part of the flood plain is removed where forms an embayment with a length L . The cases of $L=16.0\text{cm}$ (i.e. $L=b$)

and $L=48.0\text{cm}$ (i.e. $L=3b$) are validated. The experiment setup is given in Fig.3.12. The experimental conditions and computational conditions for the verification are shown in Table 3.3 and Table 3.4, respectively.

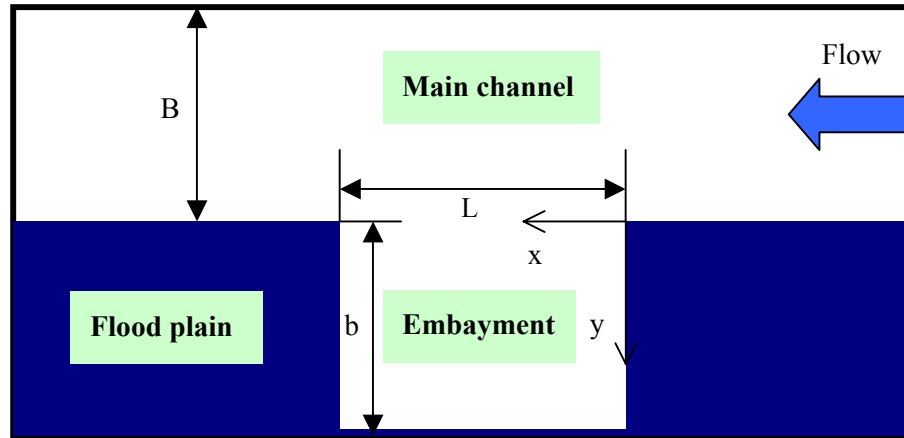


Fig. 3.12 Top View of the Experiment Setup

Table 3.3 Experimental conditions

Discharge Q (l/s)	Water depth h (cm)	Friction velocity u_* (cm/s)	Reynolds number	Froude number
2.271	3.8	1.78	9,650	0.74

Table 3.4 Computational conditions

Cases	Aspect ratio (L/b)	Meshing strategy	Numbers of CVs
Case1	1	Hexahedral/ Hybrid	19,683/13,086
Case2	3	Hexahedral	16,902

Different meshing strategies have been used as described in Table 3.4. From the standpoint of mesh generation, the hexahedral mesh is easy to generate for the current study domain. It is adopted for both cases to investigate the different flow patterns due to different aspect ratios. In order to investigate the sensitivity of the model to different meshing strategies, a hybrid polyhedral mesh of both hexahedra and prisms is also tested in Case 1. With this mesh, the areas

inside and around the embayment are tessellated by graded prisms, while in areas far from the embayment and near the boundaries, the hexahedral mesh has been used.

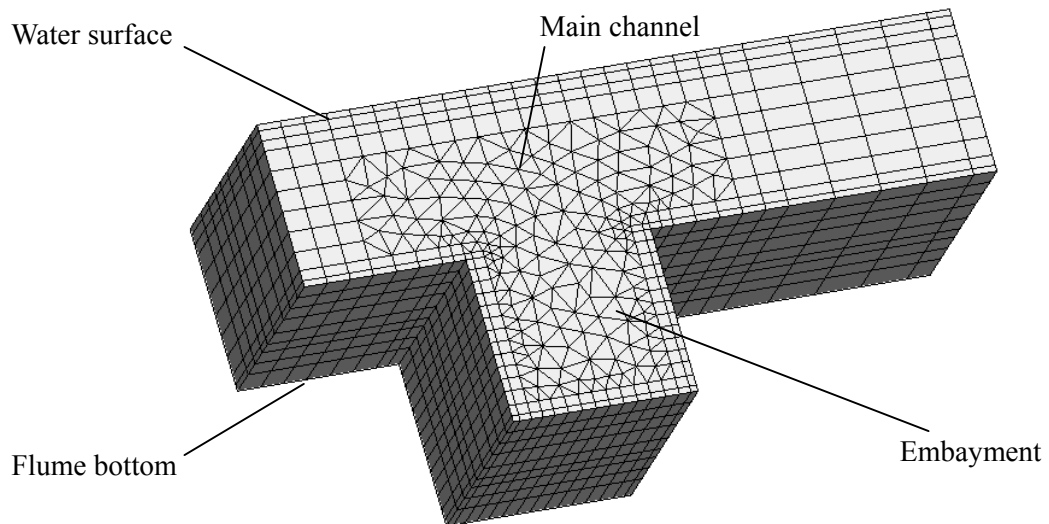


Fig. 3.13 Hybrid Polyhedral Mesh Around the Embayment in Case 1

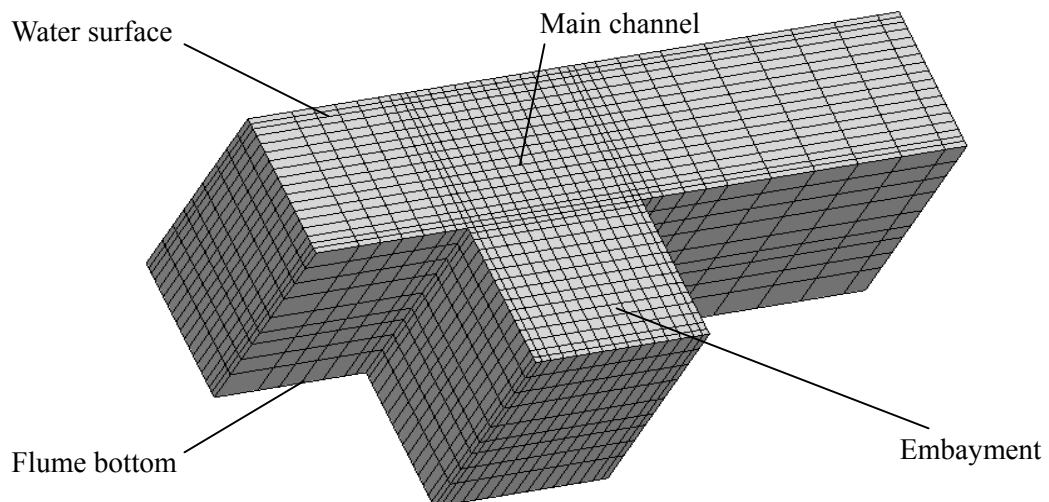


Fig. 3.14 Hexahedral Mesh Around the Embayment in Case 1

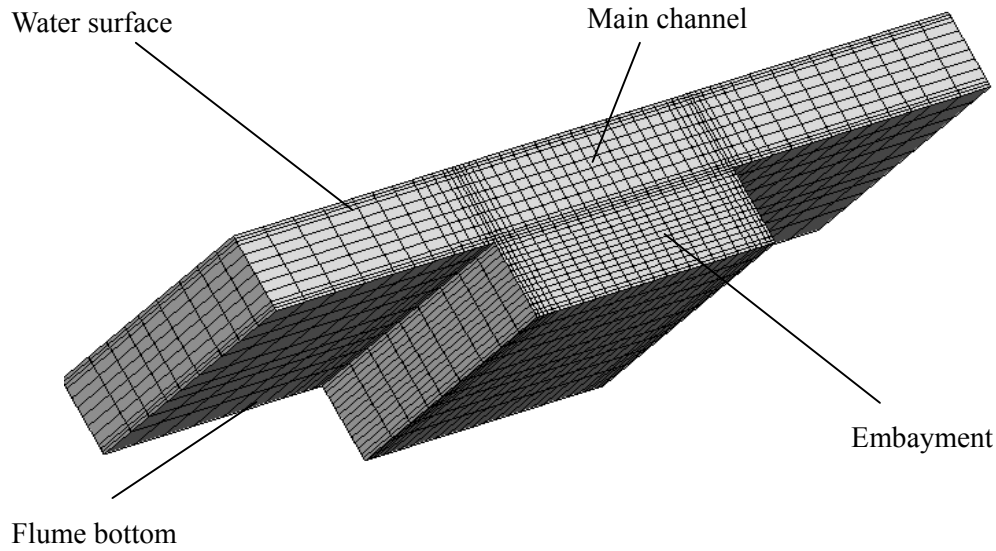


Fig. 3.15 Hexahedral Mesh Around the Embayment in Case 2

The mesh systems around the embayment in both cases are depicted in Fig. 3.13, Fig. 3.14 and Fig. 3.15.

In the computation, the inlet boundary is set at the distance of about $10b$ from the upstream of the embayment and the outlet is $20b$ from the downstream of the embayment. This distance seems long enough and may eliminate the effect of the inlet and outlet boundaries. The inlet flow is assumed to have a logarithmic velocity profile in the vertical direction. At the beginning of the calculation, this velocity profile is applied to the whole domain. The turbulence kinetic energy and its dissipation rate are specified using the same method as described in the previous verification.

(a) Result of Case 1

Two kinds of meshing strategies have been tested with a standard $k-\varepsilon$ model and a non-linear $k-\varepsilon$ model under the same initial and boundary conditions. The computed results are compared with the experimental measurement.

In Fig. 3.16, the surface flow pattern is visualized. It is very obvious that a large circulating flow occupies almost the whole embayment area. The center of the circulating flow is stagnant and locates at almost the geometrical center of the embayment. The velocity in the embayment is much smaller than that in the main channel. The circulation is also found to be stable and have almost no exchange with the main flow.



Fig. 3.16 Flow Visualization Result in the Experiment (Muto et al., 2000)

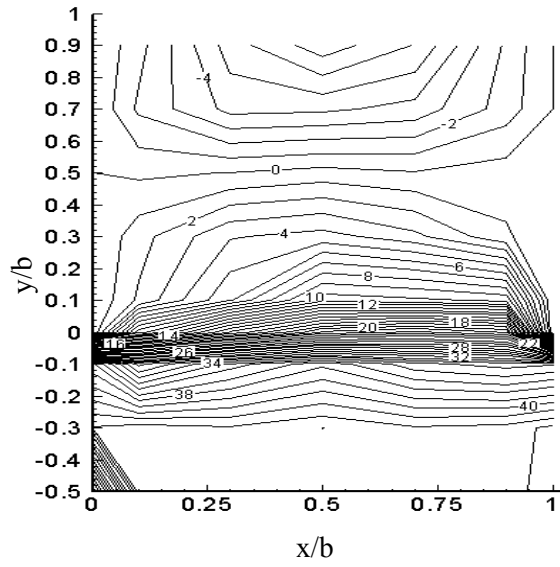
The velocity field was measured at $z=1.9\text{cm}$ (i.e. half of the water depth). The measured longitudinal velocity profile and the velocity vectors in the horizontal plane ($z=1.9\text{cm}$) are shown in Fig. 3.17 (a) and Fig. 3.18 (a). The computed results are plotted with the same scale in these two figures. Only the area in the proximity of the embayment is discussed. Moreover, the coordinate system has been normalized by the width of the embayment.

The simulation results are encouraging. The big circulating flow is reproduced by all the computations. The center of the circulation is exactly at the geometrical center of the embayment. The longitudinal velocity at $y/b = 0.5$ is almost zero. At the interface between the embayment and the main channel, there is steep velocity gradient, in particular around $x/b = 0$ and $x/b = 1$.

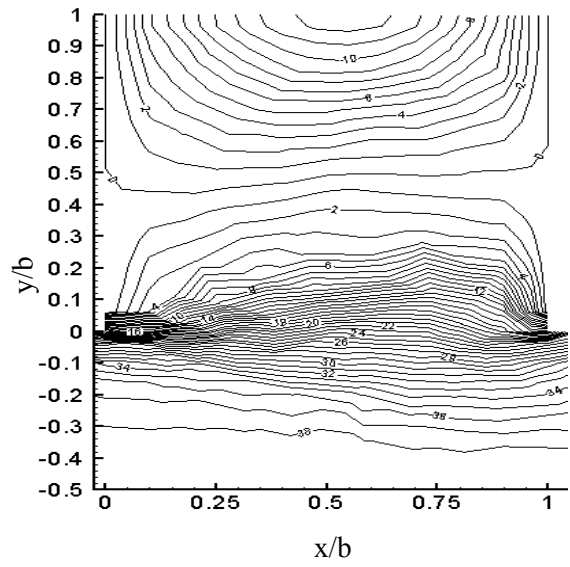
It may be found that the standard $k-\varepsilon$ model and the non-linear $k-\varepsilon$ model based on the hybrid mesh give almost the same result in terms of not only the flow pattern but also the magnitude of the velocities. This indicates that the turbulence-driven secondary motion in this case is very weak. The shape of the geometry plays a more important role compared with the turbulence anisotropy. In other words, the secondary currents of Prandtl's first kind are predominant. As a result, the non-linear terms introduced in the non-linear $k-\varepsilon$ model are almost negligible. However, these non-linear terms still take a lot of computational time because the standard $k-\varepsilon$ model is again found to be much faster than the non-linear one.

Concerning the different mesh systems, i.e. the hexahedral mesh and the hybrid mesh, the difference between the computed results is not very evident. However, it has to mention that the hybrid mesh is found to be a little more costly than the hexahedral mesh although the former one is coarser than the latter one in this test case. This may be caused by the mesh skewness.

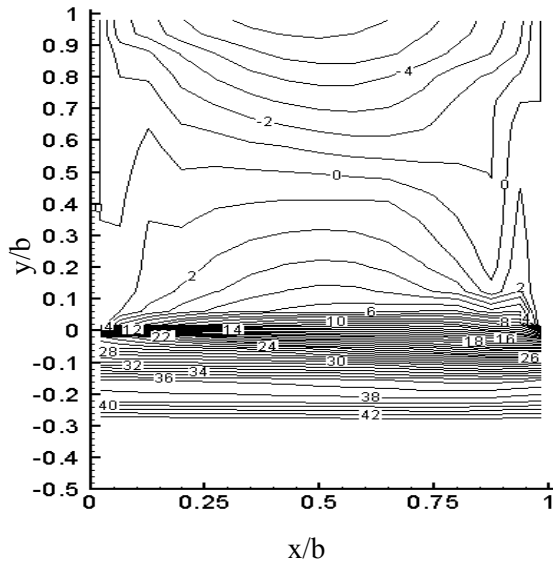
The hexahedral mesh in this case is orthogonal, while some of the CVs in the hybrid mesh are skewed.



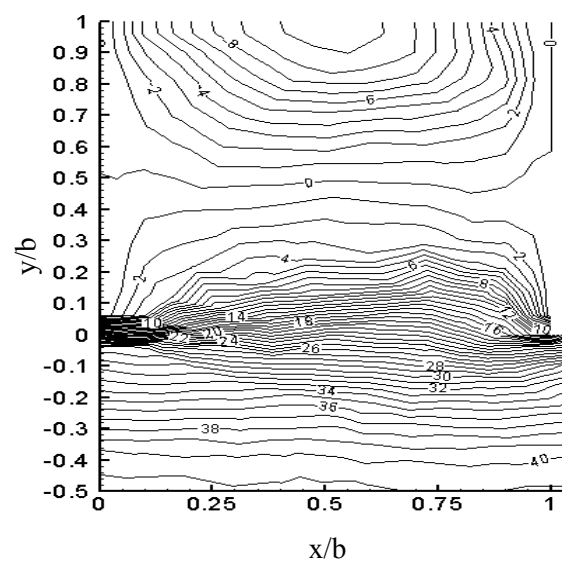
(a) Experiment result



(b) Hybrid mesh & Standard model result

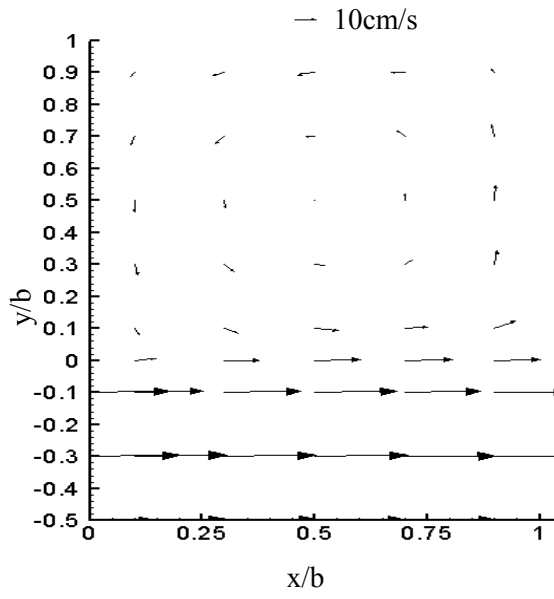


(c) Hexahedral mesh & Non-linear model result

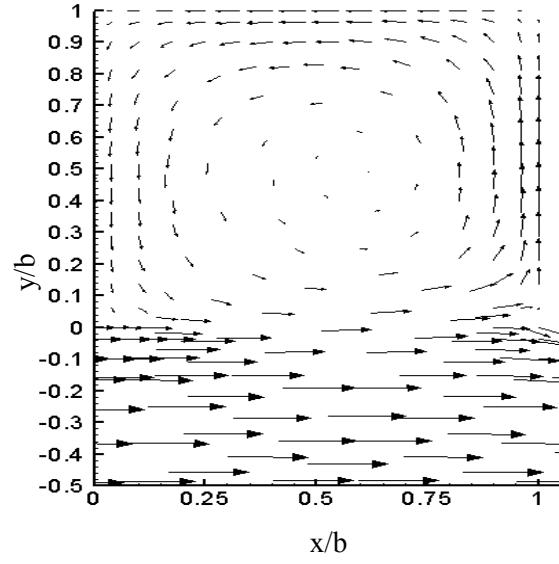


(d) Hybrid mesh & Non-linear model result

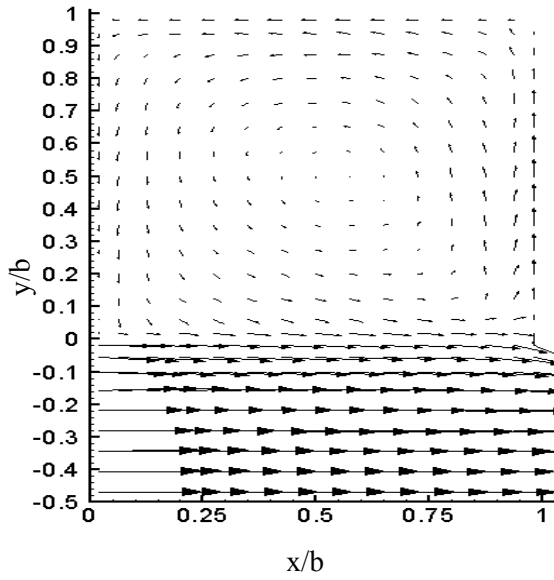
Fig. 3.17 Comparison of the Longitudinal Velocity Profile u



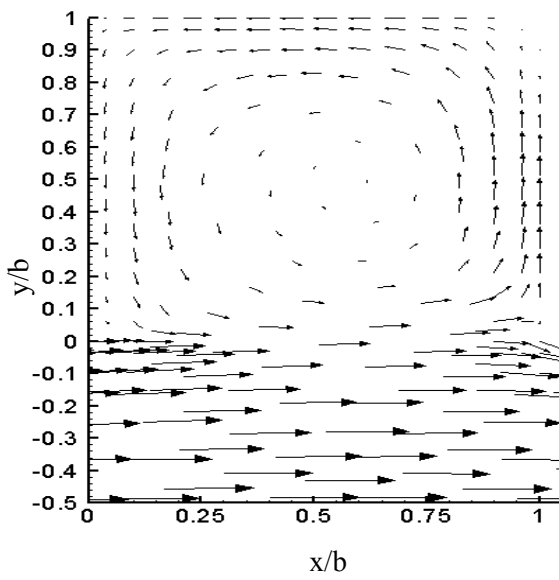
(a) Experiment result



(b) Hybrid mesh & Standard model result



(c) Hexahedral mesh & Non-linear model result



(d) Hybrid mesh & Non-linear model result

Fig. 3.18 Comparison of the Horizontal Cross-sectional Velocity Vectors (u, v)

Fig. 3.19 shows the velocity vectors in some representative transverse cross-sections predicted by the non-linear $k-\varepsilon$ model based on the hexahedral mesh. Shortage of experimental data, the comparison with measurement is not fulfilled. However, the understanding of the 3D flow structure may be furthered through the computation.

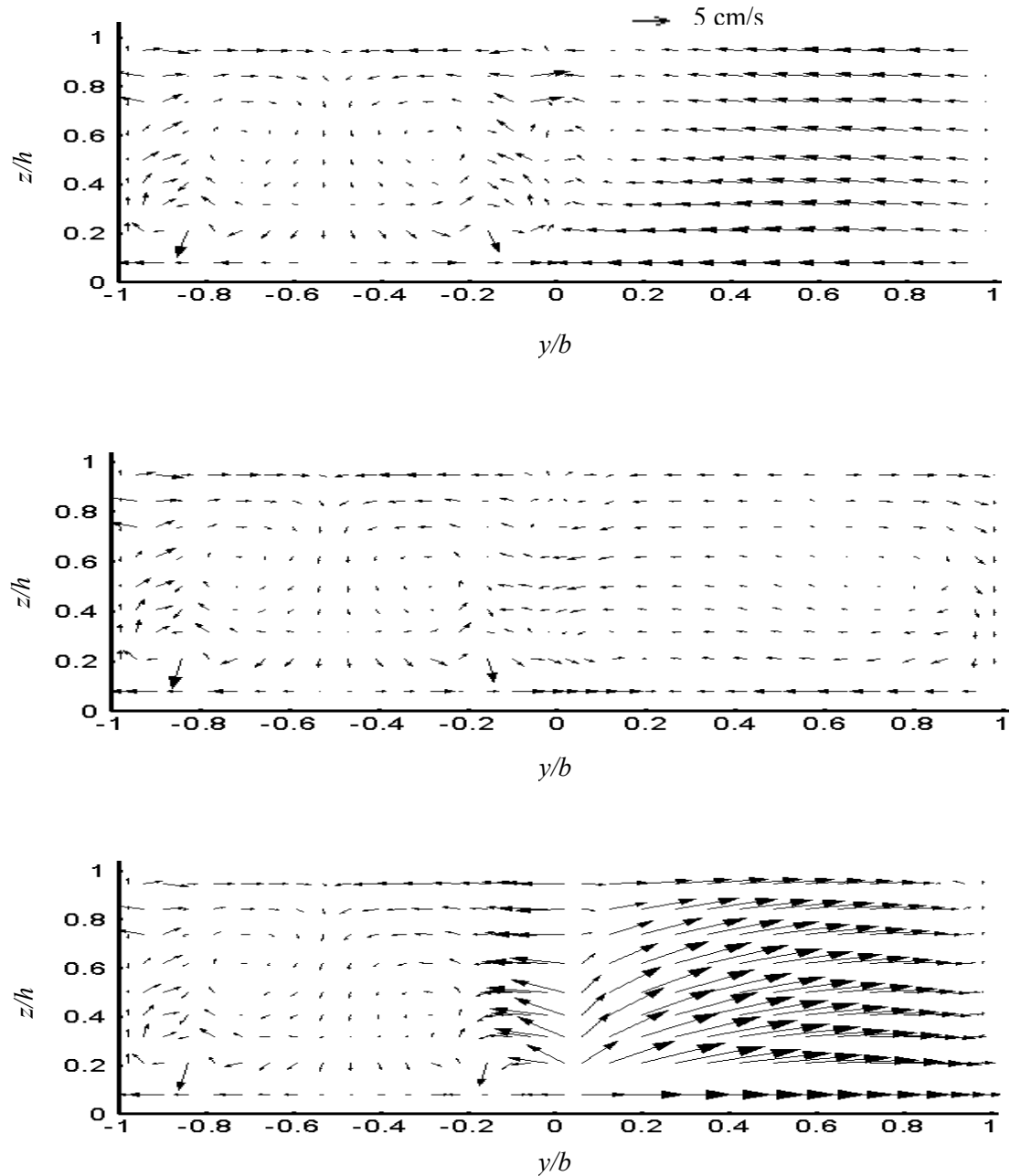


Fig.3.19 Transverse Cross-sectional Flow Vectors (v, w)
 ($x/b \approx 0$: Top; $x/b = 0.5$: Middle; $x/b \approx 1$: Bottom)

These cross-sections are located at the near-inlet of the embayment ($x=0.3\text{cm}$, i.e. $x/b \approx 0$), the center area of the embayment ($x=8.0\text{cm}$, i.e. $x/b = 0.5$) and the near-outlet of the embayment ($x=15.7\text{cm}$, i.e. $x/b \approx 1$). The propagation of the velocity profile in the cross-sections including the embayment area from the upstream to the downstream can be obviously observed. In the main channel area, there are two large secondary-current cells in the vertical plane: one is clockwise and the other is anti-clockwise. The two secondary-current cells transport from the upstream to the downstream without observable exchanges with the embayment. This agrees with what has been found in the surface visualization. Significant momentum exchanges occur at the interface between the main channel and the embayment, but it is limited to a very small range. In the embayment, the direction of the transverse velocity in the upstream is almost opposite to that in the downstream. No obvious secondary currents can be observed in the embayment area.

(b) Result of Case 2

The surface flow visualization is shown in Fig. 3.20. A large circulating flow occupies the latter three fourth of the embayment. At the upstream corner area in the embayment, a small eddy can be observed, which is very weak compared with the large one. Different from Case 1, the large eddy in this case is not stable, it exchanges with the main flow frequently.

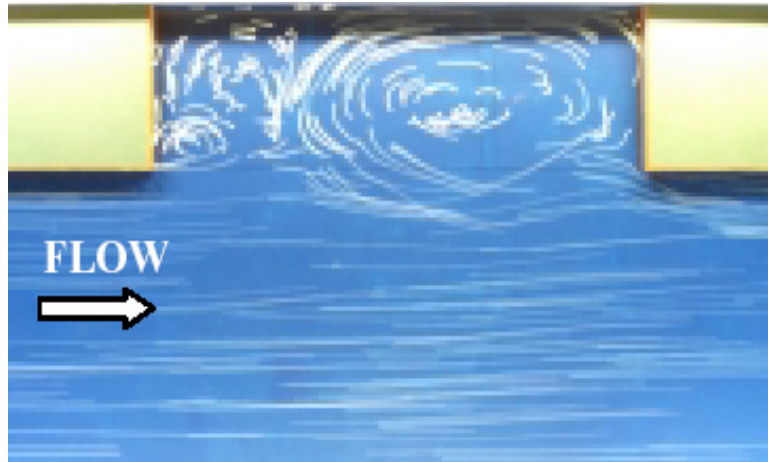


Fig. 3.20 Flow Visualization Result in the Experiment (Muto et al., 2000)

Both the standard $k-\varepsilon$ model and the non-linear $k-\varepsilon$ model are implemented with the same hexahedral mesh, some parts of which have been shown in Fig. 3.15. The computed results and experimental data at half of the water depth are given in Fig. 3.21 and Fig. 3.22.

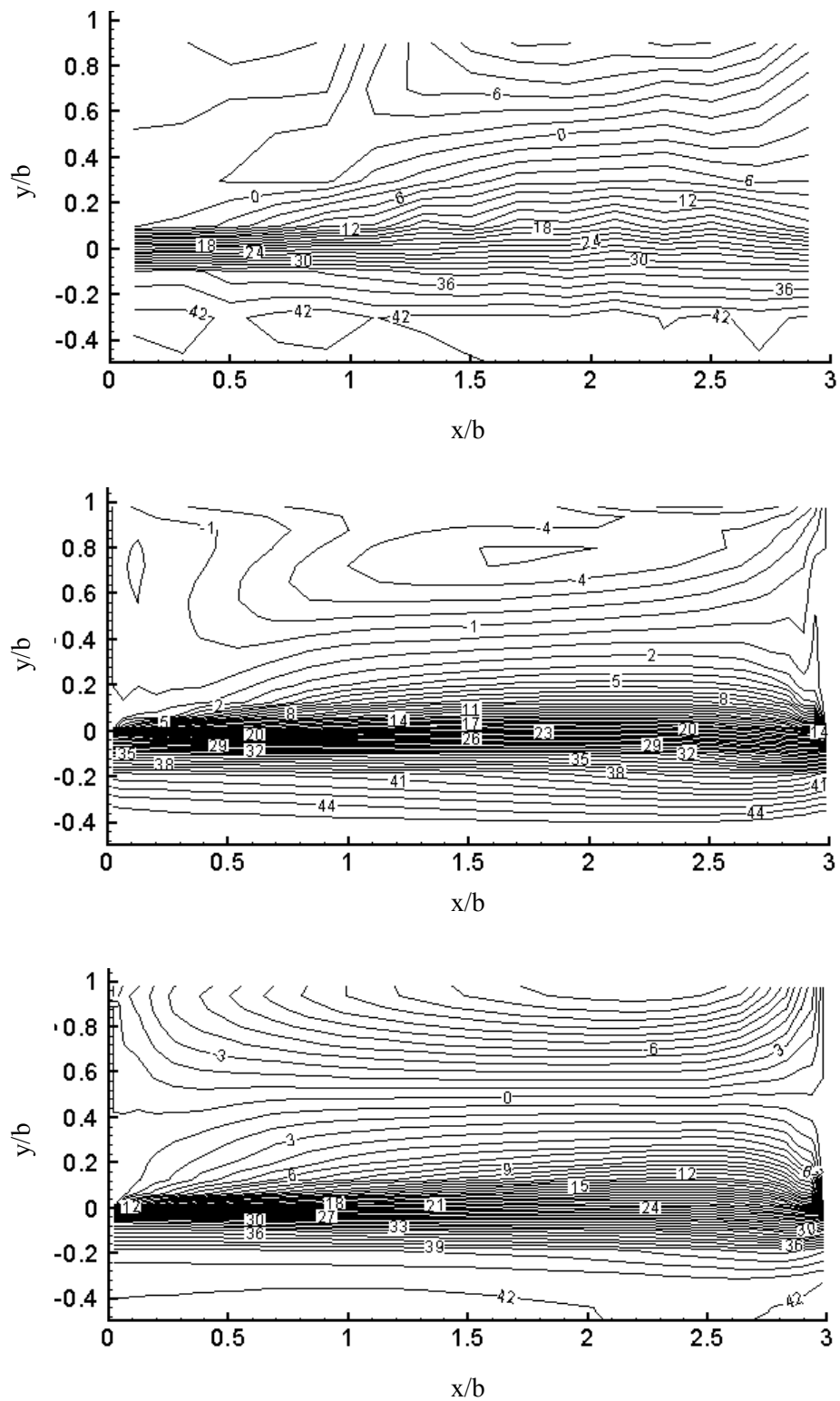


Fig. 3.21 Comparison of the Longitudinal Velocity Profile
(Experiment: Top; Non-linear model: Middle; Standard model: Bottom)

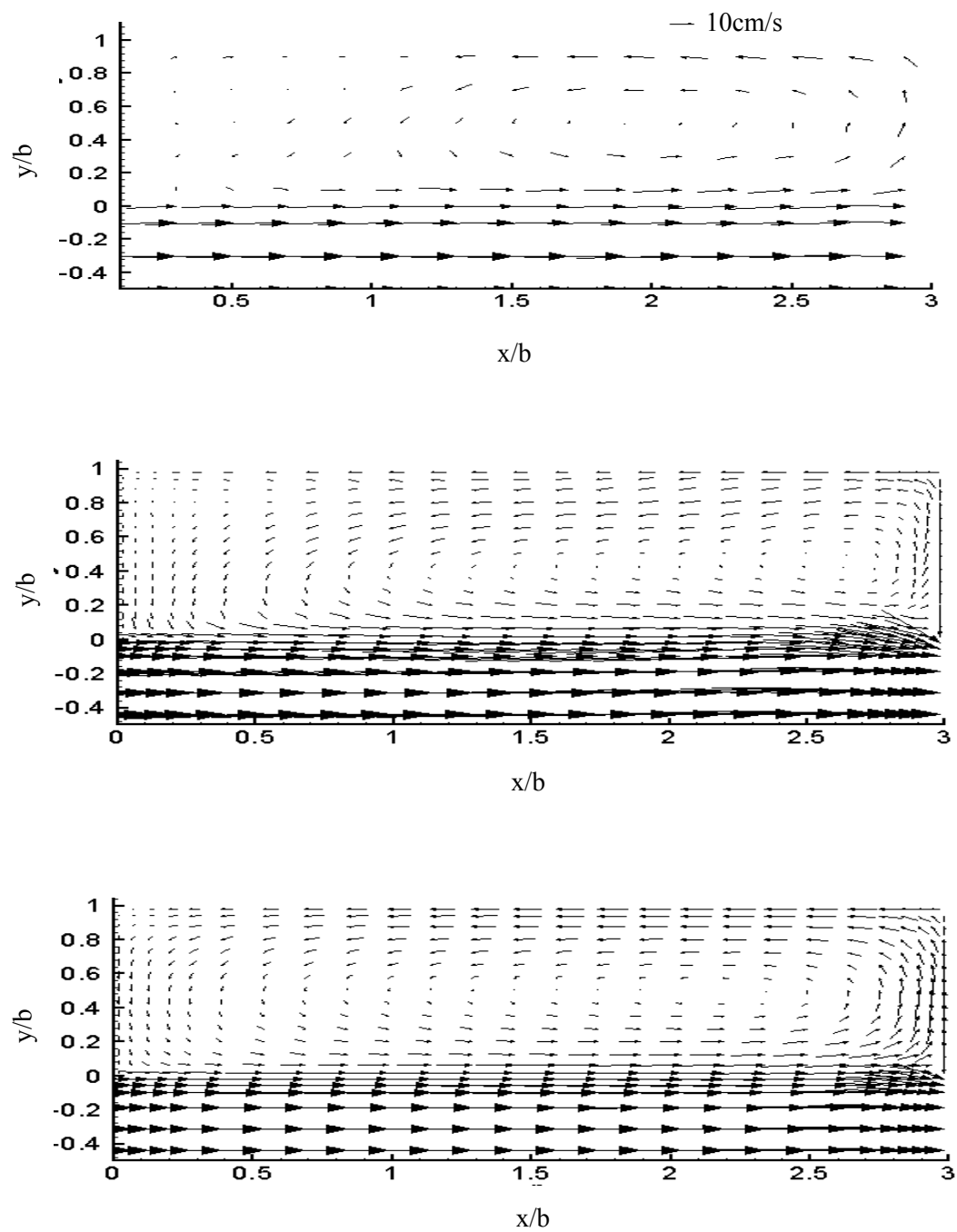


Fig. 3.22 Comparison of the Horizontal Cross-sectional Velocity Vectors (u, v)
(Experiment, Top; Non-linear model: Middle; Standard model: Bottom)

In Fig. 3.21, the longitudinal velocity profile around the embayment is depicted. And the

horizontal cross-sectional velocity vectors are shown in Fig. 3.22. One may have a look at the experimental result firstly. The longitudinal velocity profile in Case 1 is almost symmetrical with respect to $x/b = 1.5$, but it is much more complex in this case due to different circulating flows. The flow field may be divided into three zones laterally: a main channel zone, an embayment zone and a mixing zone. The flow in the main channel is relatively simple and is almost parallel to the bank. A large eddy occurs at the latter part of the embayment, which has also been observed at the free surface. The velocity in the upstream area in the embayment is very small especially near the corner area. This area may be considered as a stagnant zone. Significant velocity gradient appears in the mixing zone. The gradient is rather large when the flow just approaches the embayment and just leaves the embayment.

In Case 1, the predicted results of the standard $k-\varepsilon$ model are very close to those of the non-linear $k-\varepsilon$ model. However, one may find obvious differences between the two models in this case. The flow pattern especially the flow in the embayment predicted by the non-linear $k-\varepsilon$ model seems to be better than the standard $k-\varepsilon$ model. In the standard $k-\varepsilon$ model result, the big circulating flow occupies almost the whole domain of the embayment. A stagnant zone can be found only in a very small area around the upstream corner. The longitudinal velocity also seems to be a little more symmetrical than either the experiment or the non-linear $k-\varepsilon$ model result.

However, a further inspection shows that the current non-linear $k-\varepsilon$ model over-estimates the velocity in the main channel area, while under-estimates the velocity in the embayment near the bank. Both the experiment and the standard $k-\varepsilon$ model show that the maximum positive velocity is about 42cm/s in the main channel and the minimum negative velocity is about -12cm/s in the embayment at this layer. But the maximum velocity predicted by the non-linear $k-\varepsilon$ model is around 47cm/s while the minimum velocity is around -7cm/s. Moreover, it has to mention that the non-linear model is found to be very difficult to get converged under the same initial conditions as the standard one in this test case. The result of the standard $k-\varepsilon$ model has been finally used as the initial flow conditions for the non-linear model. And the convergence was found to be significantly accelerated.

From this verification, one may come to the following conclusions.

(a) The aspect ratio of the embayment determines the shape and the stability of the circulating flow induced in the embayment, which furthermore affects the exchange process between the embayment and the main flow. In a square embayment, a large horizontal circulating flow occupies the whole domain of the embayment and has almost no exchange with the main flow. With the increase of the embayment length, the large circulation goes to the latter part of the embayment and small eddies occur in the former part especially near the upstream corner area.

The exchange process is also found to be much more effective.

(b) The importance of the aspect ratio has been also documented by some other researchers. According to Nakagawa et al. (1995), the number and the shape of the circulations in the embayment change with the aspect ratio. Uijtewaal et al. (2001) observed two different types of exchange processes. For embayment with small aspect ratio, the large eddies in the mixed layer are the main causes. For embayment with large aspect ratio, the mechanism is governed by the large shed vortices when the flow approaches the embayment. These observations may serve as supplements for the current study.

(c) The performance of the $k-\varepsilon$ model also differs with the change of the aspect ratio. When the aspect ratio is small, a standard $k-\varepsilon$ model gives almost the same result as a non-linear $k-\varepsilon$ model. With the increasing of the aspect ratio, the differences become evident. In a straight channel which may be treated as an embayment with an aspect ratio of infinity, one gets the largest differences as described in the first verification. This phenomenon may be explained from the classification of secondary currents as shown in Fig. 3.23. It is easy to understand that the effect of the channel geometry becomes more and more important with the decreasing of the aspect ratio. As a result, the secondary flows of the 2nd kind (turbulence-driven secondary flow) plays a less and less important role. This leads to a result that the non-linear model loses its advantages over the standard one even if the latter is not able to resolve the 2nd kind of secondary flows.

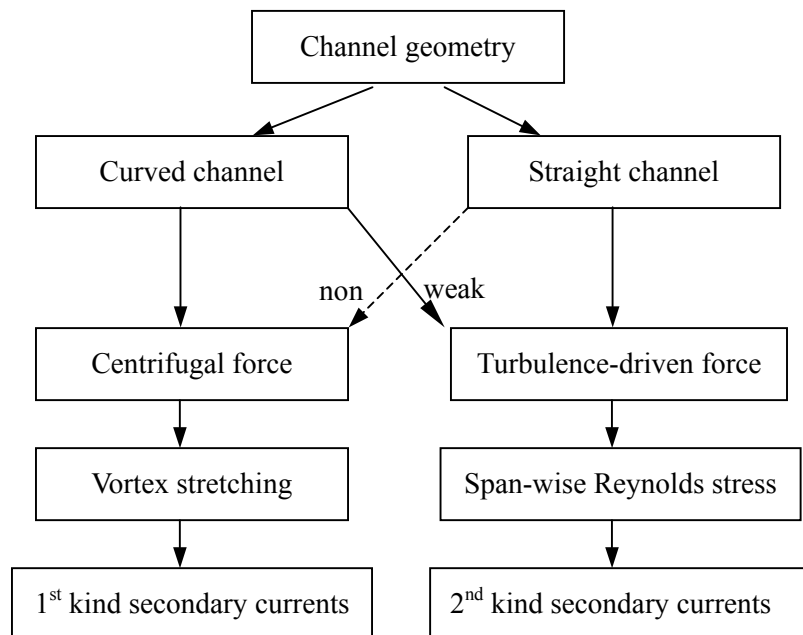


Fig. 3.23 Classification of Secondary Currents (after Nezu, 2005)

(d) Although the non-linear $k - \varepsilon$ model is more elaborate than a standard one theoretically. In practice, it is not always the better solution compared with the standard one. Sometimes, the accuracy of the turbulence models seems problem-dependent. This may be found in the calculated result of Case 2.

(3) Flow around a series of spur dykes with scour holes

In Chapter 2, two experiments with a series of spur dykes have been described detailedly. As a final application, the proposed turbulence model is applied to predict the flow field in those experiments under equilibrium condition. The simulation is based on the final bed configuration resulted from the experimental measurements as shown in Fig. 2.6. Since the study domain is geometrically symmetrical, only the half on the right side of the channel is calculated after employing a symmetrical plane condition at $y=49.5\text{cm}$.

For the impermeable case the total mesh number is 67,250 and for the permeable case it is 68,600. In either case, the mesh is clustered around the spur dykes and becomes coarse far from the spur dykes. A 2D mesh is firstly generated on the x-y plane and then it is extended to 3D space by adding more layers vertically to accommodate the complex flow domain. The mesh structures around the first 5 spur dykes in the upstream projected on a horizontal plane are shown in Fig. 3.24 and Fig. 3.25.

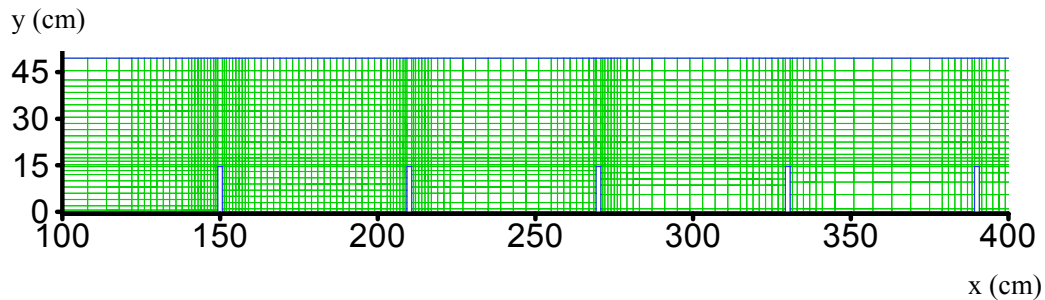


Fig. 3.24 Mesh Structure Projected on the x-y Plane (Impermeable Case)

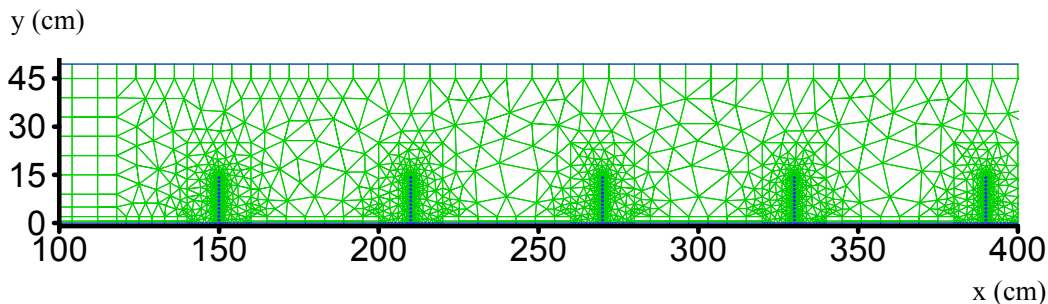


Fig. 3.25 Mesh Structure Projected on the x-y Plane (Permeable Case)

The standard $k - \varepsilon$ model is adopted in this simulation. The principles of the setting of the boundaries at the inlet and the outlet together with the initial conditions are the same as those for the previous verifications. Concentration has been focused on the first several spur dykes in the upstream because this area is of the most interest in the engineering practice as has mentioned in Chapter 2.

(a) Impermeable case

The computed velocity components (u , v , w) at two different layers (i.e. $z=3.25\text{cm}$ and $z=1.0\text{cm}$) are shown from Fig. 3.26 to Fig. 3.31 which are interpolated from the computational results. The measured data at these two layers in the experiment has already been given from Fig. 2.11 to Fig. 2.13 in Chapter 2.

As has mentioned before, the experimental data seems not much enough to express the details of the flow structure. Hence, only an overview of the basic flow characteristics in the whole study domain can be acquired through Fig. 2.11, Fig. 2.12 and Fig. 2.13. However, the numerical model results from Fig. 3.26 to Fig. 3.31 provide very detailed information on the flow structure. This will further the understanding of the local scour mechanism.

The flow structure in the proximity of the most upstream spur dyke is evidently different from that of the other spur dykes in both the upper layer ($z=3.25\text{cm}$) and the lower layer ($z=1.0\text{cm}$). The stream-wise velocity profile is firstly discussed.

After the flow passes the second spur dyke (i.e. the spur dyke immediately following the most upstream one), the flow pattern shows some kind of similarity. In the embayments formed by two consecutive spur dykes, two circulating flows can be observed (Fig. 3.27 and Fig. 3.30). The circulation in the upstream of each embayment is relatively large which has also been resolved by the experimental data. The circulation downstream locates at the corner of the embayment and is much smaller compared with the large one. The small circulation is not obviously presented by the experimental data, but similar circulation has been found in the surface visualization. It is also found that the directions of both circulating flows are clock-wise.

When the flow approaches the most upstream spur dyke, it diverts at the head of the spur dyke. Near the bank, a small eddy with a clock-wise direction forms at both layers. This kind of eddy is also found in the surface flow visualization. In the embayment just behind the most upstream spur dyke, the flow pattern in the two layers is different. In the lower layer, two circulations can be observed in the embayment (Fig. 3.29). In the upper layer, only a small circulating flow forms at the downstream corner area of the embayment. There is no obvious large circulating flow in the upstream of the embayment. The water in the embayment flows in three ways: downstream, upstream and towards the main channel (Fig. 3.26). This phenomenon is a little similar to that in the surface visualization.

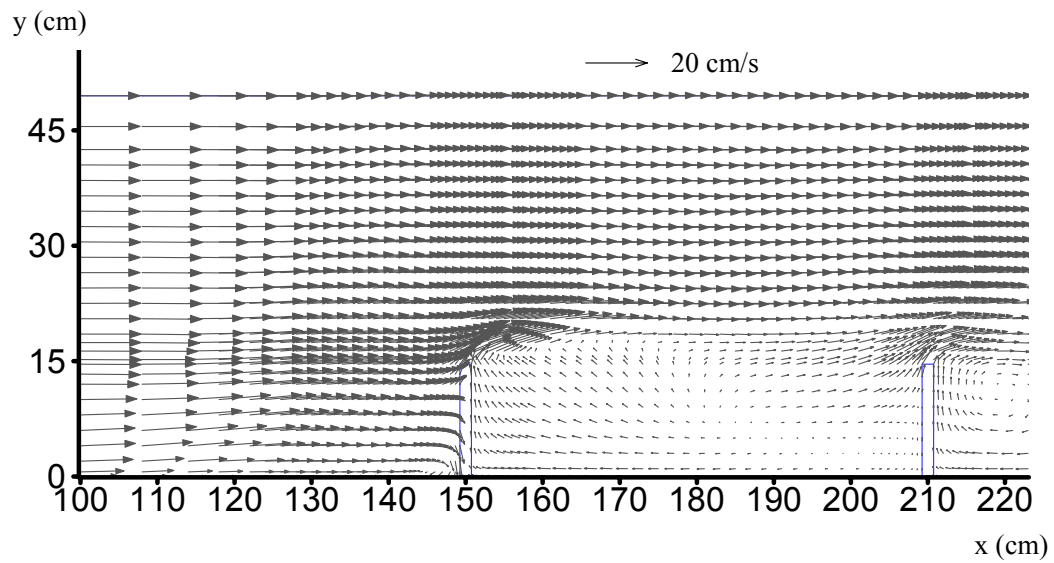


Fig. 3.26 Velocity Vector (u, v) at $z=3.25\text{cm}$ around the First Two Impermeable Spur Dykes

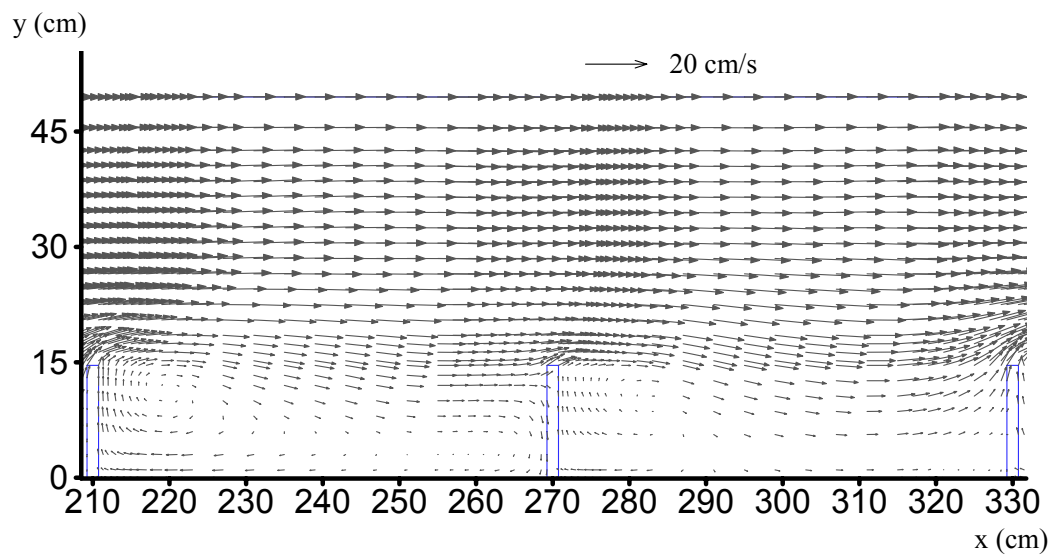


Fig. 3.27 Velocity Vector (u, v) at $z=3.25\text{cm}$ around the Third Impermeable Spur Dyke

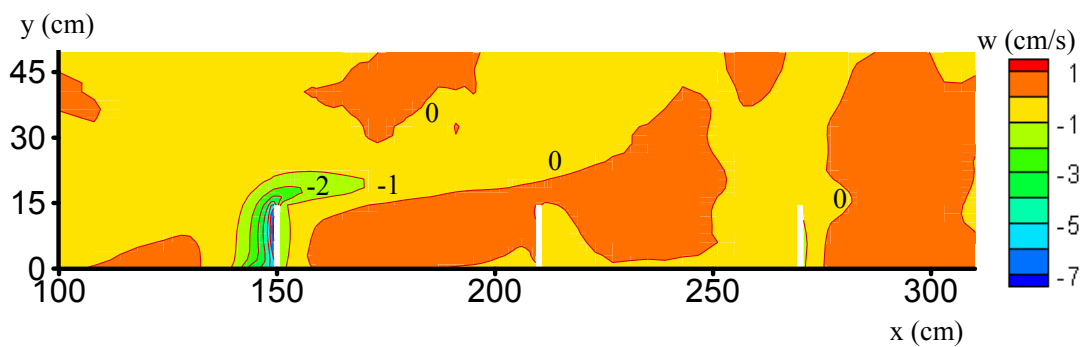


Fig. 3.28 Vertical velocity at $z=3.25\text{cm}$ around the First Three Impermeable Spur Dykes

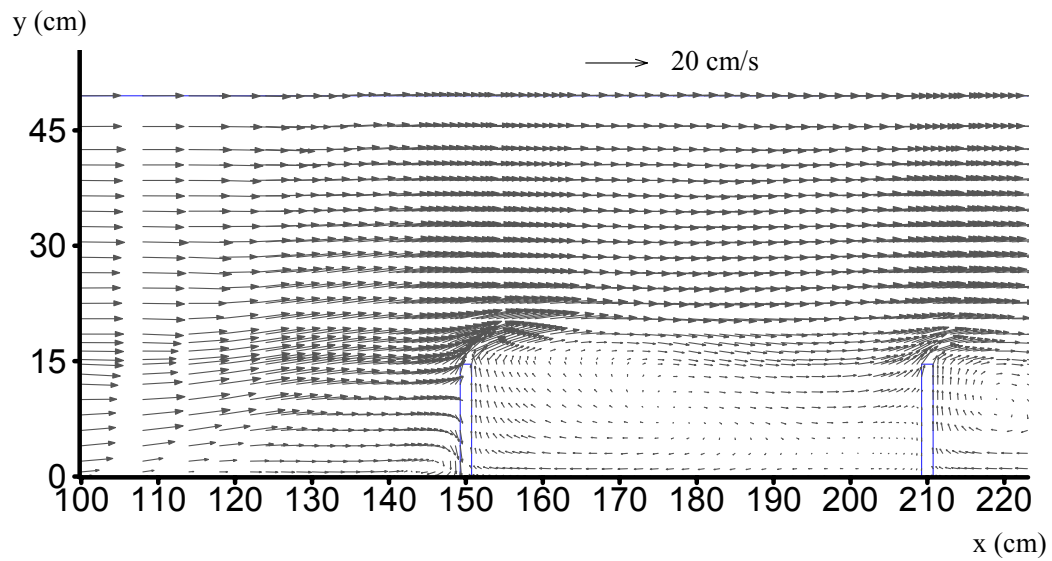


Fig. 3.29 Velocity Vector (u, v) at $z=1.0\text{cm}$ around the First Two Impermeable Spur Dykes

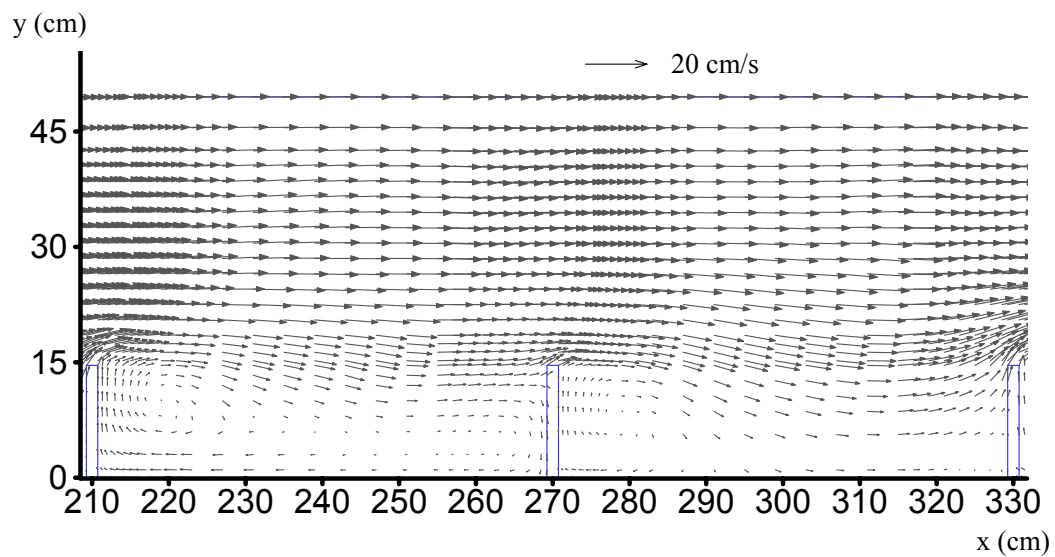


Fig. 3.30 Velocity Vector (u, v) at $z=1.0\text{cm}$ around the Third Impermeable Spur Dyke

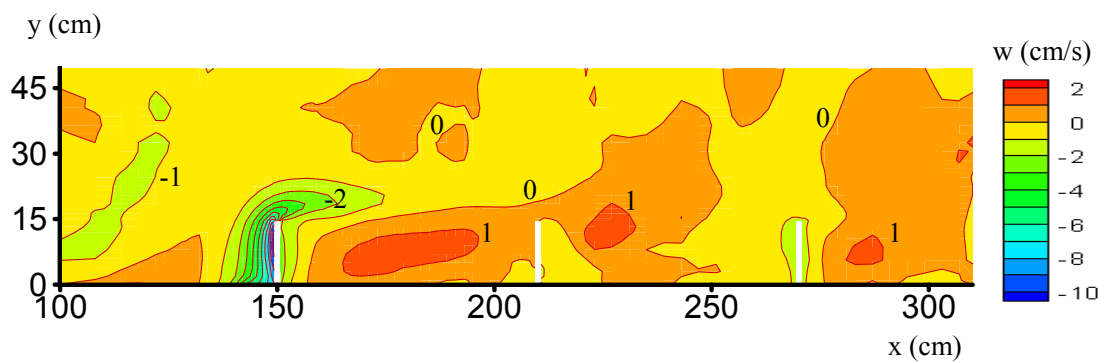


Fig. 3.31 Vertical Velocity at $z=1.0\text{cm}$ around the First Three Impermeable Spur Dykes

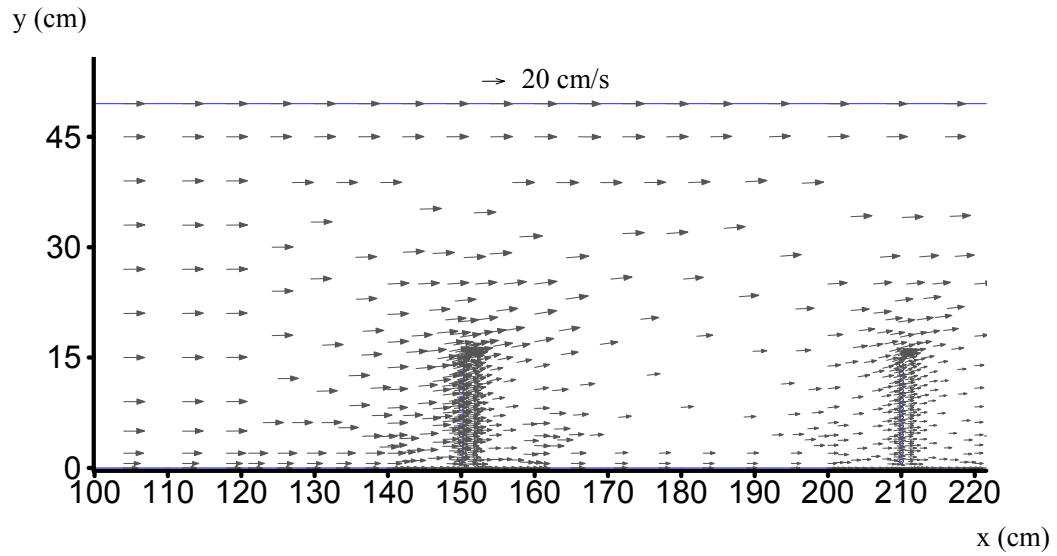


Fig. 3.32 Velocity Vector (u, v) at $z=3.25\text{cm}$ around the First Two Permeable Spur Dyke

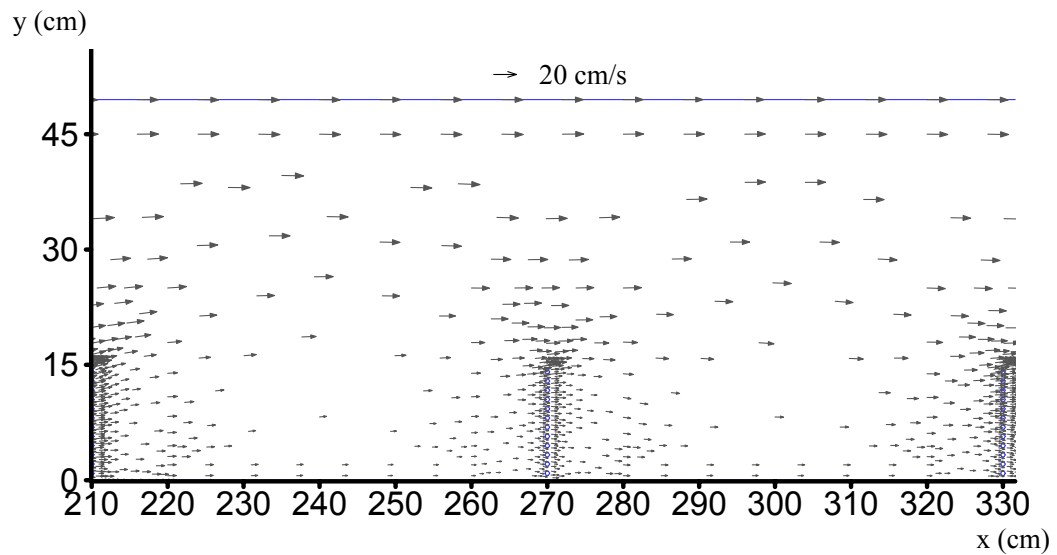


Fig. 3.33 Velocity Vector (u, v) at $z=3.25\text{cm}$ around the Third Permeable Spur Dyke

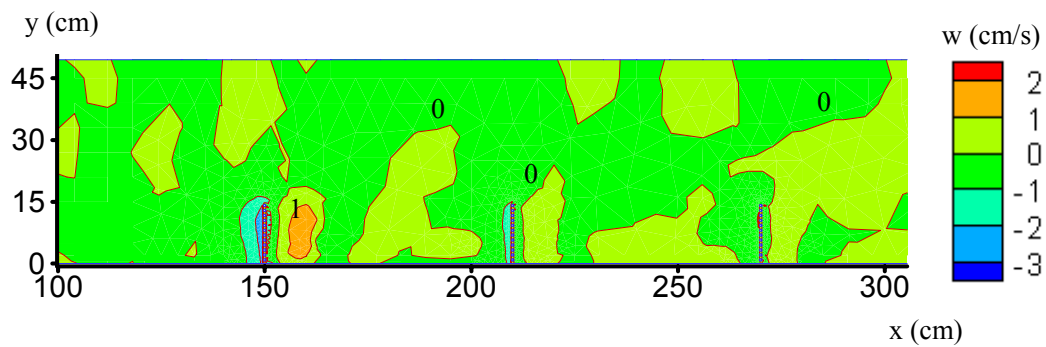


Fig. 3.34 Vertical Velocity at $z=3.25\text{cm}$ around the First Three Permeable Spur Dykes

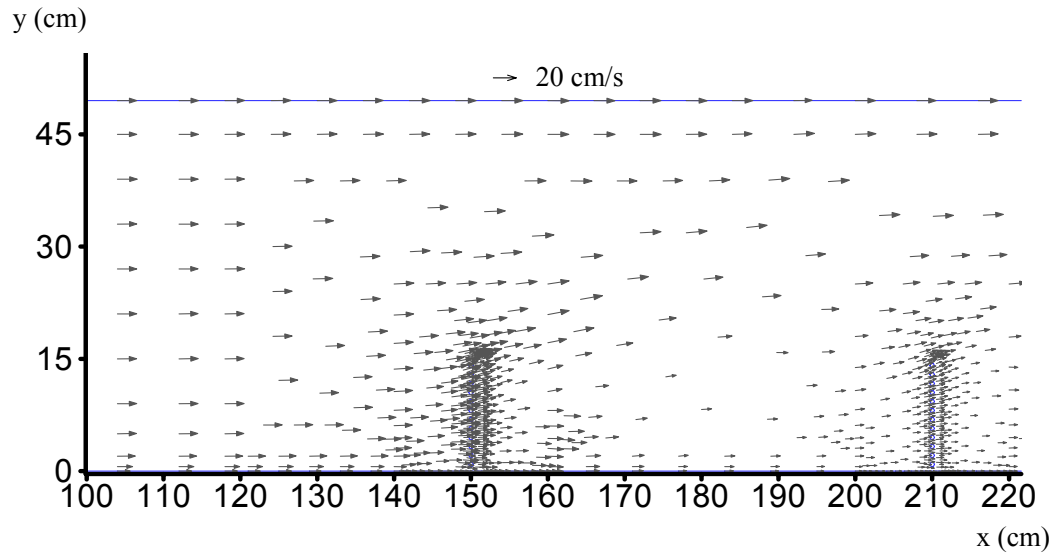


Fig. 3.35 Velocity Vector (u, v) at $z=2.0\text{cm}$ around the First Two Permeable Spur Dyke

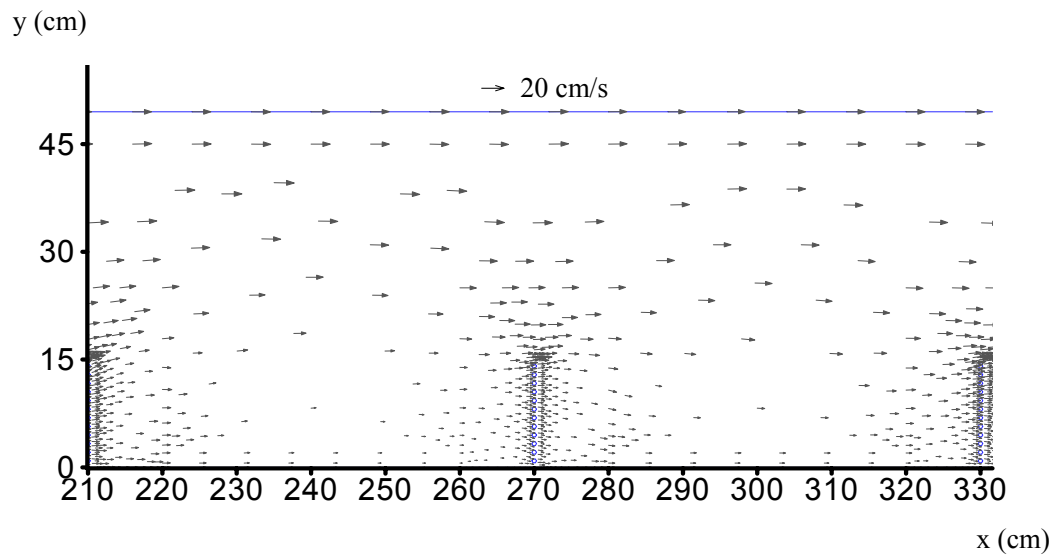


Fig. 3.36 Velocity Vector (u, v) at $z=2.0\text{cm}$ around the Third Permeable Spur Dyke

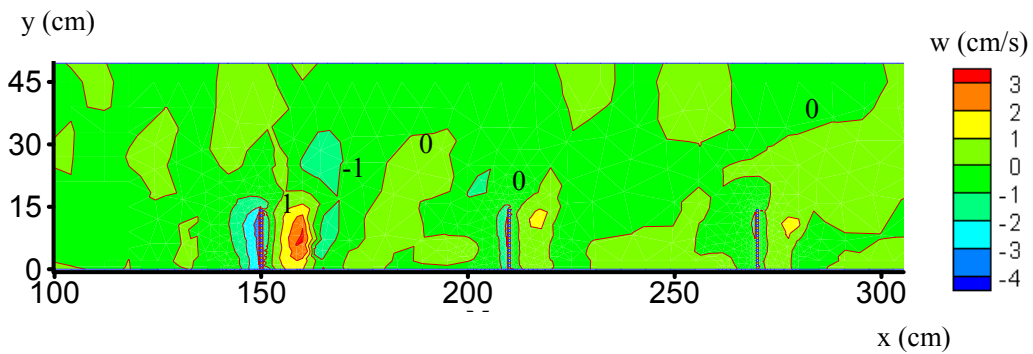


Fig. 3.37 Vertical Velocity at $z=2.0\text{cm}$ around the First Three Permeable Spur Dykes

If one takes a look at the vertical velocity distribution in Fig. 3.28 and Fig. 3.31, one can find that the vertical velocity in the vicinity of the most upstream spur dyke is quite large and this velocity increases from the upper layer to the lower layer. This is related to the severest local scour around this spur dyke. The bed form in this area is very complex and the flow structure is strongly three-dimensional.

(b) Permeable case

The computed velocity components (u , v , w) at two different layers (i.e. $z=3.25\text{cm}$ and $z=2.0\text{cm}$) are shown from Fig. 3.32 to Fig. 3.37 which are also interpolated from the computational results. The measured data at these two layers in the experiment has been given from Fig. 2.18 to Fig. 2.21 in Chapter 2.

Compared with the impermeable case, the flow structure is less complex. The flow may be considered as a 2D flow. There is no obvious circulating flow and the longitudinal velocity is predominant through the whole domain. The flow pattern in the two different layers is quite similar although there are some differences in the magnitude.

The longitudinal flow velocity is gradually reduced in the embayment from the upstream to the downstream. In the embayment just behind the most upstream spur dyke, the reduction amount is not very large. However, a significant velocity reduction is observed after the flow passes the second spur dyke. This coincides with the experimental observations.

This application further verified the capability of the proposed methodology. The standard $k-\varepsilon$ model is also confirmed to be valid in much sophisticated flow and bed conditions.

3.4 Summary

In this chapter, the flow module in the morphological model is presented. The main contents are summarized as below.

From the viewpoint of practicability, different levels of modeling is very important. At the beginning of this chapter, models of different levels are briefly introduced, together with their advantages and defects. A 1D model and a 3D model have been developed in this study. The 1D model is used for large scale and long-term prediction of river basins and the 3D model is employed for local resolution. Since the 1D flow model has been well developed, only the basic ideas have been given. The unstructured mesh based 3D model is emphasized in this chapter, which is considered to be a breakthrough and of significant value for both engineering use and scientific research.

The current 3D model solves the RANS with the standard and some non-linear $k - \varepsilon$ models for the turbulence closure. The wall function approach is adopted in the near wall area to bridge the turbulence area and the non-slip impermeable wall. A brief overview of the turbulence modeling and meshing strategy is firstly presented, together with previous efforts made related to spur dykes induced flow phenomena. After that, the governing equations of the turbulence models are listed. A standard $k - \varepsilon$ model is the most commonly used turbulence models in the engineering practices. But the defects of this model are also well-known, e.g. the isotropic eddy viscosity. Some of the defects may be cured by introducing some non-linear terms, which constitute the non-linear $k - \varepsilon$ models.

The governing equations are integrated over a series of CVs covering the study domain with an FVM procedure. These CVs are organized in an unstructured manner. As a result, the connectivity between each CV and its neighbors has to be explicitly defined, which is quite different from a structured mesh and may lead to a tremendous consumption of computer memories. Since polyhedral mesh more than six faces is seldom used in the engineering practice, a hybrid mesh may be considered as a hexahedral mesh provided that the polyhedra with less than six faces are assigned some nominal faces. The final data structure for the mesh system is a list of hexahedral CVs, a list of CV vertices in an anti-clockwise order and a list of CV neighbors in a prescribed order. Moreover, all the quantities are defined at the centers of the CVs, which is also called a collocated mesh.

With the aforementioned mesh structure, the integrated governing equations are discretized into a series of algebraic equations. The power law scheme is adopted for the spatial discretization and the 2nd order implicit Crank-Nicolson scheme is employed for the temporal integral. In order to avoid the checkerboard phenomenon which generally occurs in a collocated mesh, the surface fluxes are evaluated with the Rhie-Chow interpolation. The mesh skewness which is almost unavoidable in an unstructured mesh method is cured by classifying the skewness into non-conjunctionality and non-orthogonality. Corrections are made according to the different type of mesh skewness. As the continuity equation does not include the pressure information explicitly, but provides constraints for the velocity field, the SIMPLE procedure is used to couple the pressure and velocity during the calculation. The movement of the mesh system is expressed by introducing a mesh movement velocity into the convective terms in the governing equations. The evaluation of the mesh movement should assure that no artificial mass sources are introduced. In this thesis, the mesh is limited to move only in the vertical direction. This has greatly simplified the treatment of mesh movement.

With the above treatments, the governing equations are fully discretized. Boundary conditions are then introduced including the inlet boundary, the outlet boundary and the impermeable wall. The free surface is considered as a symmetrical plane for the time being. And as mentioned

before, the wall function approach is used for the impermeable wall, which can include the bed roughness flexibly and is thus attractive for sediment related flow phenomena.

The final equation system is under-relaxed and then submitted to the equation solver. A Bi-CGSTAB solver and a preconditioned GMRES solver incorporated with an ILUTP preconditioner are integrated in the program.

The proposed turbulence models are applied to several flows in laboratory flumes: the flow in a rectangular straight channel, the flow in a square embayment, the flow in a rectangular embayment and the flow around a series of impermeable and permeable spur dykes. In the verification, different models and different meshing strategies have been tested. Such conclusions may be drawn.

- (a) The proposed unstructured mesh methods are applicable in fixed bed conditions. The computational result is comparable with structured mesh methods in simple domains. In complex geometries, implement of structured mesh methods is generally difficult if not impossible. As flows of engineering interest usually involve complex geometries, the current unstructured mesh methods are more promising for industrial use.
- (b) The standard $k-\varepsilon$ model is not able to reproduce the turbulence-driven secondary flows. If the turbulence-driven secondary flows are very important, a more elaborate model, say, a non-linear model may give a better result.
- (c) The standard $k-\varepsilon$ model is found to be more stable than the non-linear one and is quicker to get converged. And the computational result is acceptable and good enough from the perspective of engineering uses. If the subroutine of the turbulence model will be repeatedly called by the main program which is the case of the morphological model in this study, the standard $k-\varepsilon$ model seems to be very effective. Another important consideration is the intensity of the two kinds of secondary currents. According to Nezu (2002), the magnitude of the 1st kind of secondary currents is about 10% of that of the primary flow in natural channels, while the 2nd kind is around 1%-2%. These do mean that the 1st kind secondary currents play a more important role than the 2nd kind for sediment transport in actual river conditions. This also suggests that a use of $k-\varepsilon$ model is generally cost-effective nowadays.

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Chapter 4

Sediment transport models

4.1 Introduction

Compared with the turbulent flow, the sediment transport involves much more uncertainties. As a result, almost all of the published sediment transport formulae are not accomplished following purely theoretical pursuit (Chang, 1988). When the value of the bed shear stress just exceeds the critical value for the initiation of sediment motion, the particles on the riverbed will roll and/or slide in continuous contact with the bed. If the bed shear stress becomes larger, some particles will move along the bed by more or less regular jumps, which are usually termed saltations. If the bed shear stress is so large that the particles can be lifted to a level where the magnitude of the upward turbulent forces is comparable to that of the submerged weight of the sediment, these particles may be carried by water and move in suspension.

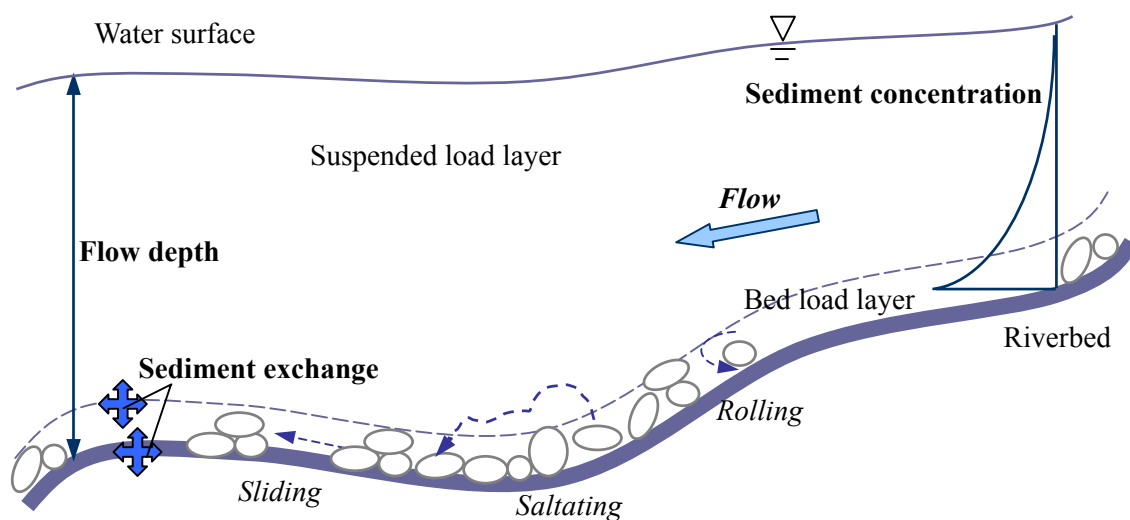


Fig. 4.1 Sediment Transport in Channels

The transport of particles by rolling, sliding and saltating along the bed is called bed load, while the sediment suspended in the water column is termed suspended load. Although there is

no sharp division between the suspended load and bed load physically, a bed load layer is assumed to be existent for the mathematical representation. As shown in Fig. 4.1, the flow domain is subdivided into two layers vertically. Bed load is confined to move in a thin layer in the proximity of the riverbed, above which is the region occupied by suspended load. The exchange of sediment between the two layers is through the upward and downward fluxes at the interface. These fluxes are termed *near-bed fluxes* hereafter.

The bed load transport can be modeled by empirical formulae because the adjustment of the transport process to the flow conditions proceeds rapidly. There are a lot of alternatives, e.g. the Meyer-Peter and Muller formula (1948), the Ashida and Michiue formula (1972a) and the van Rijn formula (1984a). Due to the complexity of the sediment transport processes, all existing formulae have been established relying on calibration using limited flume and field data. Hence every formula has limitations and should be used prudently. For example, the Meyer-Peter and Muller formula is developed based on experiments made in laboratory flumes with widths ranging between 15cm and 2m, water depth between 1 and 120cm, effective sediment size between 6.4 and 30mm and specific weight of particles from 1.25 to over 4. It is evident that this formula is more applicable for coarse sediment. The transport of suspended load takes time and space and should be simulated by considering the convective and diffusive processes. This means that one may have to solve an equation system in order to obtain the concentration of suspended load. Furthermore, the alluvial rivers generally transport a wide spectrum of sediment sizes. The behavior of mixed sediment is quite different from that of uniform one. For instance, the erosion of particles with a certain size from the riverbed will depend on not only the flow conditions but also the amount of that size available in the bed surface. In the numerical simulation, a mixed sediment is usually simplified to a series of size fractions and each fraction has a representative size. Algorithms are then constructed to evaluate the transport of a series of size fractions.

When the channel is equipped with spur dykes either for river training or for river restoration, the balance of the sediment transport will be broken. As has mentioned in Chapter 1, special attention should be paid to the local scour and fine sediment transport. In the local scour hole area, the bed slope is very steep. This will affect the bed load transport rate very much. The conventional bed load transport formula for a horizontal bed or a mild bed is not valid without suitable modifications. Spur dykes create a wide spectrum of turbulences, which brings new problems on the fine sediment transport. Fine sediment consists of cohesive particles and cannot be predicted with the classical methods for suspended load. Moreover, as fine sediment exchanges with the local bed, the riverbed may show some characteristic of cohesiveness. The bed cohesiveness tends to affect the erosion of both non-cohesive and cohesive particles from the riverbed.

In this study, the local scour is simulated by solving the mass conservation equation of the sediment. The bed load transport rate is evaluated with the Ashida-Michiue formula. Taking the local bed slope into account, the formula has been extended from a horizontal bed to a sloping bed. Suspended load is subdivided into two classes according to the different motion mechanisms. The coarse part of suspended load, dislodged mainly by particle gravity and moving individually, is termed *coarse sediment* hereafter. Coarse sediment consists of non-cohesive particles and coincides with the conventionally defined suspended load. Consequently, traditional methods for suspended load may be used to evaluate the transport of coarse sediment. The fine part of suspended load, tending to aggregate and forming larger units of flocs due to cohesiveness, is called *fine sediment* as has already mentioned in Chapter 1. The transport of fine sediment has to be estimated from the perspective of cohesive sediment transport. In the water column, the interaction between fine sediment and coarse sediment is assumed to be negligible if the sediment concentration is relatively low. However the mixture of coarse sediment and fine sediment on the riverbed has a potential to change the bed behavior. This phenomenon should not be neglected. It is included in the study by constructing a new bed concept model.

This chapter is organized as follows. In the next section (i.e. Section 4.2), methods for the bed modeling are presented, which include the bed morphology, the bed sorting process, the bed cohesiveness and the bed slope. After that, the transport of suspended load is described in Section 4.3. The governing equations and the boundary conditions are emphasized. The differences between coarse sediment and fine sediment are also discussed in detail. In Section 4.4, the transport of bed load is discussed. And finally, a brief summary is given in Section 4.5.

4.2 Bed modeling

4.2.1 Bed morphology

If the transport rate of bed load and suspended load is known, the bed morphology may be simulated by applying the mass-balance equation to the bed load layer. The transport of bed load is explicitly included in the equation. And suspended load participates the bed shaping process through the interchange with the bed load layer, which is interpreted as near-bed fluxes in the equation. For a size fraction k during a mixed sediment transport, this results in

$$(1 - \lambda) \frac{\partial z_{bk}}{\partial t} + \frac{\partial q_{bkx}}{\partial x} + \frac{\partial q_{bky}}{\partial y} + (E_k - D_k) = 0 \quad (4.1)$$

where λ = sediment porosity; q_{bkx}, q_{bky} = bed load transport rate for size fraction k in x and y

direction, respectively, E_k = the upward near-bed flux for size fraction k and D_k = the downward near-bed flux for size fraction k . Solution of Eq. 4.1 leads to the bed change due to each size fraction at one time step. The summation of the bed changes due to all the size fractions is the resulted bed variation, i.e.

$$\delta z_b = \sum_{k=1}^M \delta z_{bk} \quad (4.2)$$

where δz_b = the bed variation at one time step δt ; δz_{bk} = the bed change due to size fraction k at time step Δt and M = the total size fractions.

4.2.2 Bed sorting

The selective transport of sediment generally occurs whenever the bed material consists of a mixture of particles. Some particles may deposit on the riverbed while some may be scoured away, which will result in a sediment size distribution in transport different from that of the bed. A realistic model must account for this phenomenon. Predictive methods keeping account of bed-sediment composition while tracking bed-profile changes have been attempted by a number of researchers (Chang, 1988). These researches may be categorized into four kinds:

(a) Mixed layer based approach (e.g. Hirano, 1971 and Lee and Odgaard, 1986)

The riverbed is vertically divided into a mixed layer and a substrate layer. The mixed layer locates at the upper part of the bed with a certain thickness. It is assumed that all particles contained in the mixed layer are homogeneous and equally exposed to the flow irrespective of their location in the layer. The sediment exchange between the mixed layer and the water column depends on the transport of both bed load and suspended load, while the sediment exchange between the mixed layer and the substrate layer only depends on the change of the bed level. This approach only keeps track of the composition in the mixed layer, whereas the substrate layer remains unchanged. Generally, the size distribution in the substrate layer varies with the bed variation as well. This leads to the method of the second kind.

(b) Multi-layer approach (e.g. Borah et al., 1982 and Liu, 1991)

This method memorizes the bed composition for more than one layer and is physically more realistic. In this chapter, one of this kind of methods will be introduced. Both the mixed layer based methods and the multi-layer methods are based on the concept of the mixed layer, which is considered to be related to the height of the bed forms. The sediment in actual riverbed is of course seldom distributed discontinuously. Furthermore, the evaluation of the thickness of the mixed layer is also a big problem. These disadvantages result in other new researches.

(c) Continuous concept modeling (e.g. Armanini, 1995)

Armanini argued in his paper that the bed composition was a continuous function of the vertical coordinate within the bed. From this viewpoint, he proposed a continuous approach. This method may be considered as a generalized multi-layer approach. Armanini showed two important differences between the mixed layer concept proposed by Hirano and the continuous approach. Firstly, the diffusive flux between the mixed layer and the substrate layer was not included in the mixed layer approach, but this diffusive flux might be physically explained by the existence of the short-term bed fluctuations. Secondly, a constant bed composition over a certain thickness has been assumed by Hirano, whereas the bed composition could vary at infinitely small distance from the bed surface in his new continuous model.

(d) Probabilistic method (e.g. Parker et al., 2000)

Parker et al. proposed a conceptual framework for the modeling of bed sorting. They argued that the division of the erodible bed into a series of discrete layers represented only an approximation of a more general formulation in which parameters pertaining to the entrainment from and deposition to the bed varied continuously with depth below the sediment-water interface. Different from the aforementioned continuous concept modeling, the bed elevation and sediment movement were expressed by probability density functions instead of deterministic functions. However, the authors did not implement the proposed models because some of the internal relations were still under development.

One may distinguish the winner from the above four kinds of methods theoretically, but it is quite difficult to declare which one is the best in applications. The applicability of these approaches strongly relies on the insight into the sediment exchange processes. In this study, the multi-layer method proposed by Liu (1991) is adopted.

The natural bed is discretized vertically into a series of layers above a datum level z_0 as shown in Fig. 4.2. The upper layer is an active layer, the top of which is the bed surface. The sediment contained in the active layer is the only material available for erosion during a calculation time step. A transition layer is situated below the active layer, which is followed by a sequence of deposited layers. The deposited layers function as a reserve for the sediment particles. The distribution of the sediment size in different layers may vary with the flow and bed evolution. But each layer is assumed to be homogeneous within itself at any given time. The active layer changes its elevation and sediment size distribution with the transport of sediment load but is kept a constant thickness. The variation of the bed morphology is expressed by the changing of the thickness of the transition layer and the total number of the deposited layers. For example, in case of erosion, some sediment with the composition of the transition layer is transferred to the active layer; and in case of deposition, some sediment with the composition of the active layer will be transported to the transition layer.

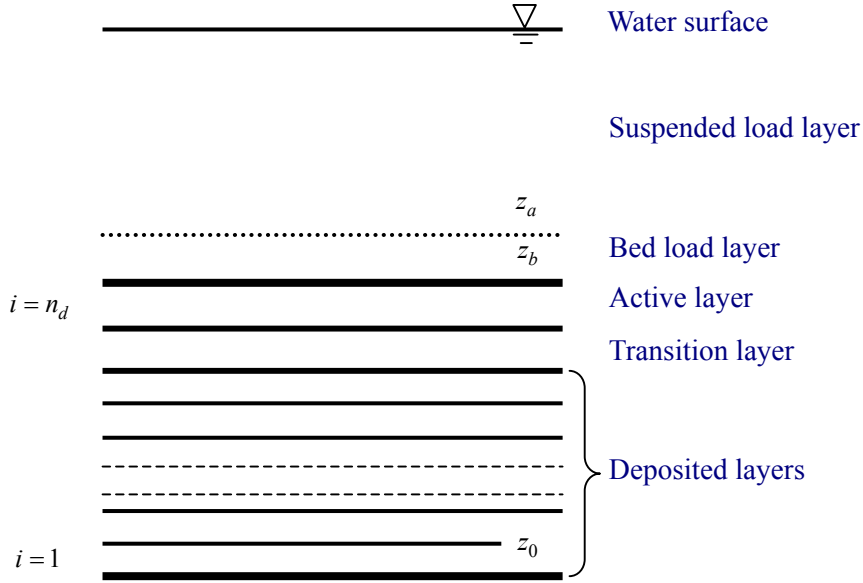


Fig. 4.2 Definition Sketch of a Vertical Profile

According to Fig. 4.2, the bed level may be expressed by

$$z_b = z_0 + t_a + t_t + \sum_{i=1}^{n_d} t_{di} \quad (4.3)$$

where t_a = the thickness of the active layer; t_t = the thickness of the transition layer; t_{di} = the thickness of the i th deposited layer and n_d = the total number of the deposited layers.

The thickness of the active layer is a little difficult to determine because it is related to both the flow and the sediment conditions. For simplicity, Liu suggested that the maximum size of the sediment fractions could be used provided that the time step should be carefully selected to assure a relative small value for $\delta z_b / t_a$. This has been confirmed to work well in practice. The thickness of the deposited layers may be prescribed according to the erodible riverbed depth and the computer memory. The thickness for each deposited layer is set to be the same. And the initial thickness of the transition layer can be set to the same as that of the deposited layer. During the sediment transport, the thickness of the transition layer should not be larger than that of the deposited layer, i.e.

$$t_t \leq t_d \quad (4.4)$$

With the above arguments, the bed sorting process is able to be modeled in an easy way. The algorithm and the equations are constructed below.

From a time step m to a new time step $m+1$, the ratio of each size fraction in different layers, the thickness of the transition layer and the total number of the deposited layers will be

updated. Assuming that the riverbed elevation has a change δz_b from time step m to time step $m+1$, one can obtain the following expressions (also see Fig. 4.3 and Fig. 4.4).

(1) Bed aggradation $\delta z_b \geq 0$

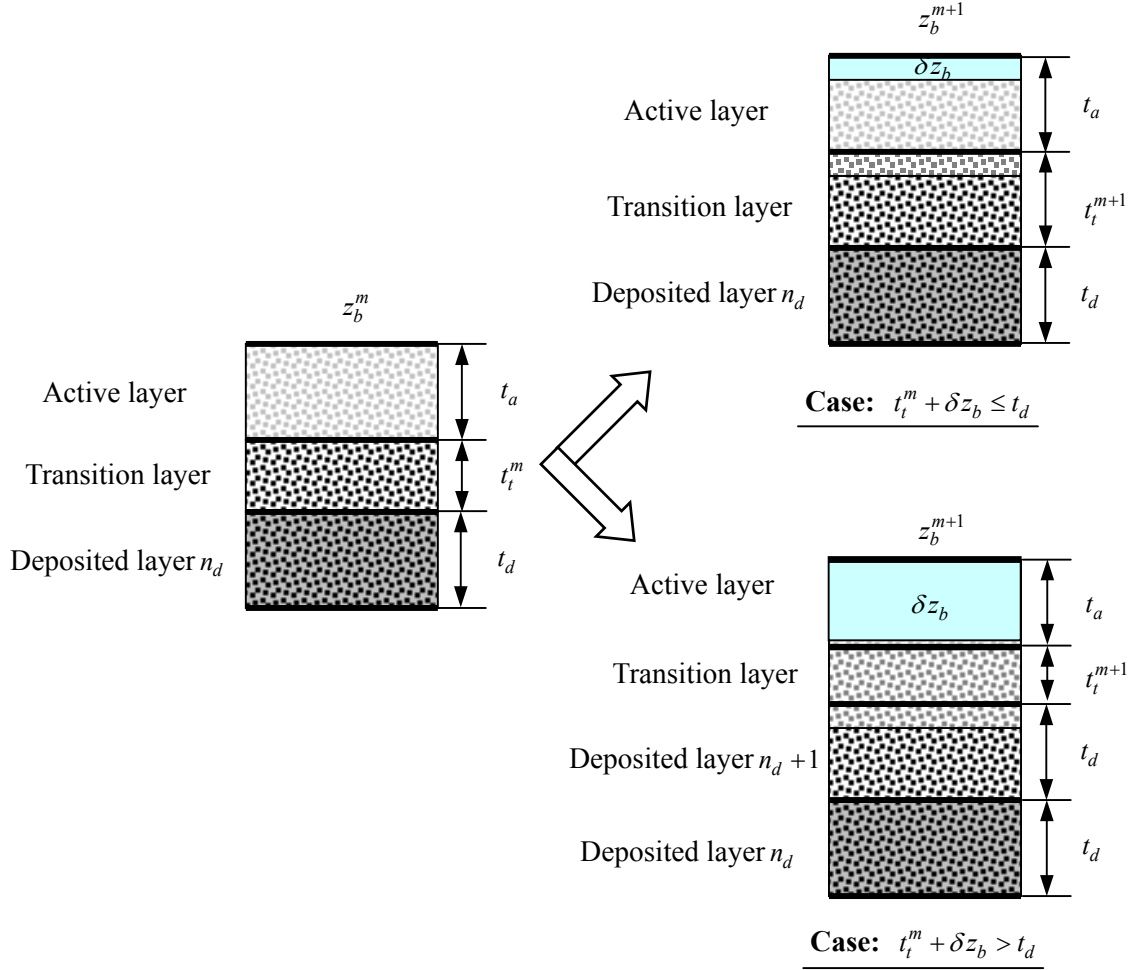


Fig. 4.3 Sorting Process during Bed Aggradation

$$p_{bk}^{m+1} = \left(1 - \frac{\delta z_b}{t_a}\right) p_{bk}^m + \frac{\delta z_{bk}}{t_a} \quad (4.5)$$

$$t_t^{m+1} = \begin{cases} t_t^m + \delta z_b & \text{if } t_t^m + \delta z_b \leq t_d \\ t_t^m + \delta z_b - t_d & \text{if } t_t^m + \delta z_b > t_d \end{cases} \quad (4.6)$$

$$p_{tk}^{m+1} = \begin{cases} \frac{t_t^m}{t_t^{m+1}} p_{tk}^m + \frac{\delta z_b}{t_t^{m+1}} p_{bk}^m & \text{if } t_t^m + \delta z_b \leq t_d \\ p_{bk}^m & \text{if } t_t^m + \delta z_b > t_d \end{cases} \quad (4.7)$$

$$n_d^{m+1} = \begin{cases} n_d^m & \text{if } t_t^m + \delta z_b \leq t_d \\ n_d^m + 1 & \text{if } t_t^m + \delta z_b > t_d \end{cases} \quad (4.8)$$

$$p_{n_d k}^{m+1} = \begin{cases} p_{n_d k}^m & \text{if } t_t^m + \delta z_b \leq t_d \\ \frac{t_t^m}{t_d} p_{tk}^m + \left(1 - \frac{t_t^m}{t_d}\right) p_{bk}^m & \text{if } t_t^m + \delta z_b > t_d \end{cases} \quad (4.9)$$

where the superscripts m and $m+1$ stand for the consecutive time steps; subscript a = the active layer; subscript t = the transition layer; subscript d = the deposited layer; subscript k = the k th size fraction; subscript b = the riverbed; p = the percentage of one size fraction in the riverbed; t = the thickness of one layer. It is noted that the percentage of each size fraction in the active layer is considered to be that of the riverbed.

(2) Bed degradation $\delta z_b < 0$

$$p_{bk}^{m+1} = \begin{cases} p_{bk}^m + \frac{t_t^m}{t_a} p_{tk}^m - \frac{t_t^m + \delta z_b}{t_a} p_{n_d k}^m + \frac{\delta z_{bk}}{t_a} & \text{if } t_t^m + \delta z_b \leq 0 \\ p_{bk}^m - \frac{\delta z_b}{t_a} p_{tk}^m + \frac{\delta z_{bk}}{t_a} & \text{if } t_t^m + \delta z_b > 0 \end{cases} \quad (4.10)$$

$$t_t^{m+1} = \begin{cases} t_t^m + t_d + \delta z_b & \text{if } t_t^m + \delta z_b \leq 0 \\ t_t^m + \delta z_b & \text{if } t_t^m + \delta z_b > 0 \end{cases} \quad (4.11)$$

$$p_{tk}^{m+1} = \begin{cases} p_{n_d k}^m & \text{if } t_t^m + \delta z_b \leq 0 \\ p_{tk}^m & \text{if } t_t^m + \delta z_b > 0 \end{cases} \quad (4.12)$$

$$n_d^{m+1} = \begin{cases} n_d^m - 1 & \text{if } t_t^m + \delta z_b \leq 0 \\ n_d^m & \text{if } t_t^m + \delta z_b > 0 \end{cases} \quad (4.13)$$

$$p_{n_d k}^{m+1} = \begin{cases} p_{(n_d-1)k}^m & \text{if } t_t^m + \delta z_b \leq 0 \\ p_{n_d k}^m & \text{if } t_t^m + \delta z_b > 0 \end{cases} \quad (4.14)$$

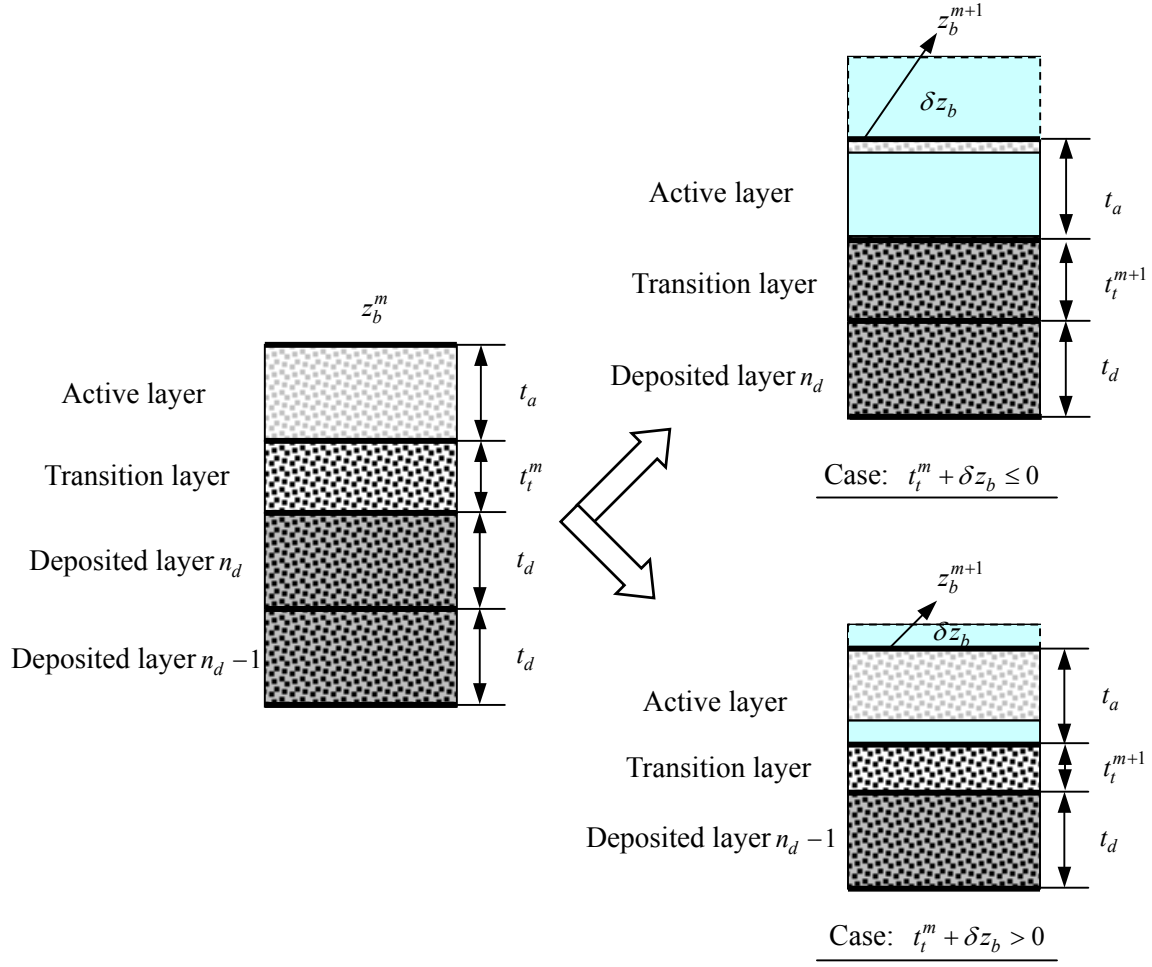


Fig. 4.4 Sorting Process during Bed Degradation

For some reason such as the improvement of navigation conditions, sediment may be dredged up from the riverbed. The dredging amount is turned to an equivalent thickness according to the shape of the cross-section and considered as an erosion process in this study. The bed sorting process then can be evaluated with the aforementioned methods.

4.2.3 Bed cohesiveness

During the bed sorting process, the bed composition changes with time. As has mentioned before, the construction of spur dykes creates a wide spectrum of turbulences, which makes it possible that very fine sediment will exchange with the local bed. If the fine sediment has a significant amount in the bed composition, the bed shows some characteristic of cohesiveness. A non-cohesive bed has a granular structure and the particle size and its weight are the dominant parameters for erosion. A cohesive bed may form a coherent mass because of the

electrochemical interaction between the particles or the organic materials which function like glues. The cohesive bed exhibits some kind of shear strength, which may prevent the sediment on the riverbed from erosion. Although literatures on both the non-cohesive and the cohesive sediment are rapidly expanding these days, researches on the mixture of coarse and fine sediment are still very few especially in the river engineering. This study attempts to do some preliminary work to bridge the gap between the two fields.

A detailed estimation of the bed cohesiveness relies on a clear insight into the bed mineralogy, inter-particle cohesiveness and the complex geochemistry process, microbiological process and consolidation process on the bed. This seems to be too sophisticated to complete with the science and technology of the state-of-the-art. There is no published paper that includes all those processes explicitly in the investigation to the best of the author's knowledge. In order to evaluate the influence of the bed cohesiveness on the erosion process, a bed concept model is introduced here to simplify the study. The riverbed is considered to consist of a non-cohesive particle cluster surrounded by a cohesive force field. As a result, the erosion from the riverbed is a two-stage process. Firstly, the particles are removed from the non-cohesive bed. After that, the particles will pass through the cohesive force field. As a result, the erosion rate of sediment is determined by both the erodibility from the non-cohesive bed and the possibility of passing through the cohesive force field. Many non-cohesive based formulae are available for the evaluation of the former amount. But the latter one is very difficult. Fortunately, the sophisticated micro-scale cohesive force analyses are usually not so much important for river engineering. One generally cares more about the macro-scale effect of the bed cohesiveness. In view of these, the experimental method may be an alternative to determine the possibility of sediment passing through the cohesive force field.

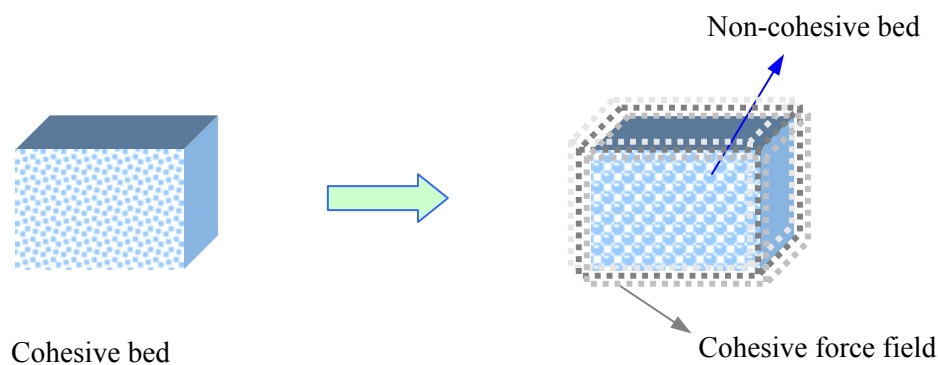


Fig. 4.5 A Bed Model to Account for the Bed Cohesiveness

Erosion experiments related to the bed cohesiveness have been carried out by several researchers (van Ledden, 2002 and van Ledden, 2003). These experiments generally concentrate

on the influence due to the mineralogy of the bed. According to their results, the clay content in the bed surface is one of the most dominant factors which affect the erosion process in a bed. A non-cohesive bed will behave cohesively if the clay content is large enough. There seems to exist a critical clay content, which may serve as an indicator to distinguish a cohesive bed from a non-cohesive one. Dyer (1986) stated that a clay content of 5%-10% is sufficient for a natural bed to have cohesive properties. This agrees with some measurements. Table 4.1 summarizes some measurement results.

**Table 4.1 Critical Clay Content in Some Measurements
(van Ledden, 2002 and van Ledden, 2003)**

Experiment	Clay Type	Critical Clay Content
Alvarez-Hernandez, 1990	Laponite	5-15%
Torfs, 1995	Kaonilite	3%
Torfs, 1995	Montmorillonite	4%
Panagiotopoulos et al., 1997	Combwich	10.8%
Houwing, 2000 (Field measurement)	Natural clay (Wadden Sea, the Netherlands)	5-7%

The discovery of the critical clay content provides an important step for the evaluation of the possibility of sediment overcoming the cohesive force field. If the clay content in the bed surface is not larger than the critical clay content, the strength of the cohesive force field is so small that particles can go into the water freely. This means that the possibility is 1. With the increasing of the clay content, this possibility will decrease. For the time being, one can assume that the possibility is a linear function of the clay content. This leads to

$$f_b = \begin{cases} 1 & \text{if } p_{clay} < p_c \\ \frac{1 - p_{clay}}{1 - p_c} & \text{if } p_{clay} \geq p_c \end{cases} \quad (4.15)$$

where f_b = the possibility of sediment particles passing through the cohesive force field; p_{clay} = the clay content in the bed surface and p_c = the critical clay content in the bed surface.

This possibility will be introduced to decrease the erosion amount of non-cohesive fractions due to the bed cohesiveness. However, it does not affect the estimated erosion rate of cohesive fractions. This is due to the fact that the erosion formulae for cohesive sediment are mostly based on experiments with cohesive bed. It is thus believed that the bed cohesiveness has already included in the empirical formulae. With these arguments, it is also easy to understand why the possibility in Eq. 4.15 will be zero if the riverbed is entirely a clay bed ($p_{clay} = 1$).

4.2.4 Bed slope

The bed slope is a great consideration whenever the local scour happens. It appears that the bed slope will affect the bed load transport in three ways: (a) the bed slope will affect the threshold condition, i.e. the condition just sufficient to initiate sediment motion; (b) the bed slope will affect the effective shear stress acting on the particle and (c) the bed slope will have an influence on the direction of the sediment movement. Of course, the bed slope will affect the bed shear stress as well, but this has been considered in the hydrodynamic models from which the bed shear stress is directly obtained.

The investigation of the bed slope effect received a great attention and achieved significant development in the past century. Many researchers divided the riverbeds into longitudinal beds and transverse beds and developed a vast number of formulae. These formulae are mostly derived from 1D or 2D analysis. A brief view was given by Dey (2003).

The difference of the critical shear stress between a horizontal bed and a sloping bed is generally expressed by a bed slope factor K , i.e.

$$\tau_c = K \tau_{c0} \quad (4.16)$$

where τ_c, τ_{c0} = the critical shear stress for a sloping bed and a horizontal bed, respectively. A typical longitudinal bed slope factor is

$$K_l = \frac{\sin(\varphi \pm \theta_l)}{\sin \varphi} \quad (4.17)$$

where K_l = the longitudinal bed slope factor; φ = the angle of sediment repose and θ_l = the longitudinal bed slope. The + for an up-sloping bed while the - for a down-sloping bed. This equation or its other forms has been used by various researchers (e.g. Luque and van Beek, 1976; van Rijn, 1993; Duan et al., 2001; Nakagawa et al., 2004). Correspondingly, the typical transverse bed slope factor has the form as

$$K_t = \cos \theta_t \sqrt{1 - \frac{\tan^2 \theta_t}{\tan^2 \varphi}} \quad (4.18)$$

where K_t = the transverse bed slope factor and θ_t = the transverse bed slope. This equation also achieved significant applications (e.g. Ikeda, 1982a; Chang, 1988; van Rijn, 1993; Duan et al., 2001; Nakagawa et al., 2004).

Furthermore, the transport rate of sediment in the transverse direction is commonly evaluated based on the transport rate along the longitudinal direction. For example, Parker proposed the following equation based on the work of Ikeda (Parker, 1984 and Ikeda, 1982b)

$$q_{bt} = q_{bl} \left(\tan \delta - \frac{1 + (C_L / C_D) \tan \varphi}{\tan \varphi} \tan \theta_t \sqrt{\frac{\tau_{*c}}{\tau_*}} \right) \quad (4.19)$$

where q_{bt} = the bed load transport rate in the transverse direction; q_{bl} = the bed load transport rate in the longitudinal direction; δ = the deviation angle of the bottom velocity from the longitudinal direction and C_L / C_D = the ratio of the lift to the drag coefficient. In this equation, the ratio of the transverse-longitudinal rate is related to the deviation of the bottom velocity due to secondary currents and the deflection of particle path due to the transverse bed slope.

However this kind of treatment seems not very suitable for the 3D modeling of local scour around structures according to the author's numerical experiments. Two papers published by the author and his colleagues may be referred with the conventional bed slope factors and the new bed slope factor to be discussed, respectively (Nagakawa et al., 2004 and Zhang et al., 2005). This is due to the fact that the geometry of the scour holes is highly three-dimensional and that the magnitudes of the velocity components in the local scour area are generally comparable. In view of these arguments, a 3D analysis of the bed slope effect is presented hereafter.

(1) Horizontal bed

A 1D force analysis of a particle on a horizontal bed is firstly described. And then it will be extended to a 3D sloping bed.

The motion of the particle is a result of two opposing forces: the driving force and the stabilizing force. The former is caused by the hydrodynamics of the flow, while the latter is associated with the submerged weight of the particle. At the critical condition, the forces acting on a particle on a horizontal bed is schematically shown in Fig. 4.6, which include a drag force F_D , a lift force F_L and the submerged weight W . The drag force is in the direction of the flow and the lift force is normal to the flow.

The particle will be set to motion when the moments of the instantaneous drag force and lift force with respect to the point of contact are just larger than that of the submerged particle weight. For simplicity, such assumptions are usually made: (a) the ratio of the lift force to the submerged weight is relatively small and (b) the Reynolds number is so high that the drag force acts through the particle center. This will yield a simple form for the threshold condition for the sediment entrainment as

$$F_D \geq W \tan \varphi \quad (4.20)$$

It may be seen that only the drag force is presented on the left side as a driving force. However, as the lift force is directly related to the drag force, the effect of the lift force may be included in the drag force by empirical coefficients (Chang, 1988 and van Rijn, 1993).

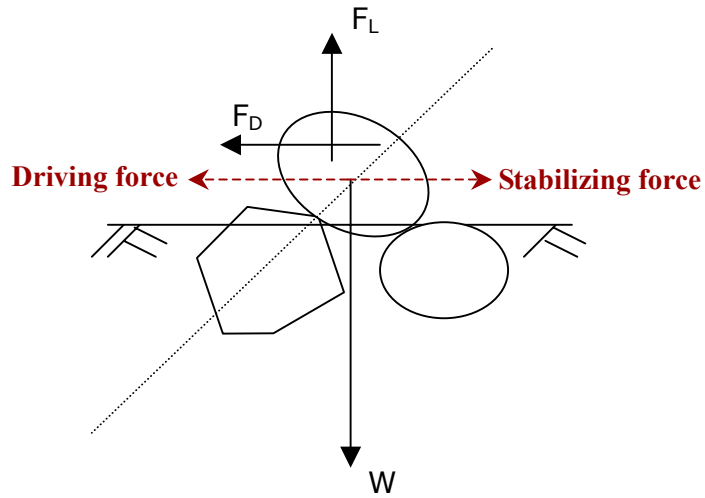


Fig. 4.6 Schematic of Forces on a Particle on a Horizontal Bed

(2) Sloping bed

The idea for a horizontal bed is easily extended to a sloping bed and the forces acting on a particle may be depicted in Fig. 4.7.

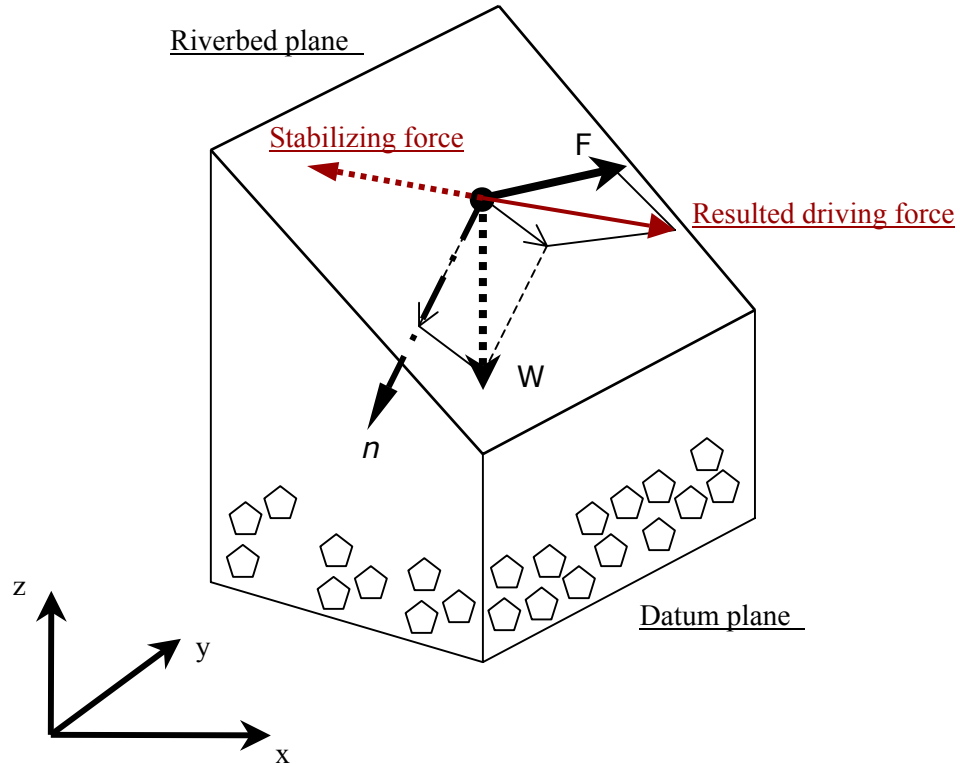


Fig. 4.7 Schematic of Forces on a Particle on a Sloping Bed

These forces include the submerged weight of the particle $\mathbf{W}$ and the hydrodynamic force $\mathbf{F}$. The riverbed has a normal direction $\mathbf{n}$. In a Cartesian coordinate system, the vectors in Fig. 4.7 may be written as

$$\mathbf{n} = \cos \alpha \mathbf{i} + \cos \beta \mathbf{j} + \cos \gamma \mathbf{k} \quad (4.21)$$

$$\mathbf{W} = W \cdot \mathbf{k} \quad (4.22)$$

$$\mathbf{F} = F(f_x \mathbf{i} + f_y \mathbf{j} + f_z \mathbf{k}) \quad (4.23)$$

where $\cos \alpha$, $\cos \beta$ and $\cos \gamma$ = the direction cosines of the vector $\mathbf{n}$; f_x , f_y and f_z = the direction cosines of the fluid force $\mathbf{F}$. It is noted that the fluid force $\mathbf{F}$ is parallel to the solid wall due to the assumption made in this study. Consequently the resulted driving force is written as

$$\mathbf{F}_R = (W \cos \gamma \cos \alpha + f_x F) \mathbf{i} + (W \cos \gamma \cos \beta + f_y F) \mathbf{j} + (-W \sin^2 \gamma + f_z F) \mathbf{k} \quad (4.24)$$

And the magnitude of the corresponding stabilizing force is

$$F_S = -W \cos \gamma \tan \varphi \quad (4.25)$$

The direction of the stabilizing force is opposite to the resulted driving force as described in Eq. 4.24. The threshold condition is then written as

$$F^2 + 2Wl_1 F + l_2 W^2 = 0 \quad (4.26)$$

where

$$l_1 = f_x \cos \gamma \cos \alpha + f_y \cos \gamma \cos \beta - f_z \sin^2 \gamma \quad (4.27)$$

$$l_2 = (\cos \alpha \cos \gamma)^2 + (\cos \beta \cos \gamma)^2 + \sin^4 \gamma - \cos^2 \gamma \tan^2 \varphi \quad (4.28)$$

The positive root of Eq. 4.26 is

$$F = W \left(\sqrt{l_1^2 - l_2} - l_1 \right) \quad (4.29)$$

Comparing the force above with that on a horizontal bed, the difference may be expressed by the bed slope factor K . In terms of shear stress, the threshold condition reads

$$K = \frac{\sqrt{l_1^2 - \sin^2 \gamma + \cos^2 \gamma \tan^2 \varphi} - l_1}{\tan \varphi} \quad (4.30)$$

One may find that the bed slope factor K is a function of not only the bed slope and the angle-of-repose but also the direction of the fluid force. Liu (1991) and Dey (2003) also theoretically derived a bed slope factor with the same force analysis. However, their formulae are quite complex and seem very difficult to ensure a numerical implementation without simplifications.

The effect of the bed slope on the effective shear stress may be observed in Eq. 4.24. One can find that a part of the sediment gravity contributes to the effective shear stress besides the hydrodynamic force. This contribution should be taken into account in the bed load transport formula, which will be discussed later. As is known that the direction of the sediment movement does not coincide with the flow direction on a sloping bed. A reasonable treatment is to assume that the sediment movement follows the direction of the resulted driving force. Then the sediment transport direction is explicitly obtained from Eq. 4.24.

For numerical simulation, ad hoc attention that should be paid related to the bed slope is the angle of sediment repose. During the bed evolution process, the bed slope may increase to a level larger than the angle of repose, in particular around the scour holes. This phenomenon cannot occur, or it is not stable in the actual case. In the numerical simulation, an indicator and a sand slide process are introduced to avoid its occurrence.

On the riverbed, the topography of the surface mesh (all the mesh elements are assumed to be planar polygons) is shown in Fig.4.8. Each time before adjusting the mesh, the program checks all the mesh nodes and corresponding neighbors on the bed surface.

For example, an angle steeper than the angle of repose (see Fig.4.8) has been detected connecting node A (x_A, y_A, z_A) and node B (x_B, y_B, z_B). If node B is higher, it should be lowered vertically to node B' , and node A should increase a distance to node A' at the same time. Finally the angle connecting A and B is equal to the angle of repose, i.e.

$$(z_B - \delta z_B) - (z_A + \delta z_A) = \tan \varphi \sqrt{(x_B - x_A)^2 + (y_B - y_A)^2} \quad (4.31)$$

in which z_A, z_B = the bed elevation at node A and node B , respectively; $\delta z_A, \delta z_B$ = vertical change amount (absolute value) for node A and node B , respectively.

On the other hand, the sediment conservation around node A and node B should be ensured amidst the adjustment, it results in

$$(S_1 + S_2 + S_3 + S_4 + S_5) \delta z_A = (S_6 + S_7 + S_8 + S_2 + S_1) \delta z_B \quad (4.32)$$

where S = projected area of the bed surface in the x - y plane. It is noted that during this adjustment, the conservation is assured for all the meshes including node A and node B .

From Eq.4.29 and Eq.4.30, the new position of node A and node B can be easily determined. For each node on the riverbed, the process is repeated. As the change of one node has an

influence on all the angles connecting the node and its neighboring nodes, new scans and possible adjustments are needed until all the angles are not greater than the angle of repose.

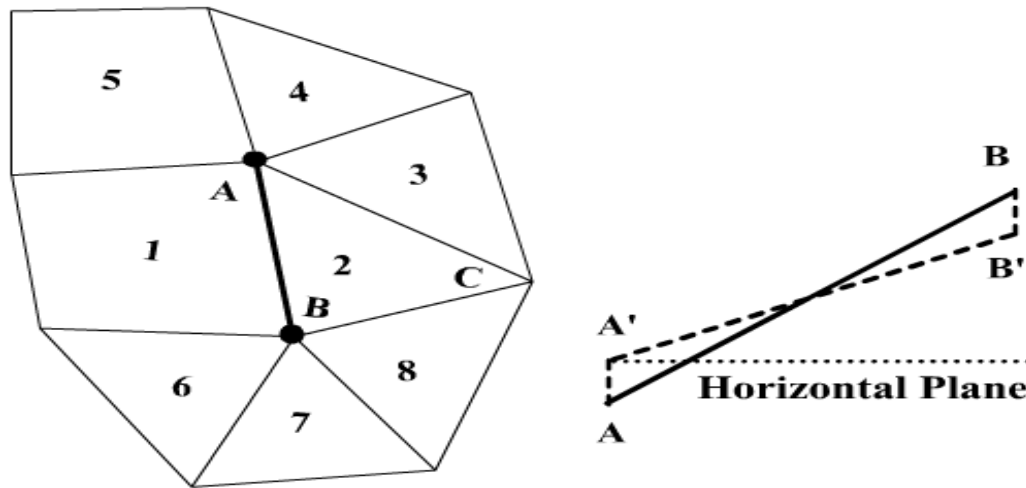


Fig. 4.8 Sand Slide Algorithm

Recently, Sekine (2004) proposed a new slope-collapse model to correct the unrealistic angle temporally occurred in the computation. In his model, the deepest slope on a plane is firstly sought. If this slope exceeds the angle of sediment repose, a collapse occurs which leads to an additional sediment transport along the deepest local slope. And the additional sediment transport is directly included in the calculation for the bed variation instead of using the sediment conservation as described in this study. He used this model to simulate the formation process of braided streams and found it was applicable.

4.3 Suspended load transport

4.3.1 Governing equations

In relatively lower concentrated sediment-laden flow, the interaction between different size fractions of suspended load may be negligible. The transport of suspended load is shown in Fig. 4.9 including both coarse sediment (transporting as dispersed particles) and fine sediment (transporting as flocs).

The transport of either coarse sediment or fine sediment in suspension is governed by the universal advection-diffusion equation, which has the following form for each size fraction.

$$\frac{\partial C_k}{\partial t} + (u_j - w_{sk} \delta_{j3}) \frac{\partial C_k}{\partial x_j} = \frac{\partial}{\partial x_j} \left[\left(\frac{v_t}{\sigma_c} \right) \frac{\partial C_k}{\partial x_j} \right] \quad (4.33)$$

where C_k = volumetric sediment concentration for size fraction k ; w_{sk} = sediment settling velocity for size fraction k ; σ_c = turbulent Schmidt number (equals 1.0 in this study) and δ_{ij} = the Kronecker delta (here $j=3$). The Einstein summation is used for the variables with repeated subscript in this equation.

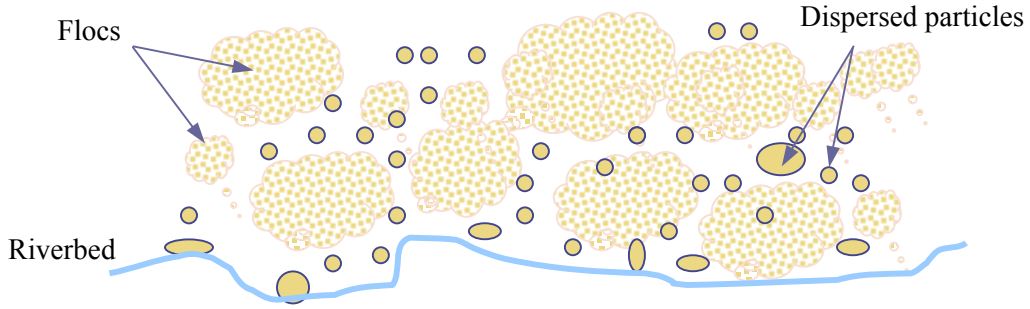


Fig. 4.9 Transport of Suspended Load

This equation has very similar form as the governing equations described in the hydrodynamic models. The same methods may be used to solve this equation. However, a sediment settling velocity has been included in the convective term. This may be treated as a convective term or a source term. Following the suggestion of Wu et al. (2000), the latter is adopted in this study.

A 1D form of the governing equation yields

$$\frac{\partial}{\partial t} (AC_k) = \frac{\partial}{\partial x} \left[A \left(\varepsilon_{sx} \frac{\partial C_k}{\partial x} - uC_k \right) \right] + B_s (E_k - D_k) \quad (4.34)$$

where ε_{sx} = the diffusive coefficient in the longitudinal direction and B_s = the sediment deposition width. When this equation is finite differenced, a TDMA (Tridiagonal matrix algorithm) solver may be used to find the root.

The settling velocity for coarse sediment is related to its gravity and is able to be evaluated from numerous formulae. For instance, the Rebey's Formula (1933) gives

$$w_s = \sqrt{\frac{2}{3} \left(\frac{\sigma}{g} - 1 \right) g d + \frac{36v^2}{d^2}} - \frac{6v}{d} \quad (4.35)$$

where σ = the sediment density and d = the representative size of the sediment fraction.

The settling velocity of fine sediment is much complex. As fine particles tend to aggregate and form flocs, their settling velocities are related to the flocs rather than the particle sizes. Aggregation of fine sediment is a consequence of the net attractive forces between particles which are brought close enough by the Brownian motion, differential settling and turbulence shear. Individual flocs contain appreciable sub-population of grain sizes and themselves may furthermore form aggregates of higher orders. A floc is different from an individual particle in several aspects: (a) The size of the floc is larger than the individual particles which contains; (b) The density of the floc is less than that of the particles due to the interstitial water; (c) The shape of the floc is more spherical than the particles which are usually plate-like and (d) The floc is very weak and tends to break up. Compared with the particles, the flocs are more unstable variables. Experimental methods are generally employed to investigate the floc related problems. The behavior of flocs is found to have strong dependence on the sediment concentration. At low concentrations flocs exist as individual units but joint together at higher concentration to form a network. As a result, the settling velocity increases with concentration at low concentrations, then attains a maximum value and thereafter decreases at large concentrations. A set of laboratory data obtained by Thorn (1981) is shown in Fig. 4.10.

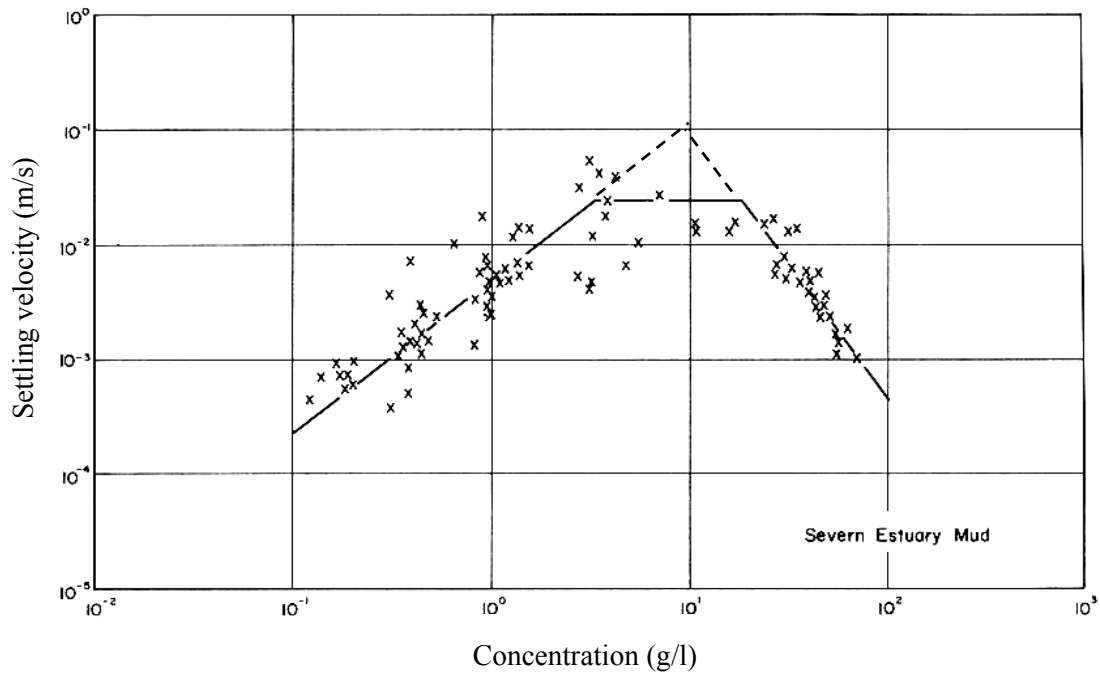


Fig. 4.10 Floc Settling Velocity versus Sediment Concentration (after Thorn, 1981)

The settling of flocs at high concentration is usually termed hindered settling. From Fig. 4.10, the concentration for hindered settling is around 10 g/l. Schweim et al. (2000) suggested a value range of 3-10 g/l. Nicholson and O'Connor (1986) used a value of 25 g/l when they applied a 3D model to the Grangemouth Harbor in Scotland. A general form of the settling velocity of a floc dominated by the sediment concentration is

$$w_f = \begin{cases} K_1 c^m & \text{for } c < c_h \\ K_1 c_h^m [1.0 - K_2 (c - c_h)]^{m_1} & \text{for } c \geq c_h \end{cases} \quad (4.36)$$

where w_f = the floc settling velocity (also presents the settling velocity of fine sediment fractions); c = the massive concentration of fine sediment fractions (i.e. $c = \sigma C$, where C is the volumetric concentration of fine sediment); c_h = onset concentration of hindered settling; K_1, K_2 = coefficients depending on particle mineralogy; m, m_1 = coefficients depending on particle size and shape. Based on the kinetics of flocculation, Krone (1962) proposed a theoretical value for m equal to 4/3. Mehta (1986) suggested a value between 1 and 2 according to experiments. The exponent m_1 is usually taken as 4.65 for small particles and 2.32 for large particles (Dyer, 1986).

This study does not intend to further the modeling of flocculation, which is also a very attractive and challenging field. Nevertheless, it is assumed that fine sediment will deposit as flocs whose settling velocity are evaluated with empirical formulae such as Eq. 4.36.

In the 1D governing equation, the near-bed fluxes appear as a source term. They are not found in the 3D governing equation because they presented as boundary conditions at the top of the bed load layer. This is discussed in the next sub-section.

4.3.2 Boundary conditions

Similar to those of the hydrodynamic models, the boundaries for the suspended load calculation include the inlet boundary, outlet boundary, the free surface, symmetrical planes and the near-bed boundary.

(1) Inlet boundary

If the inlet sediment distribution is known, the known values should be of course used. However, the data available at the inlet is usually not detailed enough for direct use in the numerical simulation. Generally, the measured sediment discharge is plotted versus the water discharge, from which the relationship between the sediment discharge and the water discharge may be obtained. The input sediment discharge is then acquired by some kind of interpolation or extrapolation from the graph. In the river engineering, an equilibrium sediment concentration profile is commonly used for coarse sediment and fine sediment is assumed to be uniformly

distributed in the vertical direction. If the Lane-Kalinske formula is used, the inlet coarse sediment has a profile in the vertical direction as

$$\frac{C}{C_e} = \exp \left[-15 \left(\frac{z - z_a}{h} \right) \left(\frac{w_s}{u_*} \right) \right] \quad (4.37)$$

where z_a = reference level (in this model $z_a = 0.05h$, is the same as the interface of the bed load layer and the suspended load layer); C_e = near-bed equilibrium concentration; z = depth in vertical direction. The near-bed equilibrium concentration may be calculated from the Ashida-Michiue formula (1971) or the van Rijn formula (1984b). In this study the former is adopted, which is obtained on the basis of the consideration of the particle motion in the turbulent flow. Considering the bed cohesiveness, the concentration is written as

$$C_{ek} = f_b p_{ek} k_0 \left(\frac{1}{\sqrt{2\pi} \zeta_0} e^{-\zeta_0^2/2} - \frac{1}{\sqrt{2\pi}} \int_{\zeta_0}^{\infty} e^{-\zeta^2/2} d\zeta \right) \quad (4.38)$$

where p_{ek} = percentage of size fraction k in the suspended load; k_0 = ratio constant; $k_0 = 0.025$, $\zeta_0 = w_{sk} / \sigma_p$, $\zeta = w_p / \sigma_p$, $\sigma_p = 0.75u_*$, σ_p = standard deviation of w_p and w_p = vertical velocity variation.

(2) Outlet boundary

At the outlet boundary, the gradient of the sediment concentration is usually assumed to be zero, i.e.

$$\frac{\partial C_k}{\partial s} = 0 \quad (4.39)$$

where s stands for the longitudinal direction of the channel.

(3) Free surface

The vertical sediment flux at the free surface is zero, this leads to

$$\frac{v_t}{\sigma_c} \frac{\partial C_k}{\partial z} + w_{sk} C_k = 0 \quad (4.40)$$

(4) Near-bed boundary

The near-bed boundary stands for the interface between the suspended load layer and bed load layer, which is expressed by the downward flux and the upward flux. As the motion

mechanism of coarse sediment and fine sediment is different, the evaluation method of the fluxes is different as well. Moreover, there is evident that the near-bed fluxes are generated by the simultaneous erosion (upward) and deposition (downward), but it is preferred to consider the two processes discontinuously from the perspective of mathematical modeling.

(a) Coarse sediment

The coarse sediment is non-cohesive. Following the conventional methods, the downward flux is calculated from

$$D_k = w_{sk} C_{ak} \quad (4.41)$$

where C_{ak} = the ambient concentration for size fraction k near the bed (i.e. the interface of the bed load layer and the suspended load layer z_a). As the calculated concentration is at the center of the CV, the ambient concentration near the bed has to be obtained from the neighboring CVs. This may be achieved by assuming a linear concentration distribution. And the upward flux is generally assumed to be the one under the equilibrium condition, i.e.

$$E_k = w_{sk} C_{ek} \quad (4.42)$$

It is noted that the bed cohesiveness has been already included in the equilibrium concentration C_e as in Eq. 4.38.

(b) Fine sediment

As has mentioned before, fine sediment deposits as flocs, which are determined by the properties of the aggregation and not of the original particles. This also indicates that all the size fractions of fine sediment will deposit at the same time with the same velocity. The most frequently used formula is proposed by Krone (1962). According to Krone's deposition law, the deposition of a floc is considered as a stochastic process depending on the settling velocity of the floc and whether or not it is capable of surviving the near bed shear stress. The mathematical form of the deposition flux of fine sediment is

$$D_f = \begin{cases} C_f w_f \left(1 - \frac{\tau_b}{\tau_{cd}} \right) & \text{for } \tau_b < \tau_{cd} \\ 0 & \text{for } \tau_b \geq \tau_{cd} \end{cases} \quad (4.43)$$

where D_f = deposition flux of fine sediment; C_f = volumetric concentration of fine sediment; τ_b = near-bed shear stress; τ_{cd} = critical shear stress for deposition, depending mainly on the properties of the flocs. Krone (1962) used a value of 0.06 N/m² in the Bay of San Francisco. The values of 0.08, 0.10 and 0.15 N/m² have been used by Metha (1986) in different riverbeds. A value range of 0.05-0.25 N/m² has been reported in a numerical simulation by van Ledden

(2002) according to field measurements. It is noted that this equation expresses the deposition flux of flocs, i.e. all the size fractions of fine sediment. However in order to solve the governing equations, the deposition flux for each size fraction D_k is needed. This may be obtained from the deposition flux of the floc D_f . After deposition, the floc is assumed to turn into dispersed particles. In order to calculate the deposition flux of each size fraction D_k , it is therefore necessary to estimate the size distribution of a broken floc. Assuming the percentage of a specific size fraction in the floc is governed by its concentration and proportional to the concentration, the following relation could be obtained

$$D_k = \frac{C_k}{C_f} D_f \quad (4.44)$$

Eq. 4.44 leads to the deposition flux for each size fraction D_k which contains the sediment concentration for each size fraction C_k . During the calculation, this term is treated explicitly in the source term.

Erosion formulae for fine sediment are divided into two kinds. The first kind is for riverbeds which are stratified with respect to cohesive property variations with depth and the second kind is for those with relatively uniform properties over the depth.

The representative formula of the first kind is that proposed by Parchure and Mehta (1985), which has the following form

$$E = \begin{cases} E_f \cdot \exp \left[\alpha_e (\tau_b - \tau_s)^{\frac{1}{2}} \right] & \text{for } \tau_b \geq \tau_s \\ 0 & \text{for } \tau_b < \tau_s \end{cases} \quad (4.45)$$

where E = the erosion amount; E_f = the empirical floc erosion rate; α_e = empirical coefficient and τ_s = the bed shear strength varying with depth and also influenced by the type of sediment, bed consolidation period, etc. Due to the increasing of the bed shear strength with depth into the bed, the erosion rate will be decreased. With the development of the bed erosion, the shear strength at the bed surface increases, which will limit the extent of the erosion.

The basic idea to evaluate the erosive amount from a relatively uniform bed (the second kind) follows Partheniades' algorithm (1965). This kind of formulae is frequently used in the cohesive sediment modeling. Compared with the first kind, formulae of this kind have a simpler form and hence include less information about the detailed physical processes. Both kinds of formulae include important parameters which should be determined empirically. And in the literatures available, the empirical parameters in Partheniades' formula have been tuned by a greater number of applications. Consequently, the use of this formula is preferred in this study.

According to Partheniades' experimental research, the erosion rate is proportional to the excess of the shear stress over the critical erosive shear stress normalized with respect to the

latter. Considering the ratio of each fraction of fine sediment available from the bed, one has

$$E_k = \begin{cases} 0 & \text{for } \tau_b < \tau_{cek} \\ m_{ek} p_{bk} \left(\frac{\tau_b}{\tau_{cek}} - 1 \right) & \text{for } \tau_b \geq \tau_{cek} \end{cases} \quad (4.46)$$

where m_{ek} = erosion parameter for size fraction k ; τ_{cek} = critical erosive shear stress for size fraction k . This equation indicates that the erosion rate is constant if the near-bed shear stress is constant. The value of the erosion parameter m_e varies very much according to the bed materials and the consolidation of the riverbed. With a test over 200 natural and made-up soil samples, Ariathurai and Arulanandan (1978) analyzed some principle factors influencing the erosion parameter and suggested a range from 0.003 g/(cm² min) to 0.03 g/(cm² min). van Ledden (2002) suggested a value range from 10⁻³ kg/(m² s) to 10⁻⁵ kg/(m² s) according to measurements for natural beds in open water. An overview of measurements shows a value range from 0.1N/m² to 1.0N/m² for the critical erosive shear stress in a cohesive bed (van Rijn, 1993).

4.4 Bed load transport

The adjustment of bed load transport to the new hydraulic conditions proceeds rapidly, so a formula type of approach can be used to model the bed load transport rate. Amongst numerous formulae, the approach for bed load transport rate proposed by Ashida and Michiue (1972a) has achieved a broad application to the rivers in Japan. Considering the bed cohesiveness, the Ashida-Michiue Formula for each size fraction is modified as below.

$$\frac{q_{bk}}{\sqrt{(s-1)gd_k^3}} = 17 f_b p_{bk} \tau_{*ek}^{3/2} \left(1 - \frac{u_{*ck}}{u_*} \right) \left(1 - \frac{\tau_{*ck}}{\tau_{*k}} \right) \quad (4.47)$$

where q_{bk} = bed load discharge for fraction k ; s = specific gravity of sediment; d_k = diameter of fraction k ; p_{bk} = percentage of fraction k in the bed composition; τ_{*k} , τ_{*ck} , τ_{*ek} = dimensionless shear stress, critical shear stress and effective shear stress for size fraction k , respectively; u_* , u_{*ck} = friction velocity and critical friction velocity for size fraction k , respectively. The following relation exists between these quantities.

$$\tau_{*k} = \frac{u_*^2}{(\sigma/\rho - 1)gd_k} \quad , \quad \tau_{*ck} = \frac{u_{*ck}^2}{(\sigma/\rho - 1)gd_k} \quad , \quad \tau_{*ek} = \frac{u_{*e}^2}{(\sigma/\rho - 1)gd_k} \quad (4.48)$$

where u_{*e} = the effective friction velocity.

To calculate the critical friction velocity for each size fraction, the following equations firstly

contributed by Egiazaroff (1965) and later extended by Ashida and Michiue (1972b) are adopted.

$$\frac{u_{*ck}^2}{u_{*cm}^2} = \begin{cases} 0.85 & \text{if } d_k / d_m < 0.4 \\ \frac{1.64}{[\lg(19d_k / d_m)]^2} \frac{d_k}{d_m} & \text{if } d_k / d_m \geq 0.4 \end{cases} \quad (4.49)$$

where subscript m = the mean size of sediment mixtures. The critical friction velocity u_{*cm} for the mean size d_m may be evaluated with the Iwagaki formula (expressed as dimensionless critical shear stress).

$$\tau_{*c} = \begin{cases} 0.05 & \text{if } R_* \geq 671.0 \\ 0.00849 R_*^{3/11} & \text{if } 162.7 \leq R_* < 671.0 \\ 0.034 & \text{if } 54.2 \leq R_* < 162.7 \\ 0.195 R_*^{-7/16} & \text{if } 2.14 \leq R_* < 54.2 \\ 0.14 & \text{if } R_* < 2.14 \end{cases} \quad (4.50)$$

where

$$R_* = \frac{\sqrt{(s-1)gd_m^3}}{\nu} \quad (4.51)$$

The critical friction velocity and shear stress in Eq. 4.49 and Eq. 4.50 are valid only for a horizontal bed. The bed slope factor introduced in Section 4.2 should be added to the equations so that they can be used in sloping bed conditions. Furthermore, the effective shear stress in Eq. 4.47 also needs corrections by including part of the particle gravity as has described in Section 4.2.

4.5 Summary

This Chapter concerns the mathematical modeling of sediment transport in alluvial rivers. Taking into account the special characteristics due to the construction of spur dykes, the sediment sorting, local scour effect and fine sediment transport have been emphasized. The contents of this chapter may be summarized as below.

For the convenience of numerical modeling, the transport of sediment load is divided into bed load and suspended load according to different motion styles. The latter is furthermore divided into coarse sediment and fine sediment according to the particle cohesiveness. Fine sediment

moves as flocs in the water column and it may alter the bed cohesiveness through exchanging with the local bed.

After that, the modeling of the riverbed is introduced. A mixture of sediment is simplified as a collection of many sediment fractions. Each fraction has a representative size. The contribution of all the size fractions to the bed level change leads to the final bed morphology. In order to simulate the bed sorting process, a multi-layer approach is employed in this thesis. The riverbed is vertically divided into an active layer, a transition layer and a series of deposited layers. The distribution of the sediment size in different layers varies with the flow and bed evolution. But each layer is assumed to be homogeneous within itself at any given time. The sediment contained in the active layer is the only material available for erosion during a calculation time step. The active layer changes its elevation and sediment size distribution with the transport of sediment load but is kept a constant thickness. The variation of the bed morphology is expressed by the changing of the thickness of the transition layer and the total number of the deposited layers. This method can memorize the variation processes of both the bed configuration and the bed composition but still maintain simplicity.

Due to the deposition of fine sediment, the riverbed becomes cohesive, which has a significant effect on the erosion process. A new bed model is introduced to consider a riverbed as a non-cohesive particle cluster surrounded by a cohesive force field. Consequently, the erosion process is considered as a two-stage process: the erosion from a non-cohesive bed followed by the passing through the cohesive force field. The former amount is evaluated with the traditional methods based on non-cohesive sediment. And the latter is expressed as a possibility which is simply related to the clay content in the bed surface. The local bed slope will have influences on the sediment threshold conditions, the effective bed shear stress and the path of the sediment movements. The change of the threshold conditions is expressed by a new bed slope factor. This bed slope factor is based on a 3D force analysis and is related to the bed slope, the angle of sediment repose and the flow direction as well. It has been confirmed to be more promising than the conventional combination of longitudinal bed slope factors and transverse bed slope factors. With this treatment, the particle gravity is also included in the effective shear stress. The direction of the sediment movements is assumed to follow the direction of the resulted driving force, which may be explicitly obtained. A sand slide algorithm is introduced to avoid the unrealistic slope possibly predicted by the numerical procedure. This algorithm guarantees that the bed slope does not exceed the angle of sediment repose as well as ensures that the sediment conservativeness is maintained.

The bed modeling is followed by the simulation of suspended load. In order to determine the sediment concentration, the same advection-diffusion equation can be used for both coarse sediment and fine sediment. However, there are some differences due to their different motion

mechanisms. Coarse sediment moves as dispersed particles, which is easily modeled with non-cohesive based methods provided that the reduction of erosion rate due to the bed cohesiveness should be accounted for. Fine sediment moves as flocs. Hence, the deposition of all the size fractions in fine sediment range is at the same velocity, which is estimated with the commonly used Krone's approach (Krone, 1962). This process is considered as a stochastic process which is controlled by the settling velocity and the probability of the survival of deposited flocs subject to the near-bed shear stress. The deposition flux for each size fraction is proportional to the percentage of the corresponding size fraction in the floc. The erosion amount is evaluated with the frequently used Partheniades' algorithm (Partheniades, 1965). The erosion rate is proportional to the excess of the bed shear stress over the critical resistance.

Finally, the transport of bed load is presented. The Ashida-Michiue formula (1972a) is adopted in this thesis. However, this formula has been modified considering the influence of the bed behavior such as the bed sorting process, the bed cohesiveness and the bed slope.

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Chapter 5

Model verifications and applications

5.1 Introduction

5.1.1 Flow-sediment-riverbed process

The bed evolution process in channels with spur dykes is complex because of the strong interaction and coupling between the flow, sediment and riverbed (see Fig. 5.1a). A small change in one aspect may cause simultaneous changes in the other two, and which in return will affect the aspect that provokes the changes. This process is repeated till a new balance is sought in river conditions.

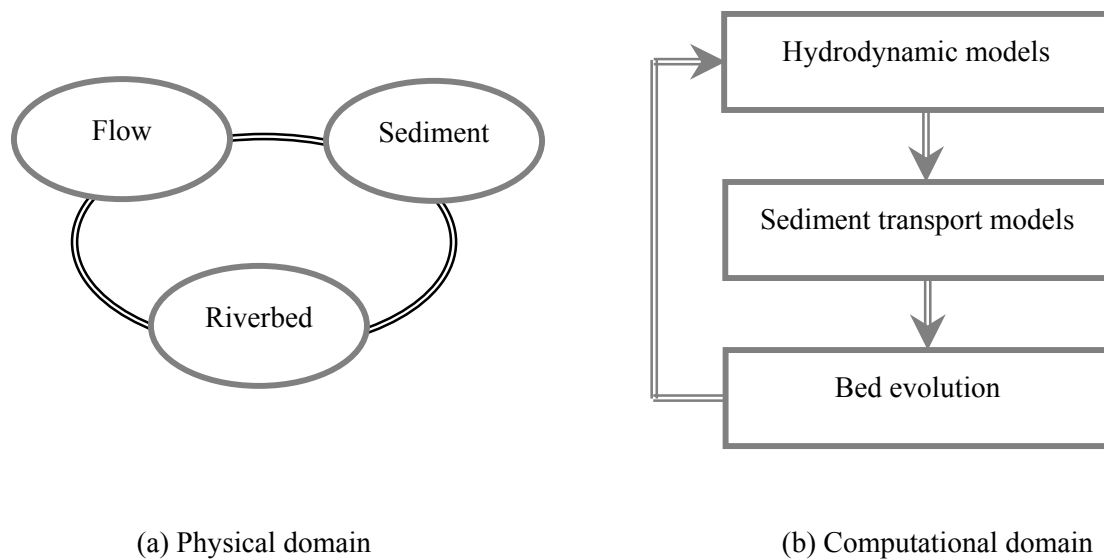


Fig. 5.1 Schematization of the Flow-Sediment-Riverbed Process

Direct simulation of this dynamic feedback process is quite challenging with the current science and technology. From the perspective of mathematical modeling, it is generally preferred to consider the complex process discontinuously and the three aspects separately.

Fig. 5.1b illustrates a typical flow-sediment-riverbed process in a computational domain. The flow field is firstly calculated with hydrodynamic models based on the current bed morphology.

During the calculation, the sediment is assumed not to affect the flow characteristics. After that, the transport of bed load and suspended load is modeled under the calculated flow conditions. The transport of sediment loads leads to the bed variation in both elevation and composition. The variation of the bed elevation is then included in the new time step for the flow field calculation. This is achieved by using a movable mesh system in this thesis. The new flow field triggers a new round of sediment transport and bed variation. This process is repeated till an equilibrium state is achieved or a prescribed time is covered. In the previous chapters, mathematical modeling of all the components in the above processes has been presented. Consequently, a numerical simulation of this process is possible.

5.1.2 Model selection

Numerical models provide effective ways to investigate the flow and sediment transport related problems in hydraulic engineering. However, a theoretically elaborate model may lead to unfavorable result in practices. In order to achieve the maximum benefit from the numerical models, a comprehensive insight into the problem involved is of great importance.

According to van Rijn (1993), the use of numerical models in engineering depends on the following aspects: (1) The available input data; (2) The available calibration data; (3) The physical reliability; (4) The scale of the problem; (5) The required accuracy and (6) The available budget.

It may be seen that the selection of the numerical models is not only a technical concern but also a concern involving many other non-technical factors. In particular, the available data is very important in selecting the mathematical models. An input data of poor quality may lead to disappointing result even with the most elaborate models. As the result of the numerical simulation is always an approximate of the reality, any numerical model to date introduces idealization and empiricism and an accurate prediction requires a careful prior calibration.

In the previous chapters, a series of numerical models have been presented including hydrodynamic models and sediment transport models. These models can be combined as morphological models to simulate the flow and bed evolution in different levels to accommodate different problems and different requirements. However, it is not possible to test all these combinations here due to the limited data and computational cost.

In this chapter, the proposed hydrodynamic models (Chapter 3) and the sediment transport models (Chapter 4) are integrated in two levels, namely, 1D and 3D. The 1D model is applied to reproduce the fluvial process of the Yodo River system in West Japan. While the 3D model is used to simulate the local flow and local scour around spur dykes in two laboratory experiments.

5.2 Bed evolution modeling of the Yodo River system

5.2.1 Introduction

The Yodo River system is situated in the west part of Japan. The catchment basin of the Yodo River system extends six prefectures of Mie, Shiga, Kyoto, Osaka, Hyogo and Nara. The total length of the main stream is 75km and the total catchment area covers 8,240km². In order of the catchment area, the Yodo River system ranks No. 7 amongst Japan's main rivers.



Fig. 5.2 Study Domain Introduction (Map Adapted from 2002 *Lake Biwa & the Yodo River*)

The origin of the Yodo River is Lake Biwa, which is the largest freshwater lake in Japan with a surface area of 670 km² and is one of the most ancient lakes in the world along with Lake Baikal and Lake Tanganyika. About 460 rivers in varying sizes feed into Lake Biwa. The water in Lake Biwa is firstly discharged into the Seta River and then runs through the Uji River, which is subsequently jointed by the Kizu River and the Katsura River. The water runs down the Yodo River and finally flows into Osaka Bay (see Fig. 5.2 for details).

In 1874, a spur dyke was constructed on the Yodo River for bank protection and navigation maintenance under the guidance of the Dutch engineer de Rijke. After that many spur dykes have been equipped in this river basin, which have significantly changed the river conditions. Amidst which, the change of the sediment transport is one of the most important considerations. Fine particles deposited around the spur dykes and formed favorite habitats for the aquatic flora and fauna. Their existence also altered the bed cohesiveness and bed topography, which could not be predicted with conventional models.

In this section, the newly proposed morphological model is applied to the Yodo River system. As a preliminary work, the calculation is implemented in 1D. This is due to the limited data available as well as the computational cost. The emphasis is put on the effect of fine sediment transport, which has thrown the conventional models into trouble. This section is organized as follows. The river system in the computational domain is briefly introduced in Sub-section 5.2.2 after this Introduction. In Sub-section 5.2.3, the initial conditions are presented, which is followed by the computational conditions in Sub-section 5.2.4. The computational results are shown in Sub-section 5.2.5, together with the comparison with the field data and conventional model results.

5.2.2 Computational domain

The bed evolution process of the Yodo River system from 1975 to 1998 is calculated with a 1D model in this study. The location of the study domain is shown in Fig. 5.2.

In the computational domain, the river system is modeled longitudinally with 339 cross-sections. The distance between two consecutive sections is 200m. Each cross-section is identified with its River Mile as shown in Fig. 5.3. For example, the River Mile 9.8k means 9.8km from the river mouth. It is noted that the two tributaries (i.e. the Kizu River and the Katsura River) have their own River Miles which stand for the distance from the confluence. For clarity, the abbreviation of the river name has been put before the River Miles for the tributaries, e.g. Ka 7.0 means that the section is in the Katsura River and is 7.0km from the confluence. In the confluence area, some sections are shared by two or three rivers and the River Miles of the main stream are adopted. The cross-section is made up of a main channel part and a

flood plain on either side, which are attributed different roughness according to the riverbed conditions.

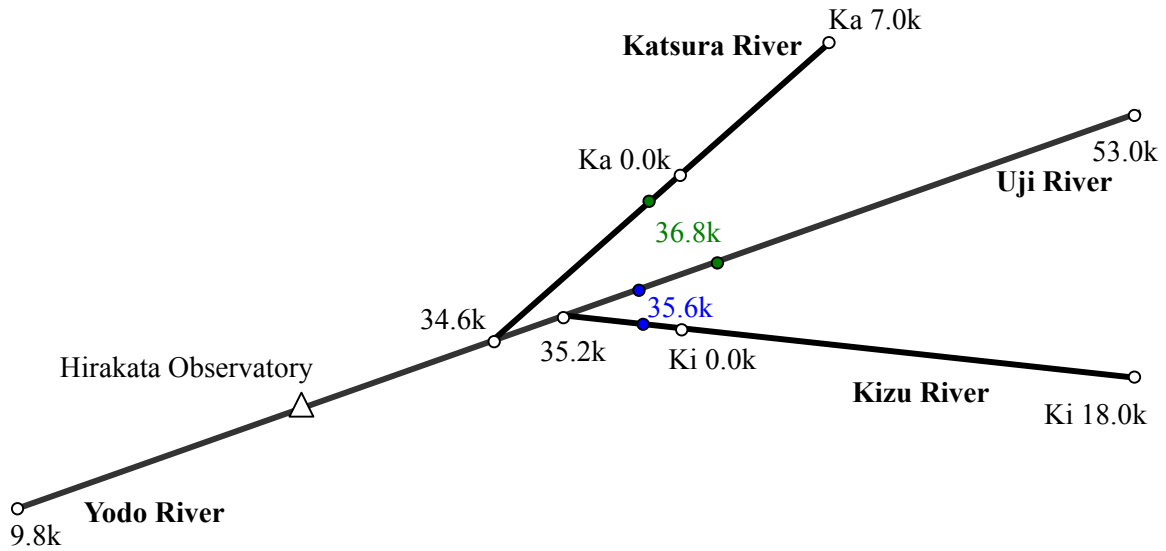


Fig. 5.3 Sketch for the River System in the Computational Domain

5.2.3 Initial conditions

The measured cross-section data in 1975 is selected for the initial bed. Shortage of field data, the elevation data for the riverbed in 1976 is used for the Kizu River. The initial riverbed is divided into 40 layers in the vertical direction with the thickness of each layer 0.5m.

The bed composition has been analyzed in some representative cross-sections including the main channel and the flood plains on both sides. The mean value is selected as the bed composition in the corresponding cross-section. Bed composition of the sections without any field data is obtained by interpolation or extrapolation from that of the known sections. The representative sizes for the sediment mixtures are chosen as Table 5.1. The finest four fractions are considered as fine sediment.

Table 5.1 Representative Sizes in the Simulation (Unit: mm)

63.048	44.581	31.524	22.439	13.435	6.718	3.082	1.304
0.601	0.326	0.163	0.089	0.064	0.046	0.012	0.002

The roughness data is evaluated according to the bed conditions such as the vegetations, which is shown in Table 5.2.

Table 5.2 Bed Roughness

River Mile	Main channel	Flood plain (Left)	Flood plain (Right)	River Mile	Main channel	Flood plain (Left)	Flood plain (Right)
11.4k	0.022	0.030	0.030	44.2k	0.030	0.030	0.060
12.8k		0.055	0.055	44.8k		0.055	0.055
20.4k		0.030	0.030	45.4k		0.030	0.060
22.6k		0.030	0.035	46.2k	0.027	0.030	0.060
24.4k	0.031	0.030	0.030	47.6k		0.030	0.030
24.6k		0.030	0.055	49.4k	0.035	0.030	0.030
25.6k		0.030	0.035	50.6k		0.030	0.030
26.4k		0.030	0.030	53.0k	0.035	0.030	0.030
26.8k		0.055	0.030	Ka 0.0k		0.055	0.030
28.2k		0.030	0.030	Ka 1.6k	0.03	0.060	0.030
29.8k		0.035	0.030	Ka 3.2k		0.060	0.030
31.0k		0.035	0.055	Ka 3.6k		0.055	0.055
31.4k		0.030	0.055	Ka 4.4k		0.030	0.030
32.2k		0.035	0.055	Ka 5.2k		0.030	0.035
33.2k		0.035	0.030	Ka 7.0k		0.030	0.030
33.4k		0.035	0.035	Ki 0.0k	0.037	0.055	0.030
33.8k		0.055	0.035	Ki 3.2k		0.055	0.055
34.4k		0.055	0.035	Ki 5.0k		0.055	0.035
34.8k		0.055	0.030	Ki 6.0k		0.035	0.037
36.8k		0.055	0.030	Ki 6.4k		0.055	0.040
37.8k	0.030	0.060	0.060	Ki 15.4k		0.030	0.040
43.6k		0.060	0.055	Ki 18.0k		0.055	0.035

5.2.4 Computational conditions

Two kinds of computations have been implemented and called Case1 and Case2, respectively. In Case1, the transport of fine sediment is simulated with the current model. In Case2, fine sediment is excluded in the bed composition and treated as conventionally termed wash load.

The computational conditions are almost the same as those of the calculation with conventional models carried out by Newjtec Inc. (2004) except the special treatment for fine sediment. The daily discharge (hourly discharge during the flood periods) and yearly navigation dredging data from 1976 to 1988 are employed for the calculation. At the downstream, the

stage-discharge relation is specified. During the 23 years, the Yodo weir was constructed at the downstream of the study domain. This is taken into account by using different stage-discharge relations resulted from the measured data.

The input fine sediment amount is obtained from the graph by plotting the measured SS (Suspended solids) data with the water discharge as shown in Fig. 5.4 and Fig. 5.5. The SS data may also include some fine parts of coarse sediment. However, as there is no quantitative information on the detailed size fraction of the measured SS, this is not taken into account for the time being.

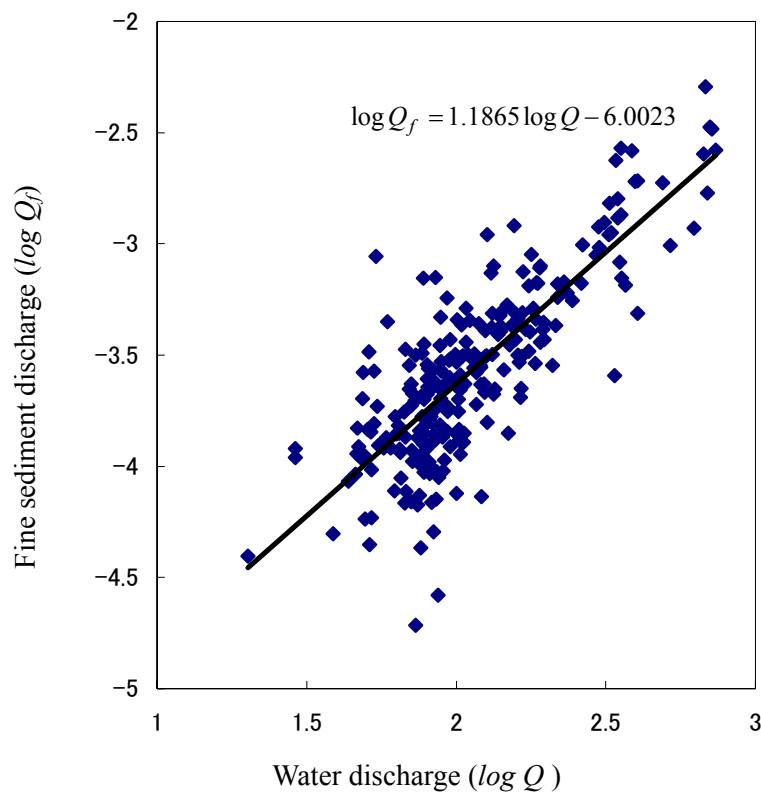


Fig. 5.4 Fine Sediment-Water Discharge Rating (Uji River)

The logarithmic coordinate system is adopted in the two figures. It may be seen that the graphs basically show somewhat linear relations between the two coordinate components. From these relations, the fine sediment discharge is able to acquire for any input water discharge. The concentration of fine sediment is then assumed to be uniformly distributed in the vertical direction, which serves as the inlet boundary for fine sediment.

Due to the complexity of fine sediment movement, the formulae for fine sediment involve strong empiricism. Some important coefficients are listed in Table. 5.3.

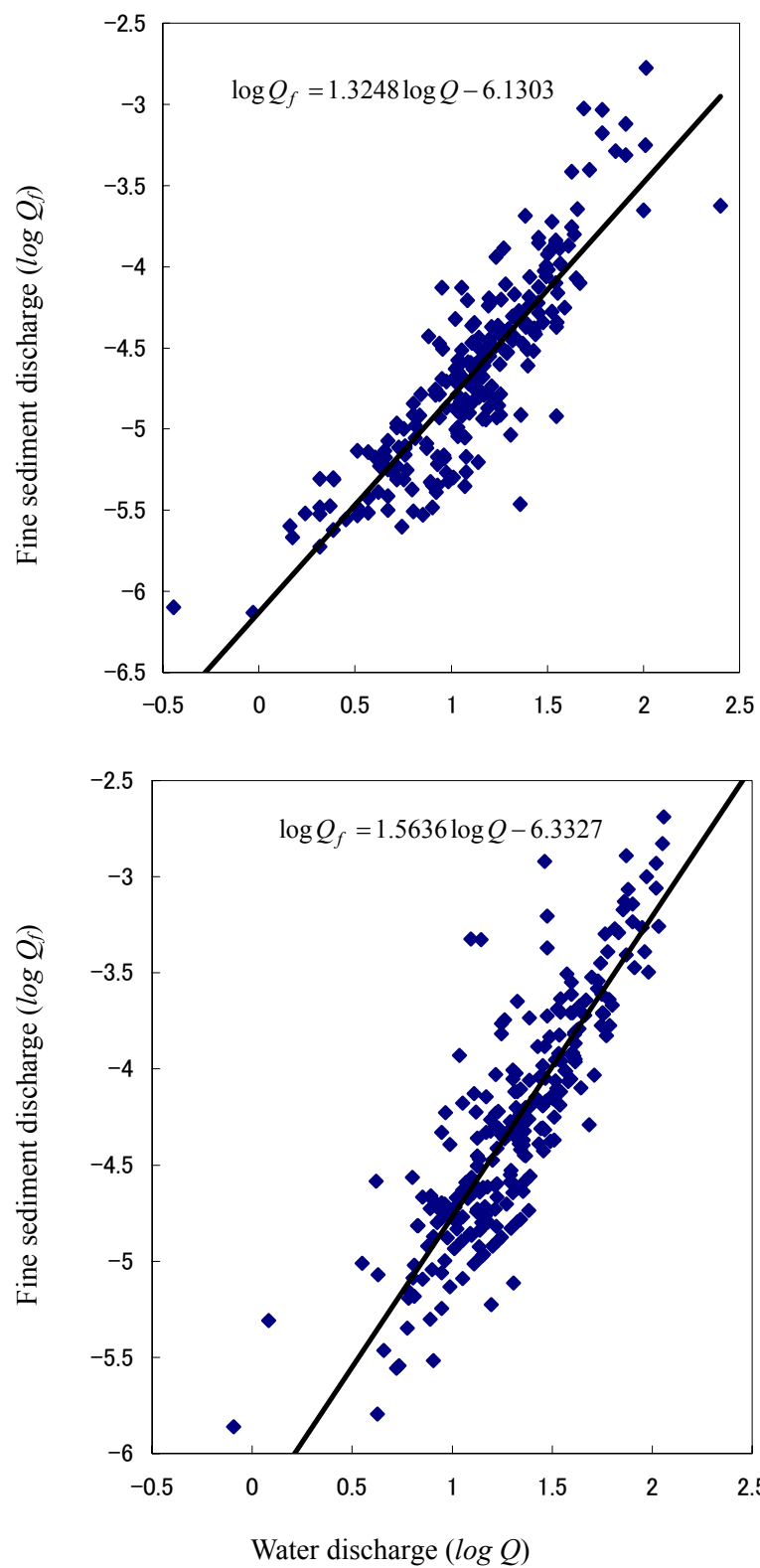


Fig. 5.5 Fine Sediment-Water Discharge Rating (Katsura River, Top; Kizu River, Bottom)

Table 5.3 Coefficients Used in the Computation

Coefficient	Description	Value
P_c	Critical clay content	10.0%
τ_{cd}	Critical deposition shear stress	0.25 N/m ²
τ_{cek}	Critical erosive shear stress	0.8-0.65 N/m ²
m_e	Erosion parameter (same for all fine sediment fractions)	2.0×10^{-10} m/s
c_h	Onset concentration for hindered settling	10.0 kg/m ³
K_1	Coefficients for the settling velocity of the floc	0.006 m ⁴ /kg/s
K_2		0.01 m ³ /kg
m		4/3
m_1		4.65
λ	Bed porosity	0.4

The values of these coefficients are selected from limited literatures. They have been tuned by either laboratory experiments or filed measurements. At the current stage, it is assumed that these values are valid in this river basin.

5.2.5 Computational results

Limited to the available data at present, the comparison of the variation of the total sediment amount and fine sediment concentration is not possible. As the sediment transport is directly related to the bed morphology, limited bed level and water stage data are used for analyses.

The mean bed elevation in 1988 is shown in Fig. 5.6, Fig. 5.7 and Fig. 5.8 for the Katsura River, the Kizu River and the Yodo River (including the Uji River), respectively. These figures compare the two computational cases with the field data.

For the two main tributaries, the Katsura River and the Kizu River (Fig.5.6 and Fig.5.7), there is no much difference in the two computational cases. Both cases are able to give reasonable results similar to the field data. From these observations such conclusions may be drawn that in these two rivers, fine sediment behaves like wash load which is mostly carried out by water and has nearly no exchange with the local riverbed. This coincides with the fact that almost no appreciable fine sediment amount is found in either of the riverbed in the particle sieve analysis.

Due to a large dam (Amagase Dam) upstream, the sediment flowing into the Uji River has been effectively controlled. Little morphological variation can be observed in the riverbed, and this phenomenon has been adequately reproduced by the calculation of either case (see Fig. 5.8).

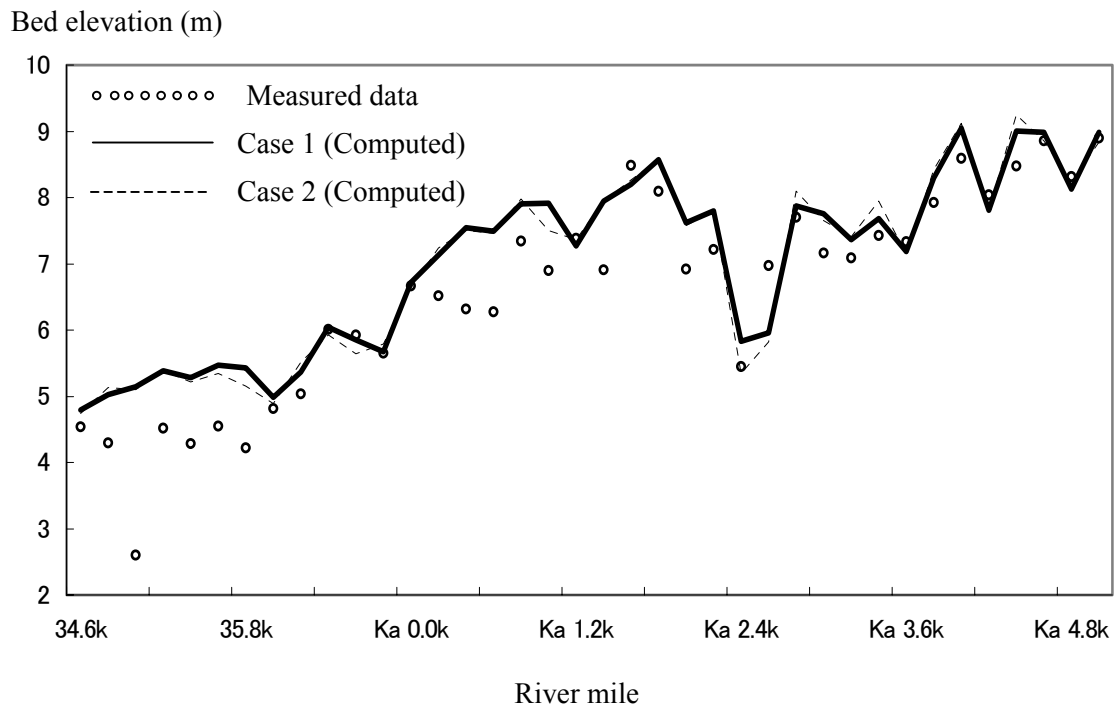


Fig. 5.6 Bed Elevation of the Katsura River in 1998

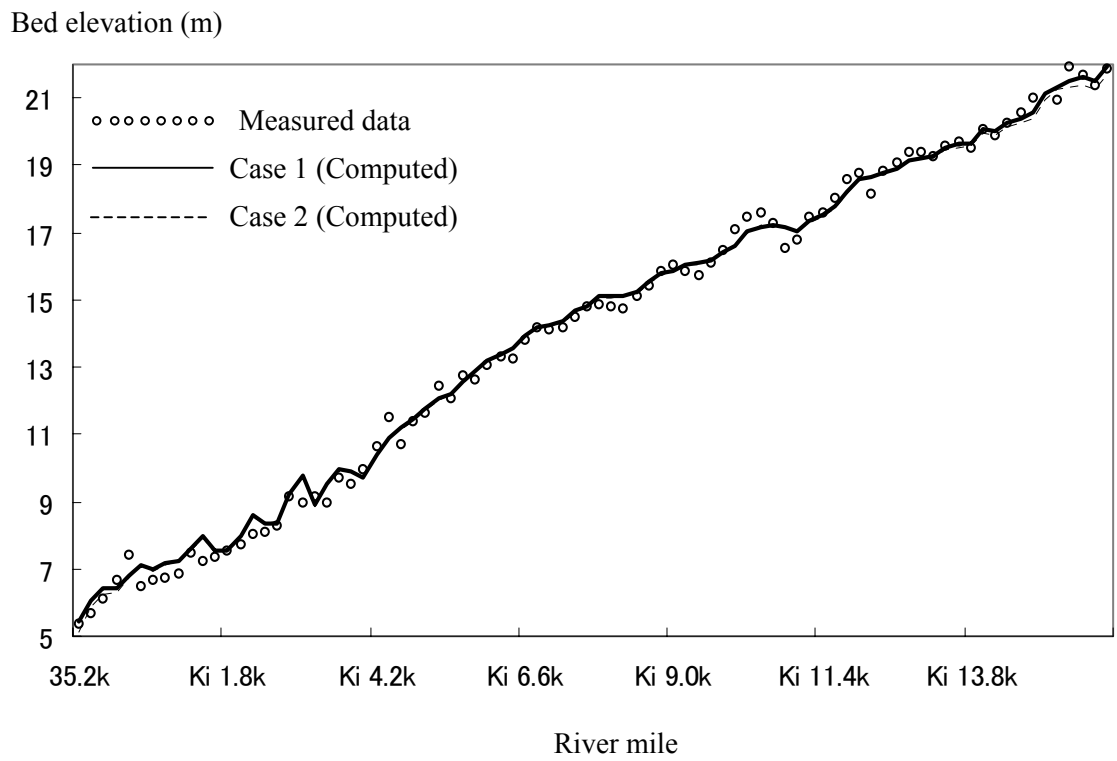


Fig. 5.7 Bed Elevation of the Kizu River in 1998

The two cases predict different results in the Yodo River stretch especially in the area downstream of the Hirakata Observatory. Omitting the effect of fine sediment, the conventional model case gives a much lower elevation than the field data in the downstream area of the Yodo River (in particular downstream of 19.4k) where fine sediment plays an important role. It means that there is a significant overestimation of the sediment erosion, which is caused by the inherent deficient of non-cohesive based models. The sediment erosion amount will be decreased due to the bed cohesiveness which is caused by the deposition of fine sediment. However, this effect is not taken into account in conventional models. The current model manifests its advantages over the classical ones by more reasonable explanation of the mechanism of deposition and erosion due to the cohesiveness of fine particles. One may find that the result predicted by the new model is very similar to the field measurements. Nevertheless, some areas seem to be slightly over-estimated. This is mainly due to an over evaluation of the input fine sediment. As has mentioned before, the input SS data may include some particles whose sizes are out of the fine sediment range.

Bed elevation (m)

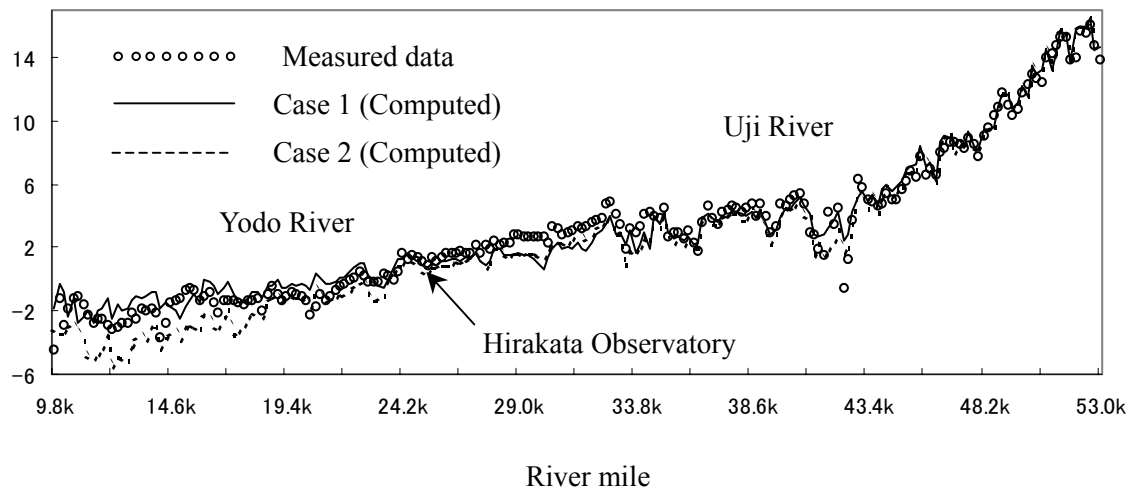


Fig. 5.8 Bed Elevation of the Yodo River in 1998

A more detailed comparison is made for the water stage variation in 1998 at Hirakata Observatory due to the shortage of the corresponding bed level data. This is shown in Fig. 5.9, which covers the water level variation during both dry seasons and flood periods in 1998. One can find that the new model case including fine sediment reproduces the water level variation quite well, which significantly humbles its counterpart. Although the water stage variation is not directly caused by the sediment transport, the reasonable prediction of the water stage implies a reasonable prediction of the riverbed here. This may be explained with Fig. 5.10.

Water stage (m)

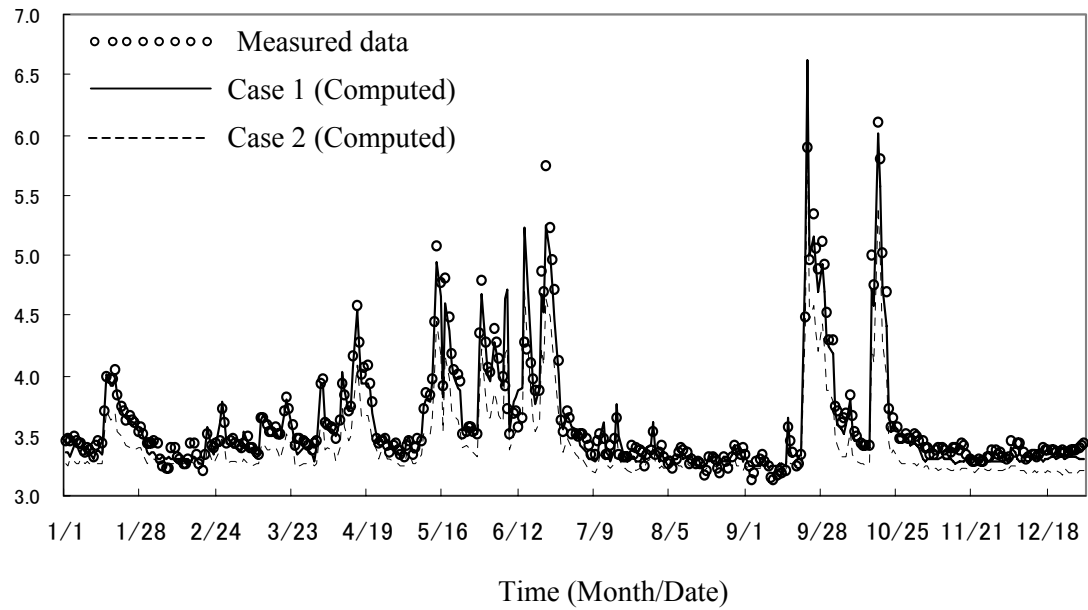


Fig. 5.9 Water Stage at Hirakata Observatory, 1998

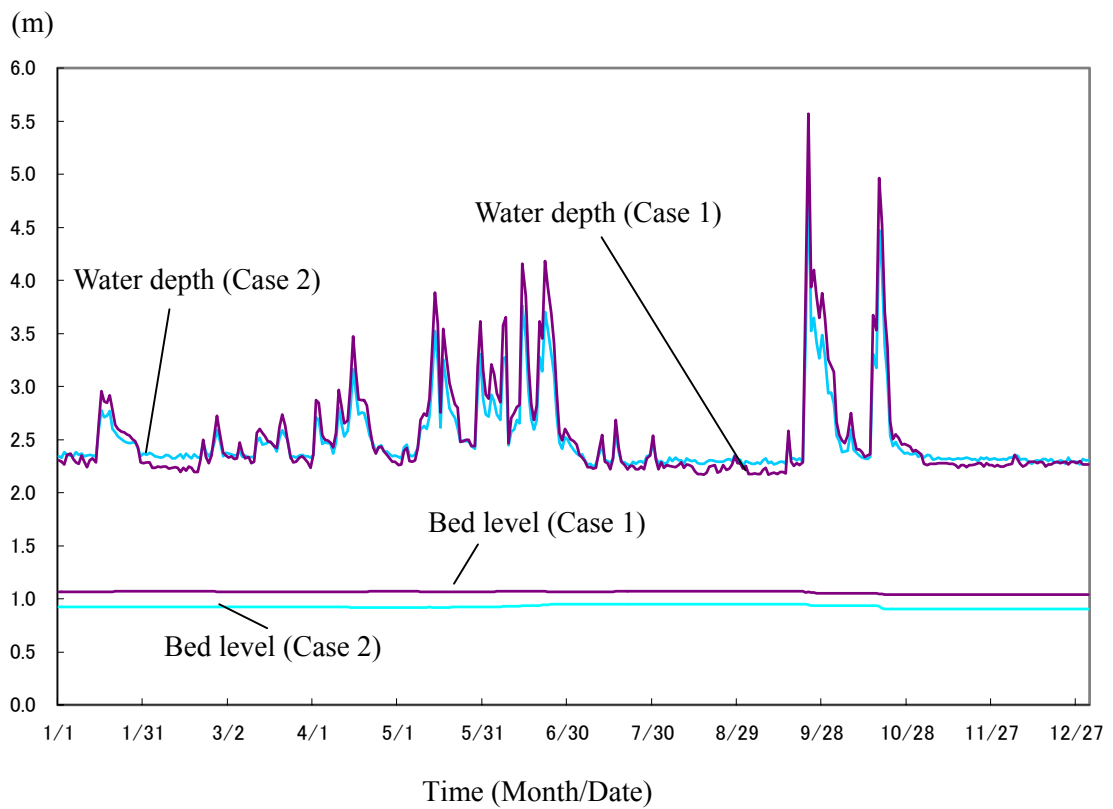


Fig. 5.10 Bed Level and Water Depth Variation at Hirakata Observatory, 1998

In Fig. 5.10, the calculated bed level and the water depth are plotted in the same figure. It is found that the water depth predicted by the two computational cases in the whole year is very similar. This indicates that the differences in the water stage are stemming from the differences in the bed level. And one may also find that there do exist differences between the two cases in the predicted bed levels. Hence, one can conclude that it is the better reproduction of the bed level that leads to the better result for the water stage.

5.3 Bed deformation due to a series of non-submerged spur dykes

In the previous section, the fluvial process of a river system is reproduced with a 1D model, from which, one can conclude that the model is capable of predicting the transport of mixed sediment with both coarse and fine sediment. From this section, the model will be applied to predict the local flow and sediment transport around spur dykes.

A series of spur dykes are commonly used in the river engineering as has mentioned before. In chapter 2, the behavior of different kinds of spur dykes has been investigated experimentally. In this section, the numerical methods will be used to simulate the spur dyke related problems in the laboratory flume.

One of the experiments carried out by Muto et al. (2003) is tested herein. The experiment is conducted in a straight compound channel with a slope 1/700 as shown in Fig. 5.11. The model channel is made of wood. Ten successive embayments are formed by equipping 9 spur dykes in the flood plain area. The initial riverbed is covered by 10cm-thick artificial sands with a mean diameter of $d=1.34\text{mm}$. The sieve analysis result of the sediment sample is shown in Table 5.4. The density of the sediment particles is 2.24g/cm^3 , which is a bit lighter than natural sands.

Table 5.4 Sediment Sieve Analysis

Size (mm)	0.410	0.500	0.609	0.743	0.906	1.104	1.346	1.641	2.000
Cumulative (%)	0.04	0.41	1.79	4.85	9.00	26.48	52.67	78.88	100

Table 5.5 Experimental Conditions

Discharge	Water Depth	Mean Velocity	Reynolds Number	Froude Number
8.23 l/s	4.30 cm	28.57 cm/s	12,285	0.44

The experimental conditions are given in Table 5.5. The spur dykes are non-submerged. After

a continuous running of 24 hours, the bed seems to be unchanged and it is assumed as an equilibrium state. The riverbed deformation is then measured with a laser sensor after the flume has been completely drained out. Cements were utilized to fix the final riverbed for the measurement of the velocity field. An I-shape and an L-shape electromagnetic velocimeters were employed to measure the flow velocities.

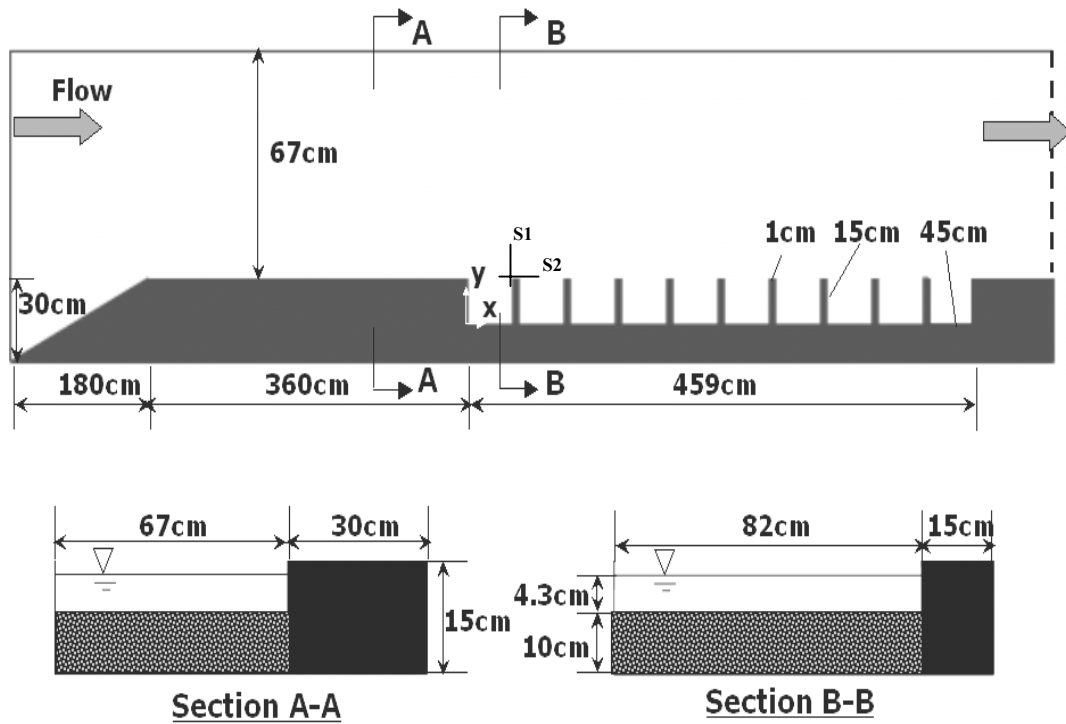


Fig. 5.11 Experiment Setup

Considering the computational convenience, the computed domain does not completely coincide with the experimental geometry. It begins $10b$ (i.e. 150cm, here b is the length of the spur dyke) upstream from the first embayment and extends to $20b$ (i.e. 300cm) downstream from the last embayment. As is seen in Fig. 5.11, there is a 180cm gradually narrowing stretch in the upstream area in this experiment. It intends to have some influence on the approaching flow. However, it is assumed at the current stage that the turbulent flow has been fully developed in the main stream. This assumption might have some impacts on the values of the simulated result, but it does not affect the generality of the conclusion.

The longitudinal velocity along the vertical direction is assumed to satisfy the logarithmic profile at the inlet. The outlet flow boundary is considered as a zero gradient boundary. A total

number of 14,196 hexahedral mesh is generated for the computation. The standard $k-\varepsilon$ model is implemented. A steady-state calculation is carried out on the flat bed at the beginning. The resulted velocity field serves as the initial condition for the movable bed computation. The mesh system changes its location according to the bed variation, but the total number of the mesh is unchanged.

The final bed variation around the first two and the last two spur dykes at the equilibrium condition is depicted in Fig. 5.12 and Fig. 5.13, respectively. The intervals of the contours are 0.70cm and 0.50cm, respectively.

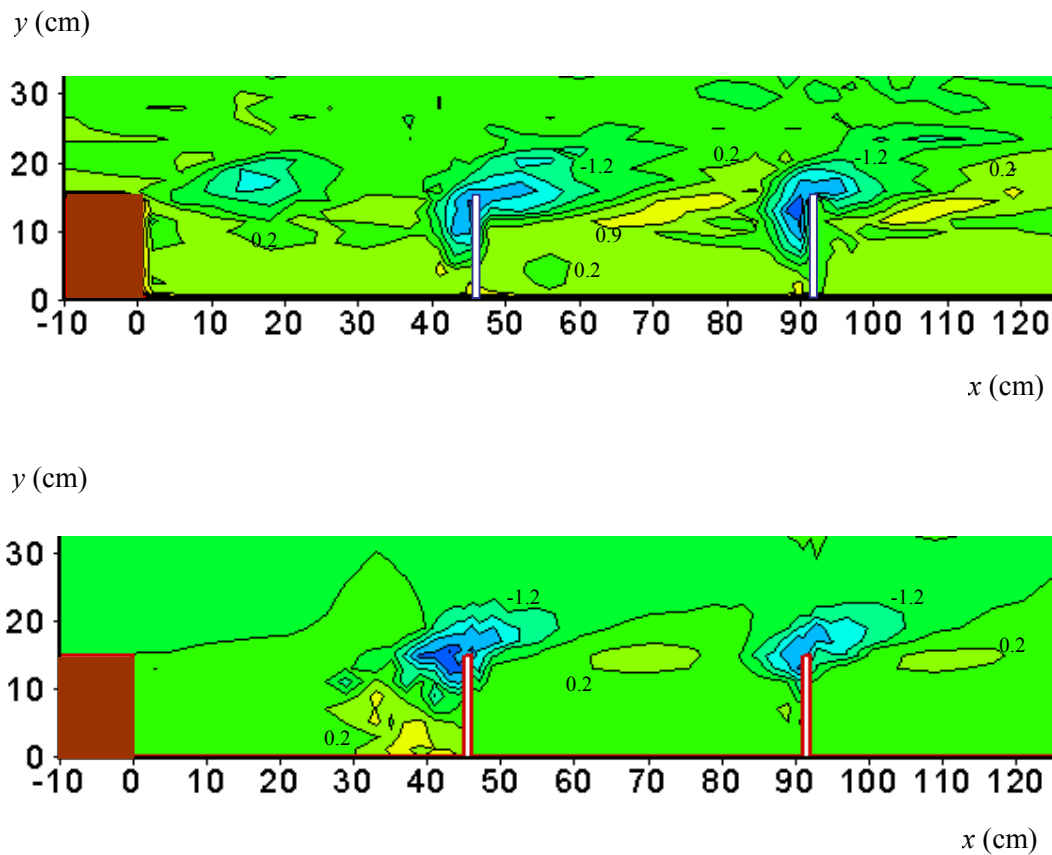


Fig. 5.12 Bed Deformation around the First Two Spur Dykes
(Experiment: Top; Computation: Bottom)

Although there are some differences between the computation and the experiment, the result is encouraging. Local scour holes occur at the toes of all the spur dykes. This can be found in both experimental measurements and computational results (Fig.5.12 and Fig.5.13). Moreover, the geometries of the scour holes are generally similar if one takes a careful inspection. The deepest scour holes take place in the vicinities of the first and the second spur dykes. According

to the measurement data, the values are 3.69cm and 4.37cm, respectively. In the computation, these values are predicted as 4.03cm and 3.68cm, respectively. There is a smaller scour a little downstream from the front of the first embayment in the experiment. This may be due to the sudden widening of the channel just at the front of the first embayment. But this phenomenon is not resolved in the computation. It indicates that the shear stress has been under-estimated in the junction zone (i.e. the area between the main channel and the embayment). On the other hand, a large area of deposition has been predicted along the bank in the corner area of the first embayment, which is not observed in the experiment. It demonstrates an over-estimation of the shear stress. The limitation of the $k-\varepsilon$ model and the relatively coarse mesh may be responsible for these differences.

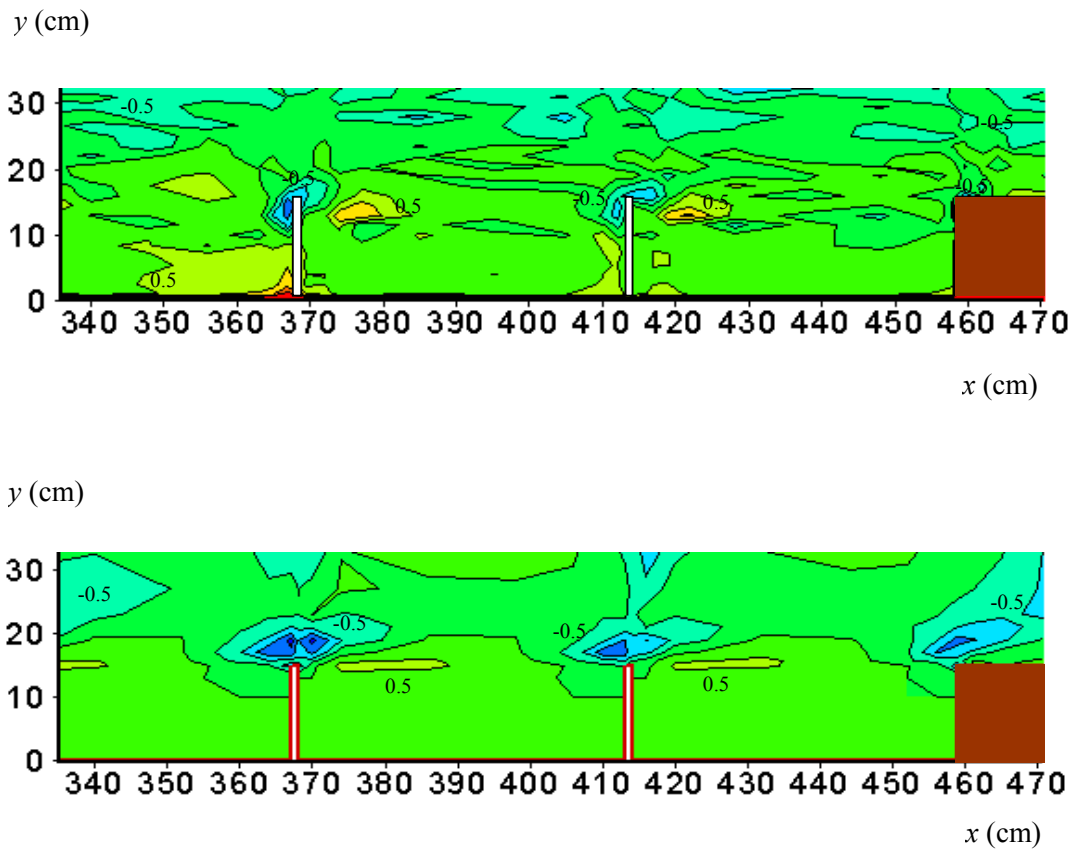


Fig. 5.13 Bed Deformation around the Last Two Spur Dykes
(Experiment: Top; Computation: Bottom)

From the third spur dyke to the final one, the maximum depth of the scour hole is much smaller compared with the first two and the scour pattern is also quite similar. So the bed deformation around the spur dykes at the central area of the group is not plotted here. Only the

final two are shown in Fig. 5.13. Since there exist many similarities between the downstream spur dykes in a group, it seems no need to organize as many spur dykes as possible in a group from the viewpoint of a cost-effective solution. This has also been found in the experimental research in Chapter 2, although the impermeable spur dykes in Chapter 2 are directly exposed to the main flow while in this case they are equipped in a large embayment.

The local scour pattern around the first spur dyke is further analyzed in Fig. 5.14.

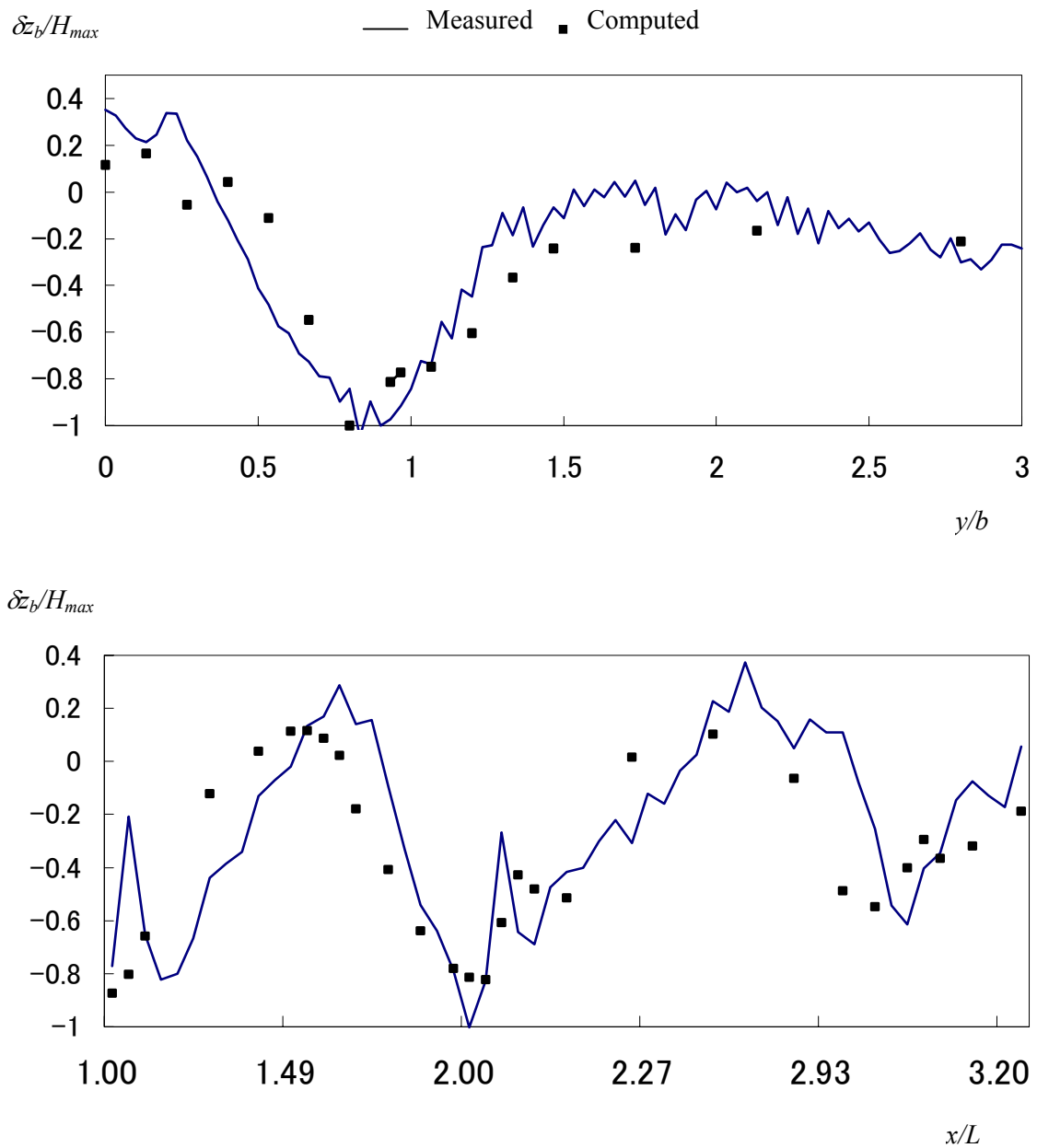


Fig. 5.14 Local Scour Hole Profile around the First Spur Dyke
(Transverse section S1: Top; Longitudinal section S2: Bottom)

Fig. 5.14 shows the local scour hole profiles along two representative cross-sections. The location of the two sections is illustrated in Fig. 5.11. They are transverse and longitudinal sections near the head of the first spur dyke (Section S1 is defined by $x/L=1$, in which L is the length of the embayment, and Section S2 is defined by $y/b=1$). The value of the bed deformation δz_b has been normalized by the maximum scour depth $H_{\max}$ at the corresponding section. One may find that the numerical result is in good agreement with that of the experimental data quantitatively.

Finally, the comparison of the velocity vectors (u, v) at depth $z=2.3\text{cm}$ from the datum level is shown in Fig. 5.15. Only the first three embayments are presented here due the fact that the flow around the remaining embayments is quite similar to the third one.

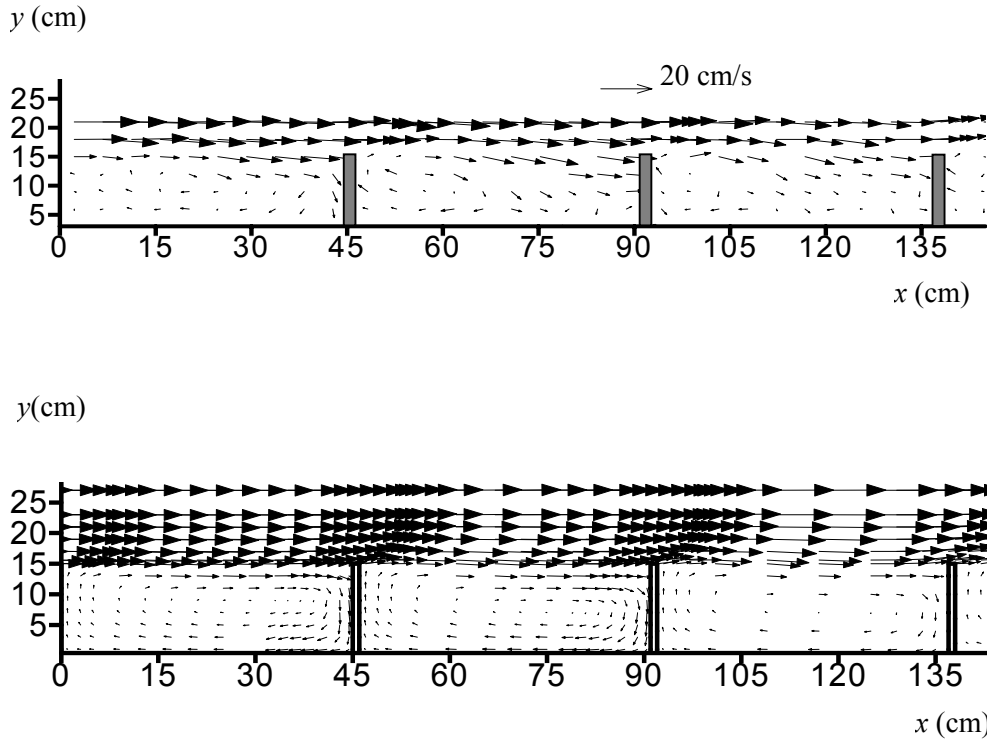


Fig. 5.15 Velocity Vectors at $z=2.3\text{cm}$ (Experiment: Top; Computation: Bottom)

The similarity of the flow pattern between the measurement and the computation is evident. The fundamental aspects of the flow field have been reasonably captured although the mesh adopted here is relatively coarse. The flow diverts at the heads of the spur dykes, which results in extreme lateral velocity gradient. The longitudinal velocity is still dominated in the main channel area. In the embayment area, horizontal vortices are obviously observed, but the

measured velocity seems to be a little larger than the computed result. In-between the embayment and the main channel area there is a mixing zone where mass and momentum exchanges occur. The lateral velocity in the mixing zone is also observed to be a bit under-estimated. This may also give an explanation why the predicted bed morphology in the mixing zone (Fig. 5.12 and Fig. 5.13) is simpler than that plotted from the experimental data. The simplification of the input flow and the error introduced by the calculated final bed topography might be responsible for this discrepancy.

5.4 Local scour around a submerged spur dyke

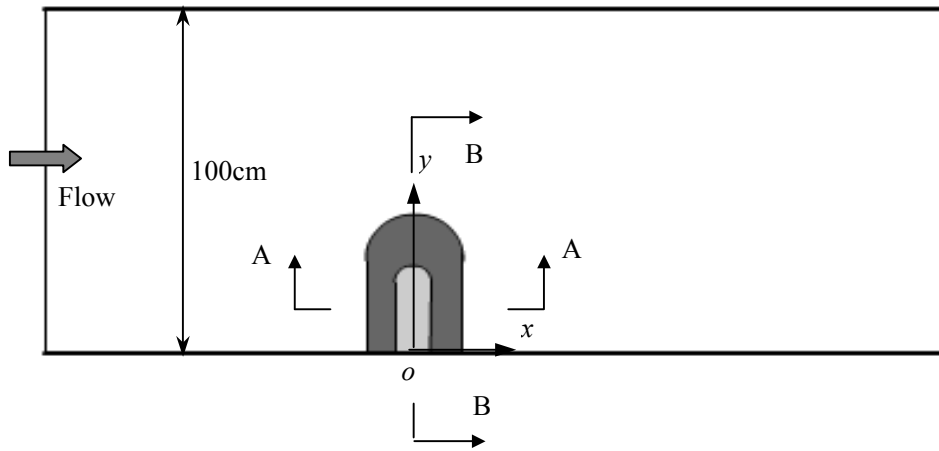
Hydraulic structures such as spur dykes usually suffer from overtopping flow during the flood season, which is of great interests in engineering practices. The overtopping flow is usually associated with greater shear velocities and changes in the local scour holes. Hence, information on the submerged spur dykes is very useful.

Ishigaki and Baba (2004) investigated the local scour induced by the flow around both non-submerged and submerged spur dykes experimentally. Two kinds of spur dyke were tested in the experiments: an attracting type and a deflecting type. The spur dyke model was set on one side of the channel. The channel was 10m long, 1.0m wide and 0.3m deep, which was equipped with a discharge control system. A 1.8m long movable bed was located in the middle part of the channel. It was filled with 0.2m-thick sand with the mean diameter of 0.26mm. The experiment setup and hydraulic conditions for one of the submerged deflecting spur dyke are shown in Fig. 5.16 and Table 5.6, respectively. The shape and the size of the spur dyke are also depicted in Fig. 5.16.

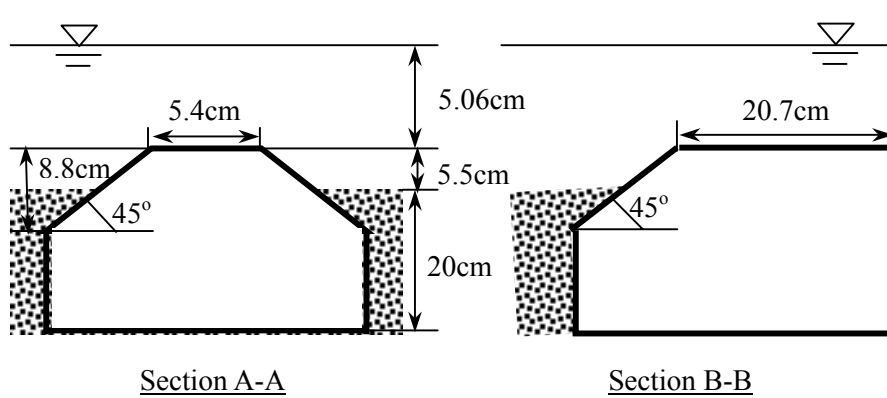
Table 5.6 Experimental Conditions

Discharge	Water Depth	Mean Velocity	Reynolds Number	Froude Number
21.12 l/s	10.56 cm	20.0 cm/s	21,120	0.20

The overtopping ratio (water depth to the spur dyke height) in this verification case is 1.92. The ratio between the friction velocity and the critical friction velocity of the bed material is 0.78. Hence, this case is a typical clear water scour. Measurements were carried out after one hour from the beginning of the experiment. The laser level meter is used for the measuring of the bed morphology, an ultra-sonic level meter is employed to record the water level and an electro-magnetic velocimeter is adopted to collect the velocity field after the riverbed is fixed with cement powder.



Top view of the experiment setup



Section A-A

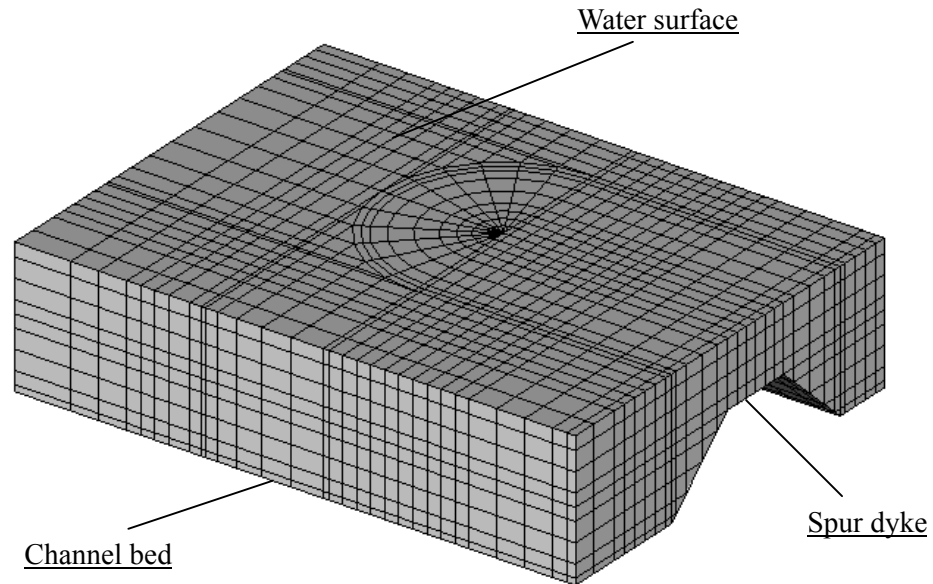
Section B-B

Fig. 5. 16 Experiment Setup for the Submerged Spur Dyke

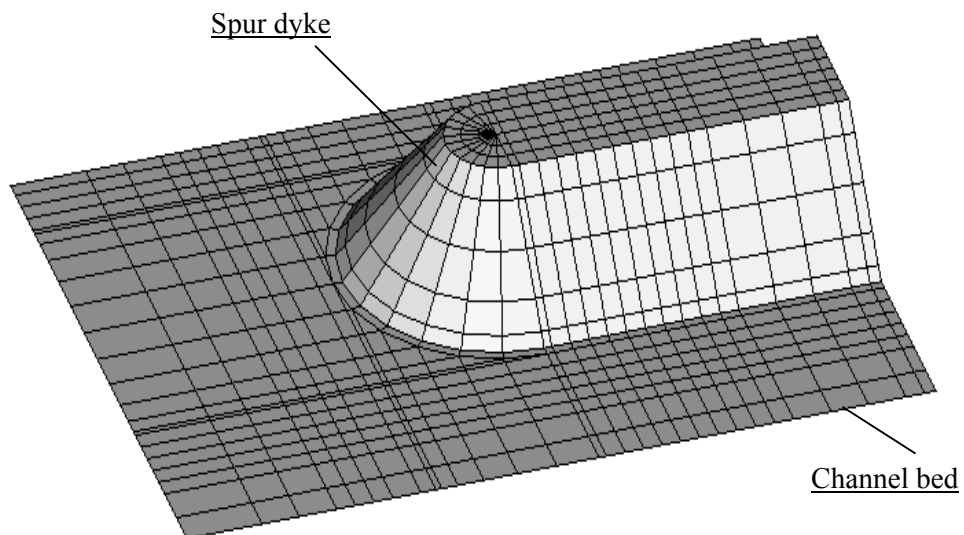
This simulation involves strongly 3D turbulent flow, complex but important hydraulic structure and movable irregular riverbed. Moreover, all those aspects are interacted with each other. The predominant structured mesh methods in river engineering are not suitable in this case. As a result, successful calculation of this kind of problem has been rarely completed to the authors' knowledge.

In order to diminish the boundary effect, the inlet boundary and the outlet boundary are selected far enough from the spur dyke in the computation. According to the experiences acquired from the previous fixed bed and movable bed calculations, the inlet boundary is chosen to be $10H$ (H is the water depth) upstream from the spur dyke and the outlet boundary is $20H$ downstream of the spur dyke. A total number of 28,944 polyhedral mesh is used. The

unstructured mesh consists of both hexahedra and prisms in order to resolve the study domain accurately, in particular the shape of the spur dyke. The mesh system around the spur dyke is shown in Fig. 5.17.



A Bird's View of the 3D Mesh Around the Spur Dyke



Mesh on the Surface of the Bed and the Spur Dyke

Fig. 5.17 Initial Mesh System around the Spur Dyke

In the computation, a logarithmic velocity profile in the vertical direction is assumed at the inlet boundary. The turbulent quantities k and ε are specified corresponding to a viscosity ratio of 10.0 and taking the turbulence intensity 8%.

Either the experiment or the computation shows that the significant bed deformation only occurs around the spur dyke area. Fig. 5.18 is the comparison of the bed variation near the spur dyke with the same scale.

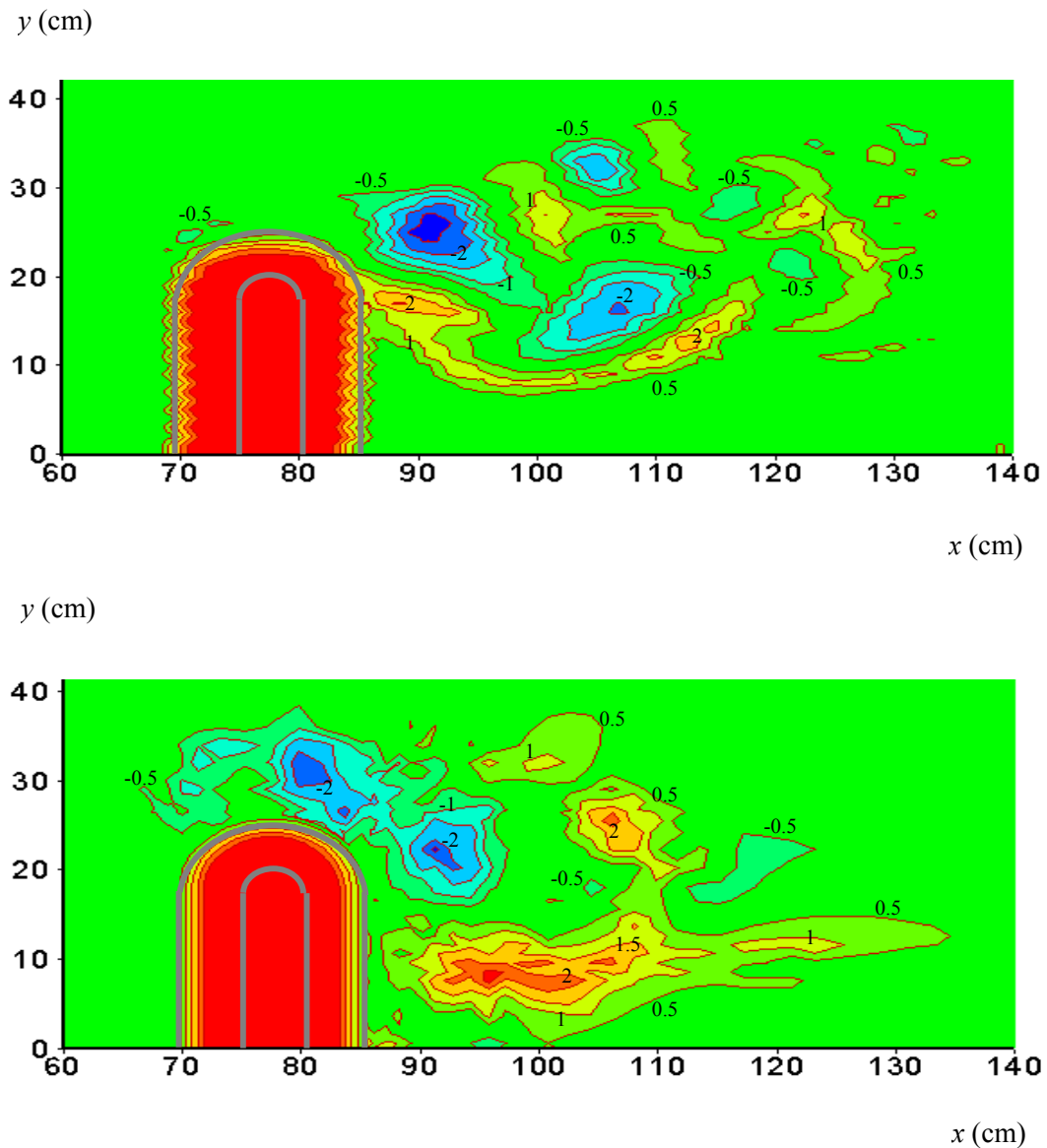


Fig. 5.18 Bed Variation around the Spur Dyke after 1 Hour
(Experiment: Top; Computation: Bottom)

In the experiment, there are two big scour holes as shown in Fig.5.18 (top). One is located near the downstream head of the spur dyke with a depth of 2.78cm. Another one is in the downstream side of the spur dyke, which has a depth of 2.05cm. The calculation also gives two obvious scour holes around the spur dyke, the depths of which are 2.71cm and 2.33cm, respectively. The positions are depicted in Fig. 5.18 (bottom). It may be also observed that the predicted locations are a little more upstream than the measured ones. This may indicate that there are some differences in the distribution of the shear stress around the head of the spur dyke. The deposition pattern downstream of the spur dyke shows some similarity as well. In the shade of the spur dyke, a belt of deposition is found surrounding the two deep scour holes. However, the belt is narrow in the experiment. While in the computation, this deposition belt is relatively wide and extends to the channel bank. The bed maintains unchanged near the bank in the experiment, but in the computation, bed evolution occurs. These seem imply that the shear stress near the bank has been over-estimated. The inherent defects of the standard $k-\varepsilon$ model may be responsible for these differences. If the area downstream of the spur dyke is considered as an embayment, the aspect ratio of this embayment is infinity. As has already discussed in the fixed bed calculation in Chapter 3. The standard $k-\varepsilon$ model in this case may be not so accurate. Nevertheless, if the uncertainties involved in the movable bed experiment are taken into account, such conclusion may be drawn that the basic scour-deposition pattern has been reasonably reproduced in this simulation. As the bed variation is caused by the dynamic feedback process between the flow and bed morphology, the analyses of the corresponding flow field is also very important to understand the model performance.

Another comparison is the velocity vectors (u , w) at the points in a plane at $z=6.25\text{cm}$ (i.e.1cm from the top of the spur dyke). It is noted that the computed velocity components have been interpolated to the points measured in the experiment for clarity (see Fig. 5.19). In the whole domain, the longitudinal velocity is predominant in this layer. Nevertheless there are some inflections in the proximity of the spur dyke if one takes a careful look at the experimental result (Fig. 5.19, top). This is due to the shape effect of the spur dyke. It may be found that the flow over the spur dyke is accelerated near the head. A downward flow occurs after the flow passes the spur dyke. The same phenomenon has been predicted by the calculation (Fig. 5.19, bottom). But the predicted longitudinal velocity is a bit smaller than the measurement. It is also noted that the transverse velocity near the head is slightly toward the bank in the experiment while it is a bit outward the bank in the computation. As the flow structure and the bed morphology are closely interacted with each other. It is very difficult to analyze quantitatively the accuracy of them separately. However, if one can have a look at Fig. 5.18 and Fig. 5.19 connectedly, one may find more confidence in the prediction capability of the current model.

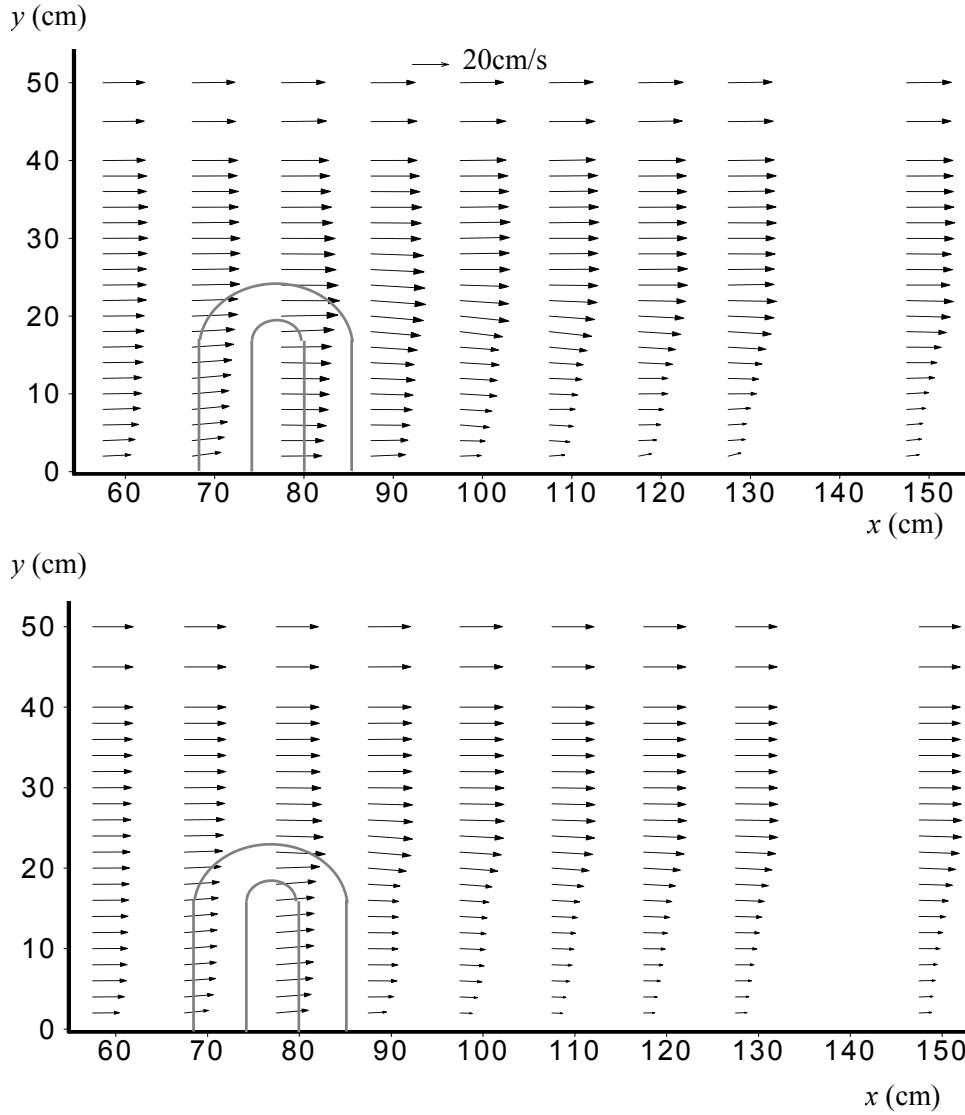


Fig. 5.19 Velocity Vectors (u, w) at $z=6.5\text{cm}$ (i.e. 1cm over the Spur Dyke Top)
(Experiment, Top; Computation: Bottom)

Finally, the computed velocity vectors (u, v) in a horizontal plane $z=2.0\text{cm}$ is plotted in Fig. 5.20. This layer is very close to the riverbed and is important for the understanding of the local scour. Due to the shortage of data, the comparison with experimental result has not been fulfilled here.

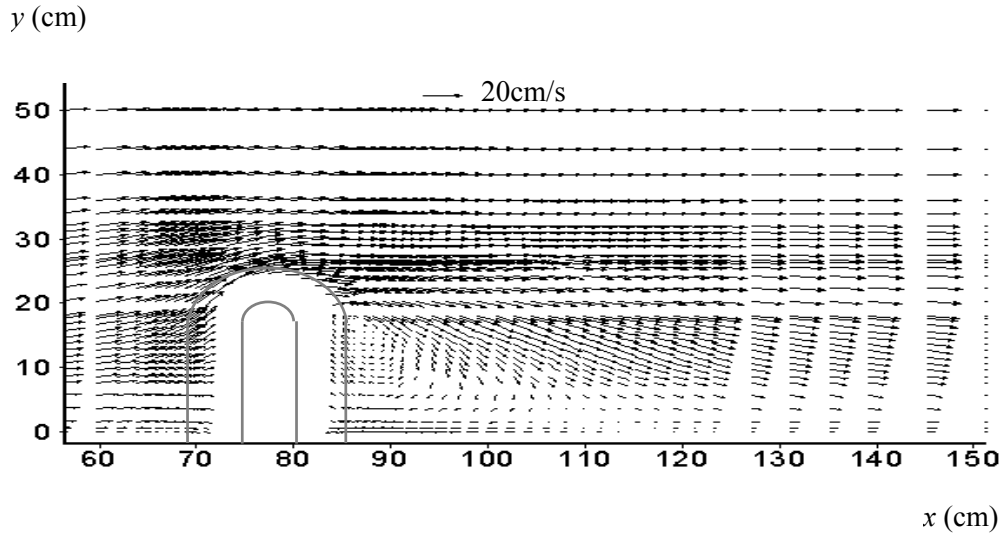


Fig. 5.20 Computed Velocity Vectors (u, v) at $z=2.0\text{cm}$

The flow velocity diverts at the head of the spur dyke. Behind the spur dyke, a horizontal vortex is evidently observed. At the center of the vortex, the flow is almost stagnant. These observations have also been found in the similar experiment research of submerged groins carried out by Kawaguchi et al. (2004). This also demonstrates that the numerical model provides an effective way to understand the details of the flow structure and some of which may be very difficult to resolve with experimental methods.

5.5 Conclusions

In this chapter, the proposed models have been applied to simulate the flow and bed evolution in channels with spur dykes in both field and laboratory conditions. As has mentioned before, there are many possibilities to combine the hydrodynamic models and sediment transport models in different levels. It is impossible and unnecessary to test all those combinations. Hence, this chapter validates the models to the best of the available data as well as takes into account the practicability of the models in the future.

The first application is the reproduction of the fluvial process of the Yodo River system in West Japan. In this application, the 1D flow model is integrated with the sediment models for mixed sediment transport. Due to a great number of spur dykes along the river bank, significant

amount of fine sediment has been found in the riverbed. The new model and a conventional model based on non-cohesive sediment analyses have been implemented under the same computational conditions. The conventional model significantly over-estimates the sediment erosion and leads to a great bed degradation in the area where fine sediment has a relatively high percentage in the bed composition. Nevertheless, the new model gives a quite close result to the field measurement. The calculated bed level shows no much difference between the two models in the area where the bed sediment is relatively coarse. These observations imply that the transport of fine sediment plays an important role in the bed shaping process. The new model considers the influence of the fine sediment on the bed level, the bed composition and the bed cohesiveness and is able to reflect more physics. It is then believed that this model may serve as a promising tool in the investigation of large-scale river systems, in particular when fine sediment is of interest.

In the second application, the model is applied to predict the bed evolution in a model flume with a series of non-submerged spur dykes. The bed morphology at the equilibrium state is found to be reasonably reproduced by the 3D model. And the details of the flow structure are also well captured. Finally, the local scour around a spur dyke under over-topping flow is computed with the new 3D model. In this test case, the shape of the spur dyke is relatively complex. A model based on an unstructured mesh may be the best solution. The flow pattern and the local scour after 1 hour are simulated with the numerical model. The model result is in good agreement with the limited experimental data. From the numerical simulation, many details on the flow structure have been observed, which have not been measured in the current experiment but have been found in similar experiments carried out by other researchers. The model used in both cases incorporates the standard $k-\varepsilon$ model and the bed load transport models for sloping beds. The model results indicate that the model is able to predict the dynamic feedback process involving the flow, sediment and bed. It may be also concluded that the numerical methods provide an effective way to understand the details of the local flow structures and scour processes, some of which may be very difficult to resolve if not impossible with experimental methods.

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Chapter 6

Conclusions and recommendations

This study concerns the flow and bed evolution in channels with spur dykes. Both experiments and numerical simulations have been carried out with different kinds of spur dykes. The main work during this study is summarized as below.

(1) Completed two experiments for a series of impermeable and permeable spur dykes

Since very few quantitative information is available concerning a series of spur dykes. Two experiments have been conducted with non-submerged impermeable and permeable spur dykes organized in sequences. This effort has two objectives: (a) further the understanding of the flow structure and bed degradation induced by a series of impermeable and permeable spur dykes and (b) further the understanding of the influence on the flow and river system after the construction of a group of spur dykes and the difference with that induced by a single spur dyke.

(2) Developed a turbulence model based on 3D unstructured mesh

This model solves the Reynolds-averaged Navier-Stokes equations with a standard $k-\varepsilon$ model and some non-linear $k-\varepsilon$ models (optional) for the turbulence closure. Arbitrary polyhedral mesh up to six faces may be used for the calculation to resolve the complex geometries. The mesh is permitted to move in the vertical direction in order to capture the moving boundaries such as the riverbed.

(3) Proposed a total sediment load transport model

Due to the presence of spur dykes, fine sediment (i.e. the fine portion of suspended load) exchanges with the local bed and the riverbed consists of a mixture of both coarse and fine particles. These phenomena are not able to be modeled with the conventional models. New methods are proposed to model the transport of suspended load and the behavior of the riverbed considering the cohesiveness of fine particles. Furthermore, the Ashida-Michiue formula for bed load transport has been extended from a horizontal bed to a sloping bed in view of the local scour.

(4) Integrated sediment transport models with hydrodynamic models in 1D and 3D

Employment of different levels of morphological models may provide a cost-effective solution for engineering practices. A 1D model is suitable for the long-term prediction of river systems, and a 3D model is capable of resolving the local bed variation. A 1D flow model has been integrated with the total sediment load transport model and manifests the ability to predict the long-term bed evolution of the Yodo River system in West Japan. A 3D morphological model including bed load transport has also been completed and verified with experimental data.

6.1 Conclusions

The conclusions of this research are summarized as below.

(1) Flow structure and bed evolution due to spur dykes

In Chapter 1, the flow structure and the local scour mechanism around a single spur dyke have been discussed. The local scour is initiated and controlled by the complex flow structure which is characterized by several vortex systems. The flow pattern around a single spur dyke may be separated into four components: (a) the downward flow ahead of the spur dyke; (b) the bow wave when the flow approaches the spur dyke; (c) the primary vortex in the local scour area and (d) the wake vortices behind the spur dyke.

In a series of spur dykes, the flow pattern becomes a bit more complex. A single spur dyke has only local effect on the flow and the river system. But a group of spur dykes will lead to some kind of artificial narrowing of the river width besides the local obstruction effect. As a result, a group of spur dykes results in both local scour and main channel degradation (due to intensified velocity in the main channel). However, the effect of the local obstruction is still predominant for the most upstream spur dyke in a group. This can be observed in both impermeable and permeable experiments. If the most upstream spur dyke is set a little far from the downstream one, it may be considered as a single spur dyke. It happens in the current impermeable experiment. The most upstream embayment is too long and the most upstream spur dyke behaves like a single spur dyke, which can be observed from the erosion of the riverbed along the bank by strong return currents. It is evident that the bank may be protected if the length of the embayment is prudently selected. Furthermore the most upstream spur dyke directly affects the immediately following one by disturbing the flow and sediment transport. As a result, the flow structure and the bed deformation around the immediately following spur dyke are strongly dependent on the most upstream one. From the third spur dyke, a relatively stable state is achieved. The intensified velocity in the main channel due to the channel narrowing governs the main channel degradation, while the local scour around spur dykes is a result of

both the local obstruction and the channel narrowing.

The influence on the river by grouped impermeable spur dykes is different from permeable ones. Impermeable spur dykes are passive structures, which lead to great flow disturbances. Extreme local scour holes are found at the toes of the spur dykes. Existing return currents from the heads of the spur dykes toward the channel bank have a potential to erode the bank if the distance between two consecutive spur dykes is not close enough. The flow field shows obviously 3D characteristics and the resulted riverbed morphology is very complex. Permeable spur dykes exhibit more active behaviors, which can reasonably control the bank-parallel flow pattern within the embayment area and the local scour around the piles. The permeable spur dykes allow the main channel degradation by the intensified main channel velocity and near-bank deposition due to the reduced velocity in the embayment. The flow field in the whole domain has 2D characteristics and the final riverbed morphology is relatively simple.

(2) Development of numerical models to predict flow and bed evolution due to spur dykes

A morphological model has been developed to simulate the flow and bed evolution due to the construction of spur dykes, which includes a hydrodynamic module, a total sediment transport module and their integration. The proposed model is able to simulate two key problems of most engineering interest and significant meaning, i.e. the local scour and the transport of fine sediment. Numerical modeling of these two phenomena has been seldom completed to date.

In the hydrodynamic module, a 3D turbulence model based on an unstructured mesh is proposed. As the study domain is movable due to the bed variation and complex due to the spur dykes and local scour. Implement of conventional structured mesh methods is quite difficult if not impossible. While the unstructured mesh methods are very attractive. This model solves the Reynolds-averaged Navier-Stokes equations with the standard $k-\varepsilon$ model and some non-linear $k-\varepsilon$ models for the turbulence closure. The main points during the construction of an applicable model with unstructured mesh are summarized here: (a) an effective data structure for the restoring of the mesh system in particular the connectivity between the control volumes; (b) suitable discretization methods which have been confirmed to be efficient and stable; (c) necessary treatments for the mesh skewness which is almost unavoidable for unstructured mesh methods and (d) inclusion of mesh movement in the governing equations which makes it possible to track the moving boundaries. This model is applied to simulate some selected laboratory experiments. It is found that the results of the unstructured methods are comparable to those of structured ones. The verifications also show that the model is capable of predicting flows in different geometries with different meshing strategies.

The total sediment load is divided into bed load and suspended load. A modified Ashida-Michiue formula is used for the bed load modeling. In order to account for the bed slope

effect especially in the scour area, the threshold condition for the sediment entrainment is corrected with a new bed slope factor and a part of the particle gravity is incorporated into the effective shear stress. The bed slope factor is derived based on a 3D force analysis, which is a function of the bed slope, angle of sediment repose and the flow direction. Compared with conventional treatments, this method is more reasonable and can be used in the cases when the magnitudes of the flow velocity components are comparable. A sand slide algorithm is also introduced to avoid the unrealistic excess slope over the angle of repose during the calculation.

The transport of suspended load takes time and space, which is simulated by solving the diffusion-convection equations. Fine sediment has been paid special attention. Fine sediment in this thesis refers to the fine portion of suspended load and consists of cohesive particles. Construction of spur dykes is found to make it possible that fine sediment deposits and exchanges with the local bed. Two aspects below have been considered in this thesis. Firstly, fine sediment moves as flocs in the water column. Secondly, deposition of fine sediment may alter the bed cohesiveness and the bed cohesiveness will affect the sediment erosion process. The deposition and erosion of fine sediment follow the well-known formulae in the cohesive sediment research field, i.e. the Krone's approach for deposition and the Partheniades's formula for erosion. The influence of the bed cohesiveness on the erosion process is treated as follows: (a) the bed is considered to consist of a non-cohesive particle cluster surrounded by a cohesive force field; (b) the erosion process is a two-stage process: sediment is firstly eroded from the non-cohesive bed and then passes through the cohesive force field and (c) the former is evaluated with non-cohesive based formulae and the latter is quantified as a linear function related to the clay content in the bed surface. This idea is based on the results of experiments concerning the bed cohesiveness in the literatures.

(3) Capability of the numerical model

The proposed hydrodynamic module and the sediment module are integrated and constitute the morphological model, which is finally applied to predict the flow and bed evolution in both field and laboratory conditions. The proposed model is applied in 1D to simulate the 23-year bed evolution process of the Yodo River system in West Japan. The result has been compared with a conventional model excluding the effect of fine sediment. The proposed model gives quite close result to the field measurement and significantly humbles the conventional one. It indicates that fine sediment should be taken in account in rivers with spur dykes in order to simulate the bed variation more accurately. The proposed model is promising and may serve as a powerful supplement in the investigations.

The bed deformation due to the bed load transport around a series of non-submerged spur dykes and a submerged spur dyke is also modeled with the proposed numerical methods. The

flow structure and the local scour pattern calculated by the proposed model are in reasonable agreement with those of the experiments. It indicates that the current model is applicable. It is also noted that the proposed model provides an effective way to understand the details of the local flow structure and scour processes, some of which may be very difficult to resolve if not impossible with experimental methods.

Although the tentative application of the proposed model has been the prediction of the flow and bed evolution in channels with spur dykes, the model may find a wider use in the hydraulic engineering. This model can serve as a promising tool in the investigation of flow and sediment related problems with its capability of treating complex geometries and movable boundaries, together with the transport of fine-coarse sediment mixtures. The important restrictions of the current model are the unchangeable free surface and the mesh movable only in the vertical direction.

6.2 Recommendations for future researches

(1) Turbulence models

The 3D unstructured mesh based turbulence model presented in this thesis is of great meaning to simulate hydraulic engineering flows in complex geometries. However, such improvements are very important before it could be used with confidence: (a) Free surface variation: In the current model, the free surface is assumed to be a symmetrical plane. This may be questionable if the flow surface has a significant change. A module calculates the surface variation should be introduced, for instance, the widely used VOF (Volume of Fluid) method may be an alternative; (b) Wall boundary treatment: The current model employs the standard wall function approach to resolve the wall-bounded area. This can avoid the possible integration through the viscous sub-layer and include the wall roughness more flexibly. However, as the sediment transport is very sensitive to the near wall flow field, it sometimes becomes a main error source of the simulation. Some more accurate methods should be introduced in the near wall area; and (c) Other improvements: These may include possible improvements of the numerical schemes adopted in the model and a high quality organization of the source codes, etc. The non-linear $k-\varepsilon$ model is sometimes unstable with the current unstructured mesh methods, which may be caused by the discretization methods.

(2) Sediment transport models

As sediment transport is strongly related to the flow field. In this thesis, it has been assumed that the flow model is accurate enough so that the verification of the sediment transport models is possible. There seems a long way to go before a model is acceptable. In the current stage,

some directions for the future researches may include: (a) Sediment sorting process: The 3D model is only valid for uniform sediment transport for the time being. But the actual river is generally symbolized by a wide spectrum of sediment sizes. So the sediment sorting algorithm in the 1D model should be extended to the 3D case. However, the verification data should be guaranteed before this work starts; (b) Bed cohesiveness: In this thesis, the effect of the bed cohesiveness on the erosion process has been expressed by a simple linear function related to the clay content in the bed surface. But this relation is very idealized. In order to be more reasonable, further researches on the bed cohesiveness should be carried out and (c) Cohesive sediment transport formulae: Fine sediment is treated as cohesive sediment and the behavior is modeled with the frequently used formulae correspondingly. However, the existing cohesive transport formulae are generally derived in entirely cohesive sediment conditions. In the river engineering, cohesive and non-cohesive particles are mixed, and very common the non-cohesive sediment is the majority. The applicability of cohesive sediment transport formulae needs to be validated.

(3) Model verifications

Verification and calibration of the numerical models are of crucial importance before the models can be used to solve problems in actual rivers. In this thesis, the main function of the models has been verified with limited field and laboratory data. Some more verifications and calibrations are suggested, these include: (a) Verification of the turbulence model with finer mesh and more complex geometries; (b) Verification of the 3D morphological model with both bed load and suspended load; (c) Verification of the cohesive sediment transport formulae with mixed fine-coarse sediment as has mentioned before and (d) Verification of the bed cohesiveness on the erosion process. As the behavior of the cohesive sediment is generally more sophisticated and problem-dependent, experiments are suggested for the verification of (c) and (d).

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List of Symbols

Roman Symbols

a_1, a_2, a_3	Coefficients used in the non-linear $k - \varepsilon$ model	
a_f, a_P, a_{nb}	Coefficients of the discretized governing equations for the unknown quantity at the center of the surface, the present CV and the neighboring CV	
A	Cross-sectional area of the channel in a 1D model	M ²
b	Width of the embayment (i.e. length of the spur dyke)	M
b_P, b_A, b_f	Source terms excluding the unknown quantity for the present CV, the adjacent CV and the surface between the present and the adjacent CVs	
B	Channel width	M
B_0	Constant in the expression of the roughness parameter, $B_0 = 5.2$	
B_b	$B_b = B_0 - 8.5 + \ln k_s^+ / \kappa$	
B_s	Sediment deposition width in 1D model	M
c	Massive sediment concentration	KG/M ³
c_V	Centroid of the CV	
C	Volumetric sediment concentration ($c = \sigma C$)	
C'	Grain-related Chezy coefficient	M ^{1/2} /S
C_a	Sediment concentration at the reference level	
C_D	Coefficient of the drag force	
C_e	Equilibrium sediment concentration at the reference level	
C_f	Concentration of flocs (fine sediment)	
C_k	Sediment concentration of size fraction k	
C_L	Coefficient of the lift force	
$C_{S\Omega}$	Parameter used in the non-linear $k - \varepsilon$ model	
C_μ	Coefficient used in the $k - \varepsilon$ model	
$C_{1\varepsilon}, C_{2\varepsilon}$	Coefficient used in the $k - \varepsilon$ model, $C_{1\varepsilon} = 1.44, C_{2\varepsilon} = 1.92$	
d	Sediment diameter	M
d_k	Sediment diameter of size fraction k	M
d_m	Sediment mean diameter	M
D_f	Diffusion conductance; Deposition flux of flocs (fine sediment)	
D_k	Deposition flux of size fraction k	M/S
E	Erosion rate of cohesive sediment	M/S

E_f	Empirical cohesive erosion rate	M/S
E_k	Erosion flux of size fraction k	M/S
E_r	Roughness parameter	
f	Center of the surface	
f'	Intersection of the surface and the line connecting two neighboring CVs	
f_b	Possibility of sediment passing through the cohesive force field	%
f_i	The i th component of the body force	M/S ²
f_x, f_y, f_z	Direction cosines of the fluid force	
F	Fluid force	N
F_D	Drag force	N
F_f	Strength of the convection at the surface	M ³ /S
F_L	Lift force	N
F_R	Resulted driving force	N
F_s	Stabilizing force	N
g	Gravitational acceleration	M/S ²
G	Rate-of-production of turbulence kinetic energy	M ² /S ³
G_p	Rate-of-production of turbulence kinetic energy at the present CV	M ² /S ³
h	Water depth	M
H	Total energy head	M
$H_{\max}$	Deepest local scour hole depth	M
i	Component of the Cartesian coordinate; Index	
I	Turbulence intensity	
I_e	Energy slope	
j	Component of the Cartesian coordinate	
J	Energy gradient in the 1D model	
$\bar{J}$	Averaged energy gradient between consecutive sections in the 1D model	
k	Turbulence kinetic energy; Component of the Cartesian coordinate; Sediment size fraction	M ² /S ²
k_0	Ratio constant using in Eq. (4.38), $k_0 = 0.025$	
k_s	Equivalent roughness height	M
k_s^+	$k_s^+ = u_* k_s / \nu$	
K	Bed slope factor	
K_1	Coefficient used in the floc settling velocity formula	M ⁴ /KG/S
K_2	Coefficient used in the floc settling velocity formula	M ³ /KG
K_l	Longitudinal bed slope factor	
K_t	Transverse bed slope factor	

l	Unit vector in the direction of the line connecting the centers of two neighboring CVs	
L	Length of the embayment	M
m	Time step; Coefficient used in the flocc settling velocity formula	S
m_1	Coefficient used in the flocc settling velocity formula	
m_e	Erosion parameter	M/S
n	Iteration step; Normal direction of a surface	
n_d	Total number of the deposited layers	
N_{ij}	Non-linear terms in the Reynolds stress expression	M/S ²
p	Pressure, $p = p^* + p'$	N/M ²
p^*	Pressure not satisfying the continuity equation	N/M ²
p'	Pressure correction	N/M ²
p_P, p_A, p_f	Pressure at the center of the present CV, the adjacent CV and the common surface	N/M ²
p_{bk}	Percentage of size fraction k in the riverbed surface	%
p_c	Critical clay content in the bed surface	%
p_{clay}	Clay content in the bed surface	%
p_{ek}	Percentage of size fraction k in suspended load	%
p_{tk}	Percentage of size fraction k in the transition layer	%
$p_{n_d k}$	Percentage of size fraction k in the n_d th deposited layer	%
q	Lateral inflow	M ² /S
q_{bk}	Bed load transport rate of size fraction k	M ² /S
q_{bkx}, q_{bky}	Bed load transport rate of size fraction k in x and y directions	M ² /S
q_{bl}, q_{bt}	Bed load transport rate in the longitudinal and transverse directions	M ² /S
P_f	Ratio of the strength of the convection to the diffusion conductance	
Q	Water discharge	M ³ /S
Q_f	Fine sediment discharge at the inlet	M ³ /S
$\mathbf{r}_c$	Center of the CV	
$\mathbf{r}_i$	The i th vertex of the CV ($\mathbf{r}_i = x_i \mathbf{i} + y_i \mathbf{j} + z_i \mathbf{k}$)	
r^0	Initial residual	
r^n	Residual of iteration step n	
R_*	$R_* = \sqrt{(s-1)gd^3} / \nu$	
R_{ep}	Particle Reynolds number, $R_{ep} = w_s d / \nu$	
s	Specific gravity of sediment, i.e. $s = \sigma / \rho$	

s_P	Part of the source term for the present CV	
S	Area of a surface	M^2
S_f	Area of the CV surface	M^2
S_{fx}, S_{fy}, S_{fz}	Projected area of the CV surface	M^2
$S_{k\varepsilon}$	Strain parameter	
t	Time	S
t_a	Thickness of the active layer	M
t_{di}	Thickness of the i th deposited layer	M
t_t	Thickness of the transition layer	M
T	Bed shear stress parameter	
u	Velocity component in the x direction; longitudinal velocity	M/S
u^+	Dimensionless velocity near the wall	
$\bar{u}$	Depth averaged velocity	M/S
u_*	Friction velocity	M/S
u_{*ck}	Critical friction velocity for size fraction k	M/S
u_{*e}	Effective friction velocity	M/S
$u_{ }$	Near-wall velocity	M/S
$u_{f\perp}$	Velocity at the surface center and normal to the surface	M/S
u_P, u_A, u_f	u at the center of the present CV, the adjacent CV, the CV surface	M/S
u_i	i th component of the velocity field, i.e. (u, v, w) and $u_i = u_i^* + u_i'$	M/S
u_i^*, u_i'	Velocity not satisfying the continuity equation and the velocity correction	M/S
u_m	Velocity of the mesh movement	M/S
v	Velocity component in the y direction	M/S
V	Volume of the CV	M^3
w_f	Settling velocity of the floc	M/S
w_p	Vertical velocity variation of the particle	
w_s	Settling velocity of the sediment particle	M/S
W	Submerged weight of a particle	N
x	Component of the Cartesian coordinate	M
y	Component of the Cartesian coordinate	M
y^+	Dimensionless distance from the near-wall CV to the wall	
$y_{\perp}$	Distance from the near-wall CV to the wall	M
z	Component of the Cartesian coordinate	M
z_0	Datum level	M
z_a	Reference level, i.e. the interface between the bed load and suspended load layers	M
z_b	Bed level	M

Z	Water surface elevation	M
Z_s	Suspension indicator, $Z_s = w_s / (\kappa u_*')$	

Greek Symbols

α	Energy coefficient; Direction angle of a surface with a normal vector $\mathbf{n}$	
α_i	Energy coefficient at cross-section i	
α_e	Empirical coefficient in Eq. (4.45)	
α_f	Weighted function during the surface interpolation	
α_ϕ	Relaxation coefficient	
β	Direction angle of a surface with a normal vector $\mathbf{n}$	
γ	Direction angle of a surface with a normal vector $\mathbf{n}$	
Γ	Diffusion coefficient	
$\Gamma_P, \Gamma_A, \Gamma_f$	Diffusion coefficient at the present CV, the adjacent CV and the common surface	
δ_{ij}	Kronecker delta	
δz_A	Change of the elevation for a node A on the bed surface	M
δz_b	Bed variation during one time step	M
δz_{bk}	Bed variation due to size fraction k	M
δz_B	Change of the elevation for a node B on the bed surface	M
Δ	Height of the bed form	M
ΔB	Roughness function defining the shift of the intercept due to the roughness effect	
Δt	Time step	S
Δx	Spatial step in x direction	M
ε	Dissipation rate of the turbulence kinetic energy	M ² /S ³
ε_P	Dissipation rate of the turbulence kinetic energy at the present CV	M ² /S ³
ε_{sx}	Sediment diffusive coefficient in the longitudinal direction	
ε_t	Tolerance in the equation solver	
ζ	$\zeta = w_p / \sigma_p$, used in Eq. (4.38)	
ζ_0	$\zeta_0 = w_s / \sigma_p$, used in Eq. (4.38)	
θ_l	Longitudinal bed slope	
θ_t	Transverse bed slope	
κ	von Karman's constant (=0.41)	
λ	Sediment porosity	
Λ	Length of the bed form	M

ν	Molecular kinematic viscosity of the fluid	M^2/S
ν_t	Eddy viscosity	M^2/S
ρ	Density of the fluid	KG/M^3
σ	Density of the particle	KG/M^3
σ_c	Turbulent Schmidt number	
σ_k	Parameter in the $k - \varepsilon$ model, $\sigma_k = 1.0$	
σ_P	Standard deviation of the vertical velocity variation of a sediment particle	
σ_ε	Parameter in the $k - \varepsilon$ model, $\sigma_\varepsilon = 1.3$	
τ_b	Near bed shear stress	N/M^2
τ_c	Critical shear stress	N/M^2
τ_{c0}	Critical shear stress on a horizontal bed	N/M^2
τ_{cd}	Critical shear stress for deposition	N/M^2
τ_{cek}	Critical erosive shear stress for size fraction k	N/M^2
τ_g	Grain-related shear stress	N/M^2
τ_{ij}	Reynolds stress	N/M^2
τ_s	Bed shear strength varying with depth	N/M^2
τ_w	Wall shear stress	N/M^2
τ_*	Dimensionless shear stress	
τ_{*c}	Dimensionless critical shear stress	
τ_{*ck}	Dimensionless critical shear stress for size fraction k	
τ_{*ek}	Dimensionless effective shear stress for size fraction k	
τ_{*k}	Dimensionless shear stress for size fraction k	
ϕ	General conserved quantity	
ϕ_P, ϕ_A, ϕ_f	Quantity ϕ of the present CV, the adjacent CV and the common surface	
φ	Angle of sediment repose	
Ψ	Bed-form steepness	
Ω	Rotation parameter	

Acknowledgements

I would like to gratefully acknowledge the enthusiastic supervision of Prof. Hajime NAKAGAWA. His forethoughtful guidances, continuous encouragement and supports in a wide aspect made my experience at Kyoto University extremely meaningful, both academically and personally. I was also deeply indebted to Prof. Guangqian WANG, my supervisor at Tsinghua University, who gave me the opportunity to get involved with sediment and hydraulics. His enthusiasm, inspiration and patience were strongly impressed on my memory. Whatever I learned from the two supervisors will be invaluable treasure throughout my life.

I would like to extend my sincere gratitude and appreciation to Prof. Taisuke ISHIGAKI. It seemed as if it happened yesterday that he detailedly explained me all the facilities in the Ujigawa Hydraulics Laboratory when I first came to Japan. And he always encouraged me, helped me stayed motivated and clarified my understanding even after he moved to Kansai University. I was also grateful to Dr. Bingyi LIU (TEPIA Corporation, Japan). He recommended me to study in Japan and was always kindly to provide me instructions and suggestions in many aspects amidst his busy schedule. I benefited much from both his expertise and personality.

I also wanted to express my sincere gratitude to Dr. Yasunori MUTO, Dr. Yasuyuki BABA and Dr. Tetsuo UENO for their valuable advice, wisdom and endless patience in both research and many other things.

I appreciated the academic and moral support from Dr. Ming LU (Hydro-soft Technology Institute, Osaka), Dr. Haoming WANG (TEPIA Corporation, Japan), Dr. Songdong SHAO (University of Plymouth, UK), Dr. Md. Munsur RHAMAN (Bangladesh University of Engineering and Technology) and Dr. Yanliang DU (University of Tübingen, Germany) throughout this work.

I have made many friends during the stay in the Ujigawa Hydraulics Laboratory. In particular, Mr. A.T.M. KHALEDUZZAMAN, Mr. Hirotugu YAGI, Mr. Koichi KITAMURA, Mr. Raj Hari SHARMA, Mr. Naoki ITO and Mr. Daizaburou TOUCHI. They helped me in one way or another to finish this work and made my life joyful during the long and tedious experiment and coding process.

Special thanks also went to the staff in the Disaster Prevention Research Institute, especially Mr. Seiji FUJIHARA, Mr. Hideo TAGAWA, Mr. Yoshinori YOSHIDA, Ms. Yoko KITAGAWA and Miss Akiko KIYONAGA. Without their cooperation and willingness to be inconvenienced, this work would not have been possible.

Last but not least, the scholarship provided by the Monbukagakusho (Ministry of Education, Culture, Sports, Science and Technology, Japan) is sincerely acknowledged.

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