5.7.2 Advances in two-dimensional modeling

5.7.2.1 Simulation of compound channel with vertical drop stuructures

Introduction

Vertical drop spillways are commonly used engineering measures to prevent bed erosion in steep alluvial rivers. They are often installed in river reaches where the channel shape or plan-form has been changed to facilitate the construction of engineering works such as meander cut-offs.

The Toyohira River, Figure 1, which flows through the centre of Sapporo City is important economically and for recreational purposes. Channel hydraulics are complicated by the fact that the cross-section is an engineered compound channel, designed to alleviate local flooding problems. It can also be seen that the river has active morphology with the low flow channel containing alternate bars that pass through the vertical drop spillways.

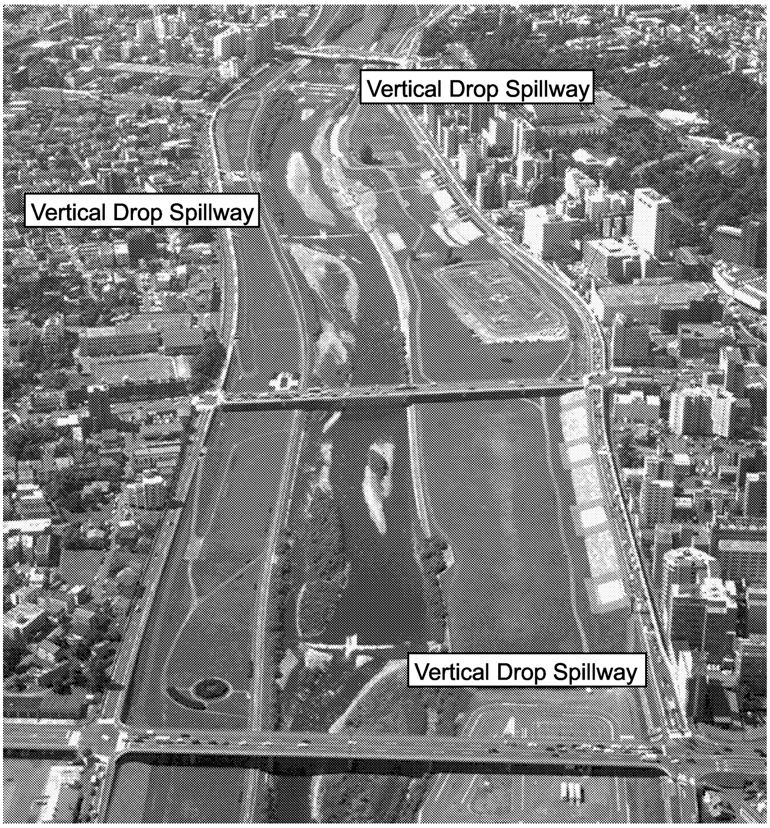

Figure 1: **The Toyohira River** (View from downstream, by Ishikari River Development and Construction Office)

Between 1950 and 1973, eight vertical drop spillways were constructed to control sediment movement [Ishikari River Development and Construction Office (1993)]. These structures significantly improved channel stability, however, the bed slope is still very steep and during floods water velocity can reach 10 m/s, resulting in bed erosion remaining a serious problem, Yamashita et al. (1992). There is therefore a growing desire to re-engineer these structures to further reduce water velocity.

Recent developments of numerical models enable them to be used to predict flow and bed movement in channels with vertical drop spillways, Kawashima and Fukuoka (1995) and Fukuoka et al (1998). However, it is not clear if these models can be applied to the Toyohira River where the section is compound, velocities are high and the bed is very active. To evaluate the various proposals for modifying the existing vertical drop spillways it is important to develop a computer model that can simulate this complex problem.

In this paper, a numerical model is proposed to calculate the flow and sediment transport in compound channels with vertical drop spillways. The basic hydrodynamic equations used are the two-dimensional shallow water equations. These have been linked to sediment transport equations to successfully predict bar formation. However, previously applied numerical methods based on techniques such as upwind differencing are not effective when the channel is compound and has discontinuities such as vertical drop structures. The CIP (Cubic-Interpolated Pseudoparticle) method proposed by Yabe (1990) is used here to calculate the flow conditions in the compound channel and over the abrupt change in level due to the vertical drop spillways. The model is verified by comparison with data from physical model experiments. Results from bank-full and over-bank flows are compared with the predictions from the computer model.

Basic equations and numerical method

The two-dimensional flow field is calculated using Eqs. ???, ??? and ???. The bed shear stresses are calculated using Eqs. ??? and ???. Kinetic eddy viscosity is calculatied smiply by the following.

$$\nu_t = \frac{\kappa}{6} u_* h \tag{1}$$

Here κ is the von Karman constant and u_* is shear velocity. The time dependent change of bed elevation is calculated from the continuity equation for bed load sediment transport.

$$\frac{\partial \eta}{\partial t} + \frac{1}{(1-\lambda)} \left[\frac{\partial q_{bx}}{\partial x} + \frac{\partial q_{by}}{\partial y} \right] = 0 \tag{2}$$

In which λ is porosity of bed material, q_{bx} and q_{by} are bed load sediment transport rate per unit width in x and y directions, which are calculated using the Meyer-Peter Muller formula and Hasegawa's formula (1980), respectively.

$$\frac{q_{bx}}{\sqrt{\left(\frac{\rho_s}{\rho} - 1\right) g d^3}} = 8(\tau_* - \tau_{*c})^{3/2} \tag{3}$$

$$q_{by} = q_{bx} \left(\frac{v}{u} - N_* \frac{h}{r_*} - \sqrt{\frac{\tau_{*c}}{\nu_s \nu_k \tau_*}} \frac{\partial \eta}{\partial y} \right) \tag{4}$$

Here ρ_s and d are density and diameter of bed material, τ_* is the non-dimensional bed shear stress $[=u_*^2/(sgd),\ s = \rho_s/\rho - 1]$, and τ_{*c} is non-dimensional critical shear stress which is calculated by Iwagaki's formula. ν_s and ν_k are static and kinetic friction coefficient of sand particles. The second term on the right hand side of Eq. 9 acts as additional transverse sediment load when the secondary flow is developed. The flow field described in this paper is two-dimensional, however, because of the development of a three-dimensional bed configuration the secondary flow is taken into account when the stream line is curved. A constant value of 7 is used for N_* according to Engelund (1974), and r_* is expressed by the radius of curvature of a streamline, and it is calculated using the following equation suggested by Shimizu and Itakura (1991).

$$\frac{1}{r_*} = \frac{1}{(u^2+v^2)^{3/2}} \left\{ u \left(u\frac{\partial u}{\partial x} - u\frac{\partial v}{\partial x} \right) + v \left(u\frac{\partial v}{\partial y} - v\frac{\partial u}{\partial y} \right) \right\} \tag{5}$$

In solving momentum equations of flow, a solution technique is used, which the equations are separated into two phases, for advection and non-advection. The CIP method is used for advection phase, while the SOR method is used to calculate the non-advection phase coupled to the continuity equation 1. CIP method was originally proposed by Yabe (1990) and modified for the calculation of open channel flow by Nakayama et al. (1998). Change in bed elevation are computed by applying central differncong to Eq. 7.

Moveable bed experimants

Experiments using a 1/50-scale model of the Toyohira River were conducted in a flume with a compound cross-section and vertical drop spillways. Figures 2 and 3 show the outline of the experimental flume. The length and width of the flume are 20m and 1m, respectively. The central part of the channel is a low water channel with a width of 48cm. The low water channel bed is covered with uniform sand of 0.2mm diameter. The flood plains on both sides are cast in mortar. Along the low water channel, three vertical drop structures are installed at 5.3m intervals. The vertical drop of each structure is 1.3cm. Two

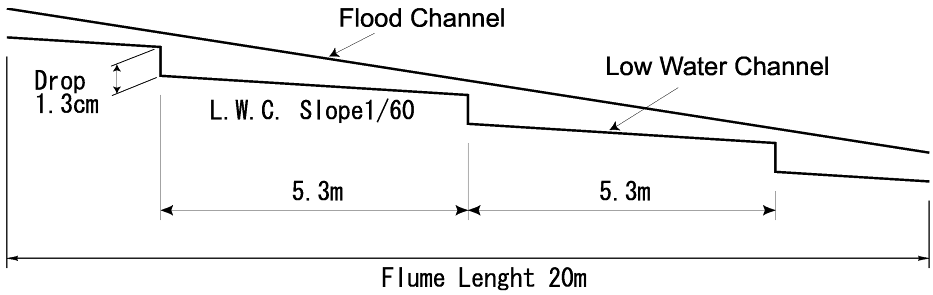

Figure 2: **Longitudinal profile of experimental flume**

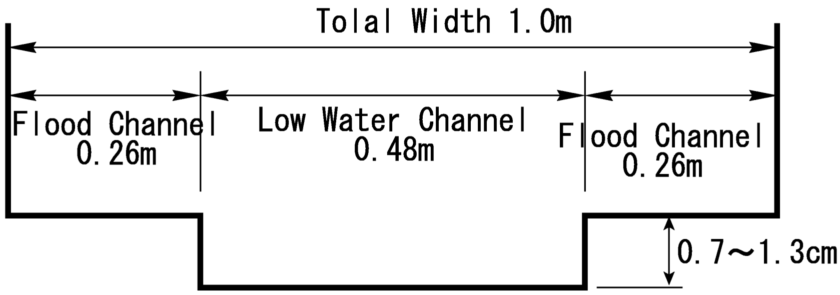

Figure 3: **Cross section of experimental flume**

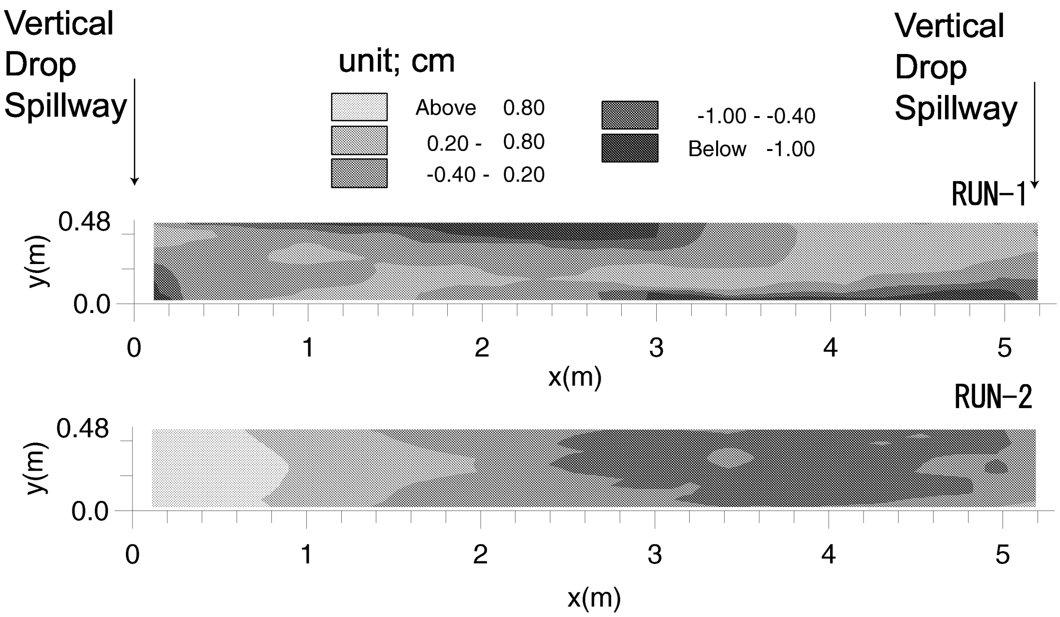

Figure 4: **Bed elevation deviation contour of experiment**

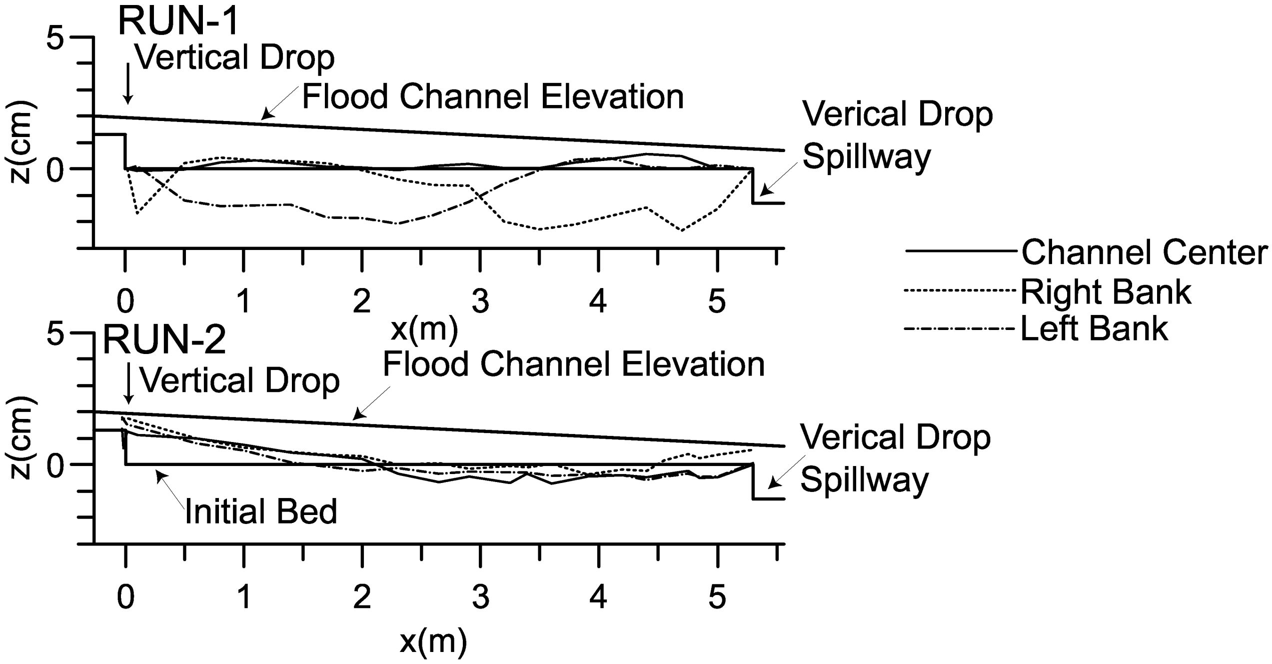

Figure 5: **Longitudinal bed elevation profile of experiment**

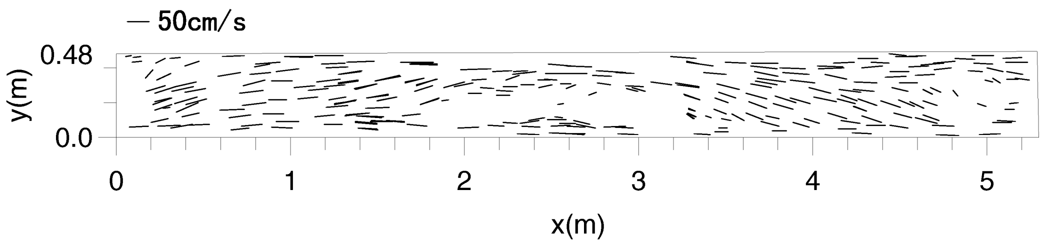

Figure 6: **Velocity vector of experiment**

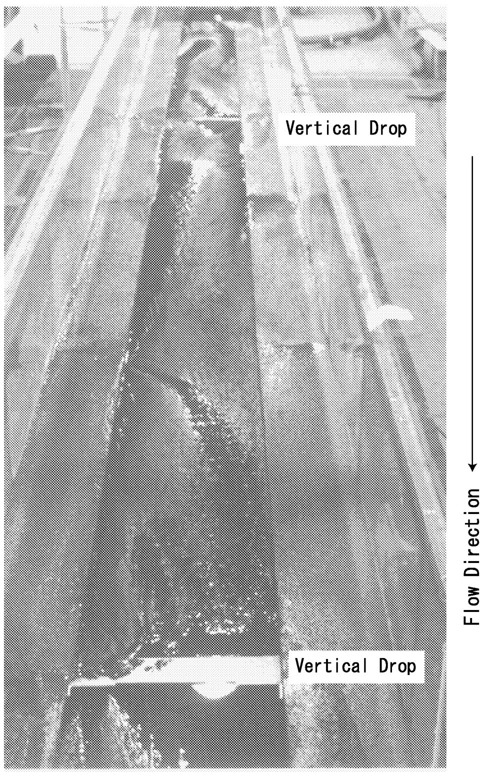

Figure 7: **Photo of Channel Bed (RUN-1, After Experiment)**

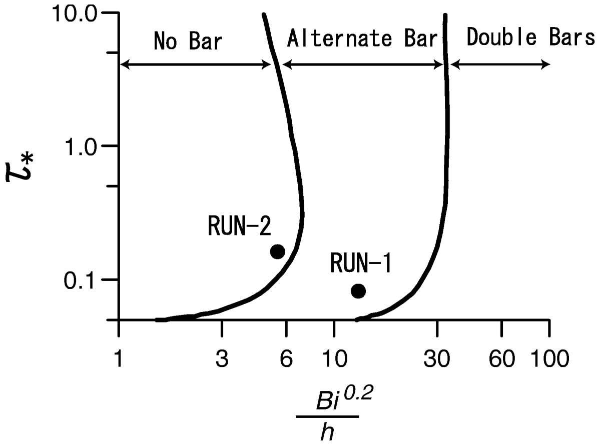

Figure 8: **Regime criteria of meso-scale bar**

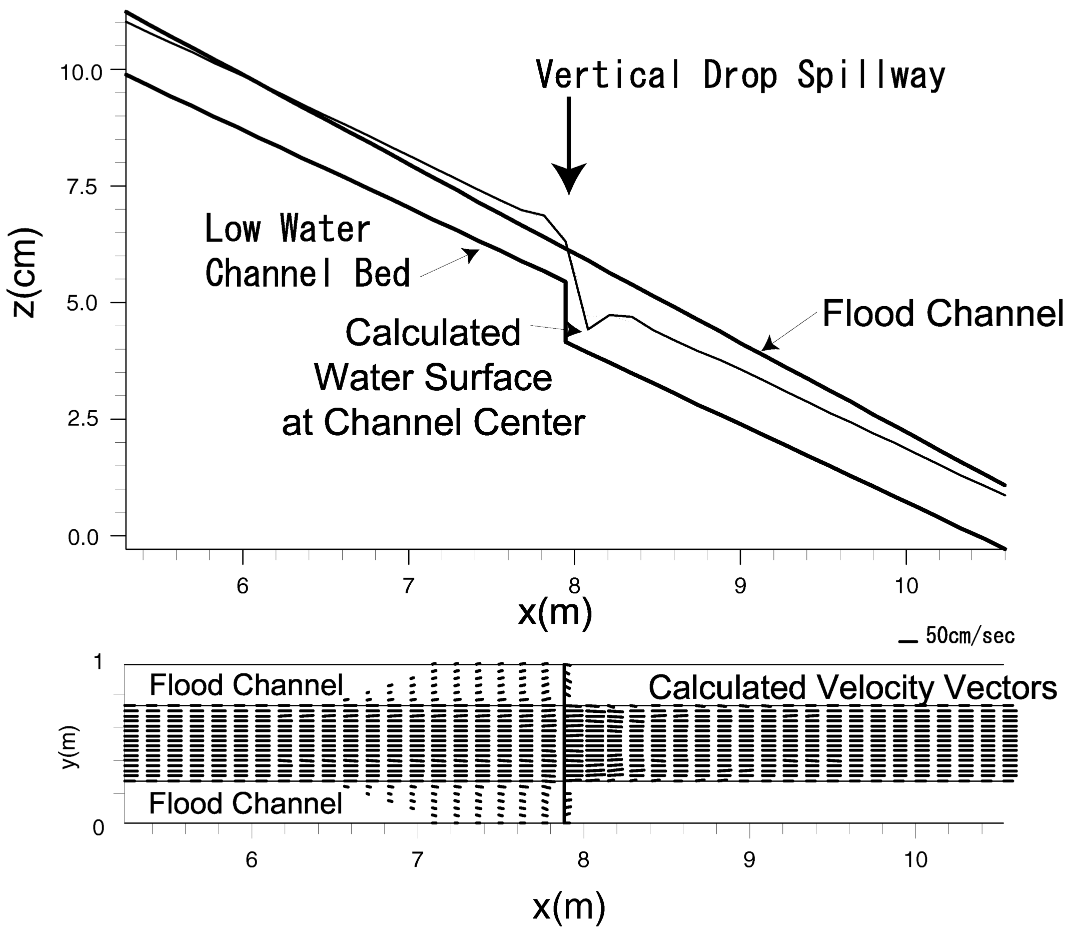

Figure 9: **Calculated results of water surface elevation and velocities (RUN-1, initial bed condition)**

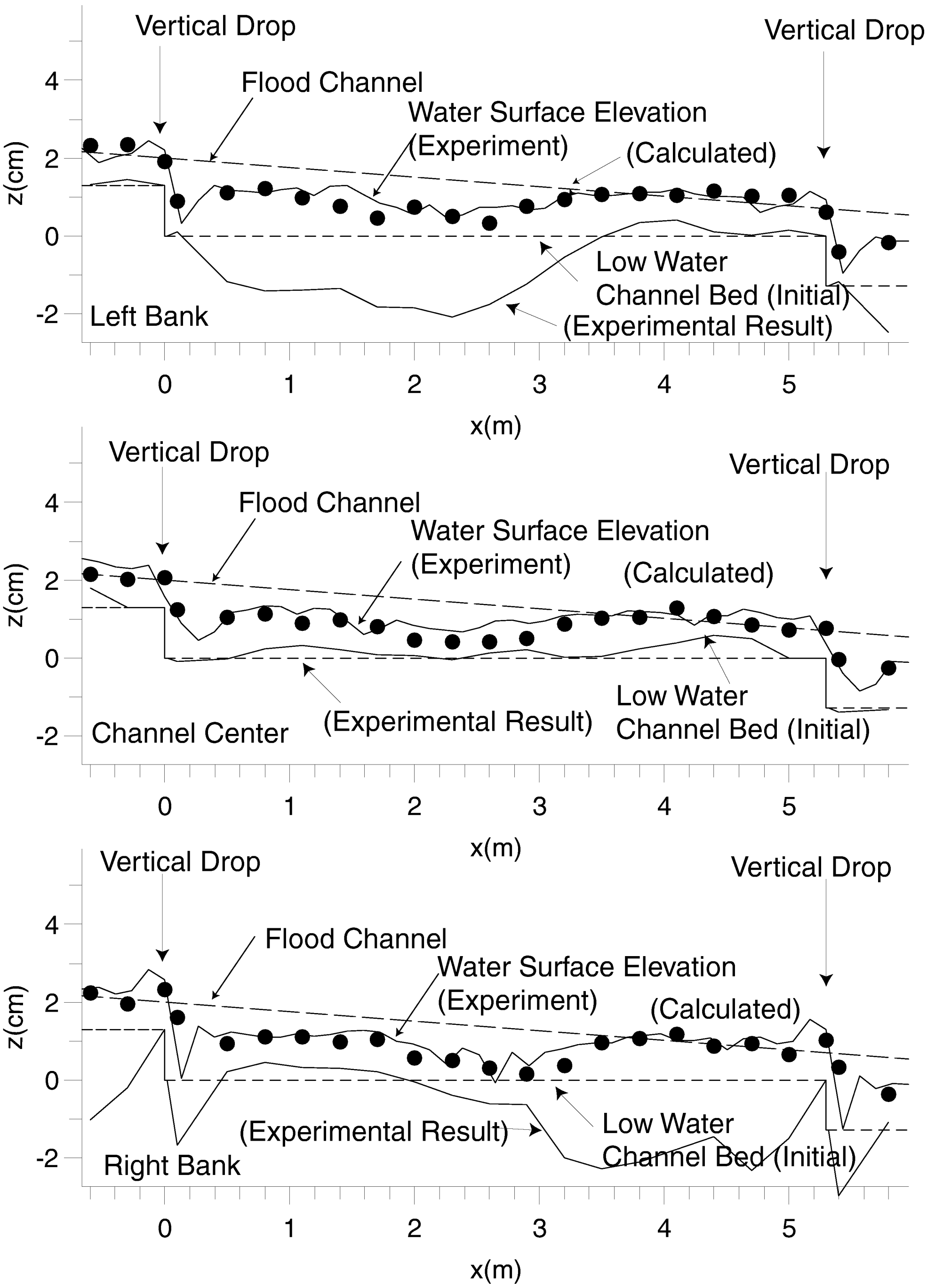

Figure 10: **Comparison of observed and calculated water surface elevation**

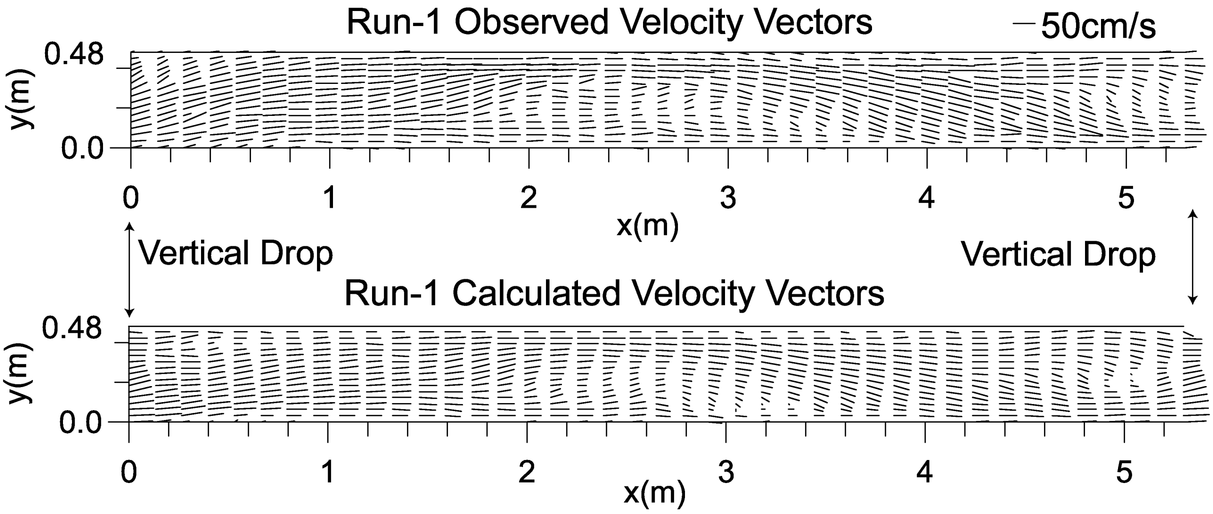

Figure 11: **Comparison of velocity vectors between observed and calculated**

experiments were conducted for bank-full and over-bank flows, with discharges of $2\ell/s$ and $7\ell/s$. These were designated as RUN-1 and RUN-2, respectively.

Sediment was supplied at a rate necessary to keep a constant bed elevation at the upper end of the flume. Experiments were continued for about an hour until the bed elevation reached its equilibrium state. At the end of RUN-1, the surface velocity was measured using PIV technique with Styrofoam grains used as tracers. In both experiments similar bed configurations were observed between the drop structures.

Figure 4 shows the bed configuration (deviation from initial bed), and Figure 5 shows bed elevations in the downstream direction. Figure 6 shows the surface velocity vectors at the final stage of RUN-1.

As can be seen from Figure 7, migrating alternate bars were observed during Run-1. These are also in evidence in the bed elevation data presented in Figures 4 and 5. The low flow water channel was deepest just downstream of a vertical drop, becoming shallower towards the next drop. This caused water to flow onto the flood plain about 2m upstream of the drop. This behaviour is observed in the Toyohira River during the snow melt season.

Alternate bars were not in evidence after RUN-2. The average hydraulic conditions for the two cases are plotted in Figure 8, as proposed by Kuroki and Kishi (10). It can be seen that the existence or otherwise of alternate bars in each experiment agrees with the regime criteria of the diagram.

Numerical calculation

Computer simulations were conducted with conditions similar to those used in the experiments. Manning's roughness coefficient for the mobile bed of the low water channel was determined using a formula proposed by Kishi and Kuroki (1984) for flat beds, a constant value of 0.01 was used for the high water channel. Computational grids in the longitudinal and transverse directions were 122 and 28, respectively. The time step for the computation was set as 0.01 seconds. At the grid points on the vertical drop spillways, where the bed was fixed, calculation of bed elevation change was omitted from the calculation.

Figure 9 shows the calculated water surface profile and velocity vectors under the initial conditions of RUN-1. The flow pattern of partial flooding around the vertical drop spillways observed in the experiments is well reproduced by the computer model. In order to examine the accuracy of the flow calculation, the observed bed elevation at the end of RUN-1 was given and the flow calculation of RUN-1 was conducted with fixed bed conditions. Computed results of water surface and velocity vectors are compared with the observed data in Figures 10 and 11. They show good agreement and thus the accuracy of the flow calculation is verified.

Next, the performance of alternate bar formation was studied using the flow conditions of RUN-1. Calculations without vertical drop spillways were also conducted to compare the effect of vertical drop spillways. Calculations with and without spillways were continued for two hours, the time necessary for the bed configuration to reach equilibrium. Figure 12 and 13 show the calculated bed elevation contours

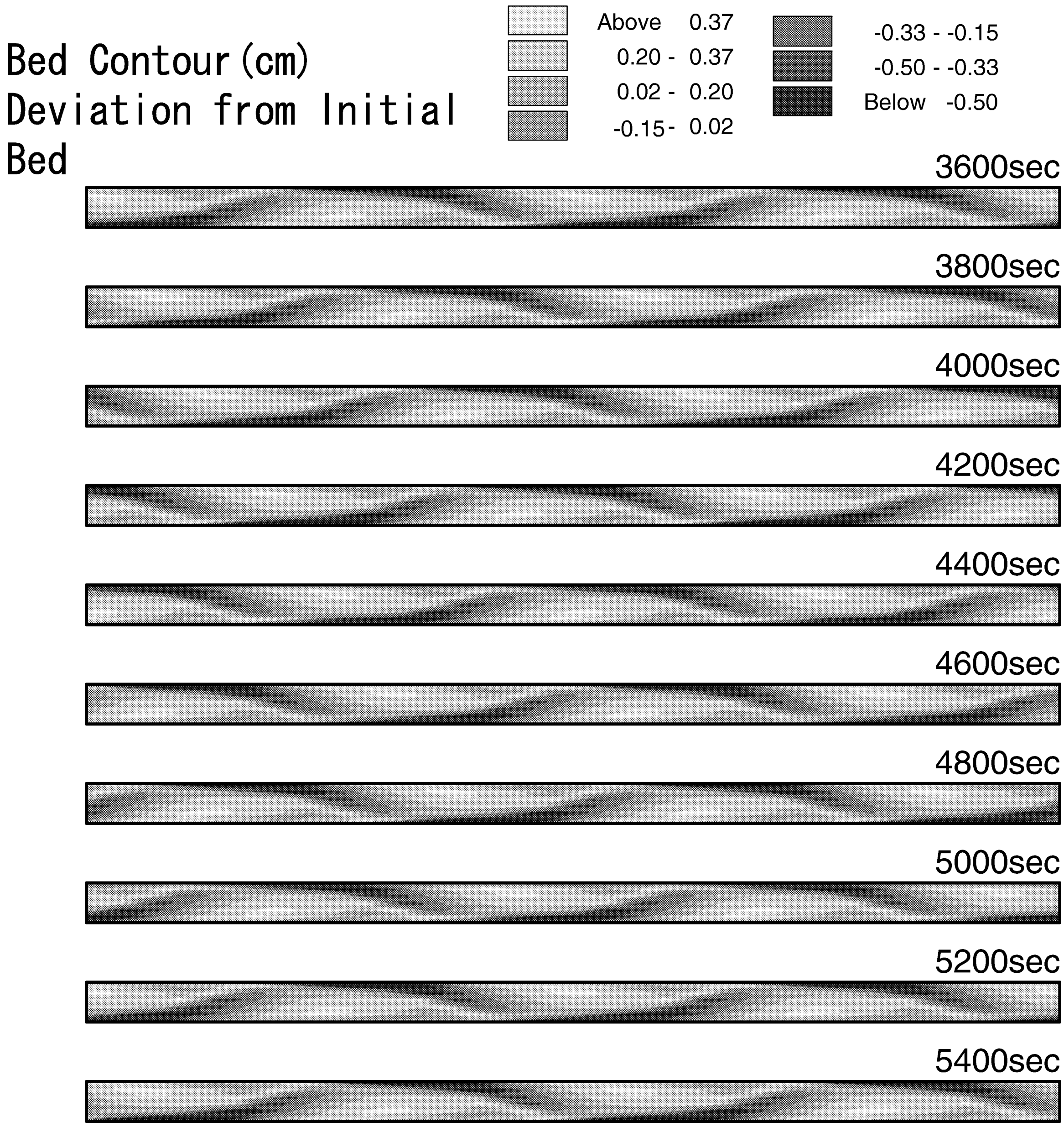

Figure 12: **Calculated bed elevation (without vertical drop structures).**

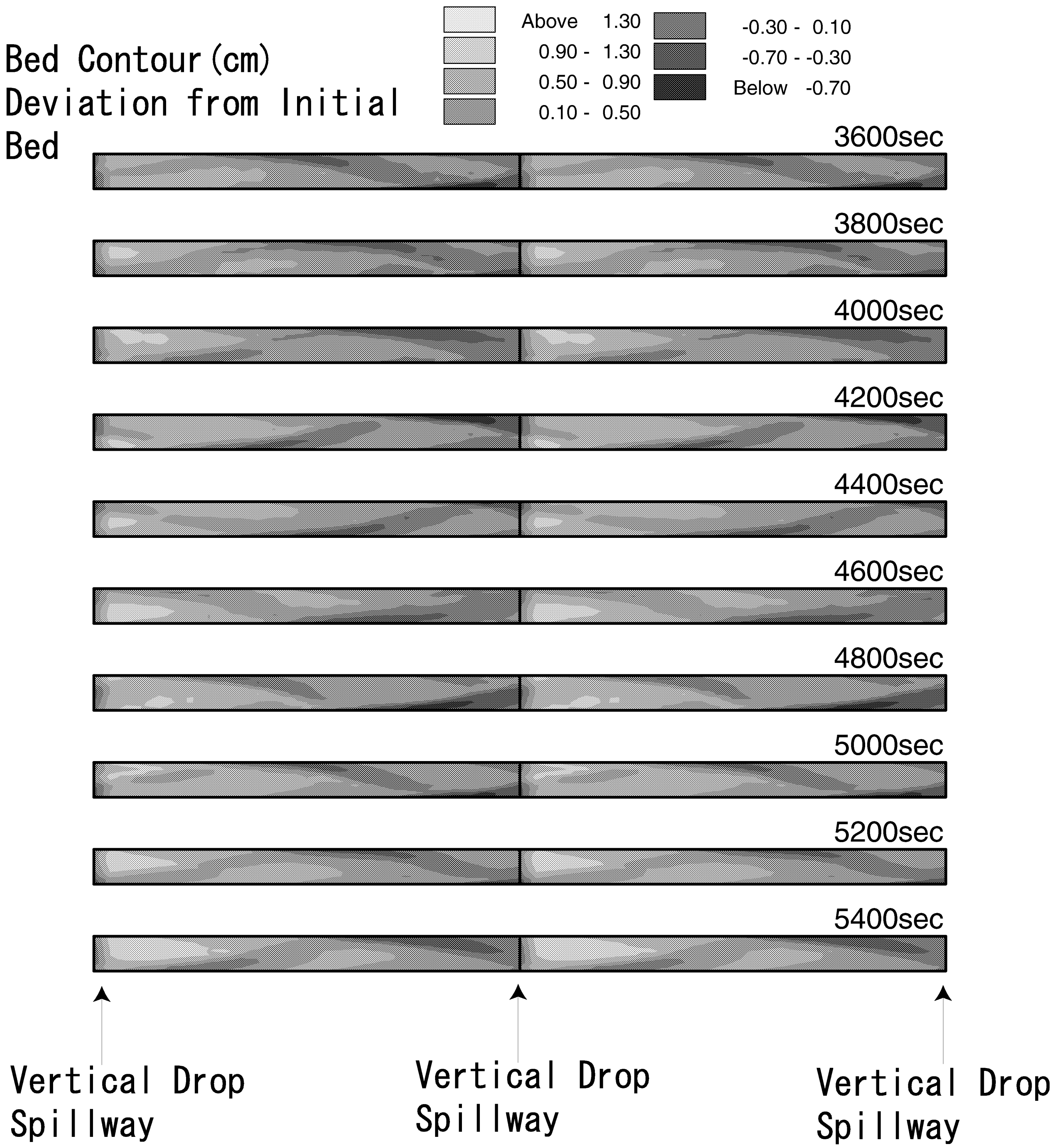

Figure 13: **Calculated bed elevation changes [Run1](with vertical drop spillways)**

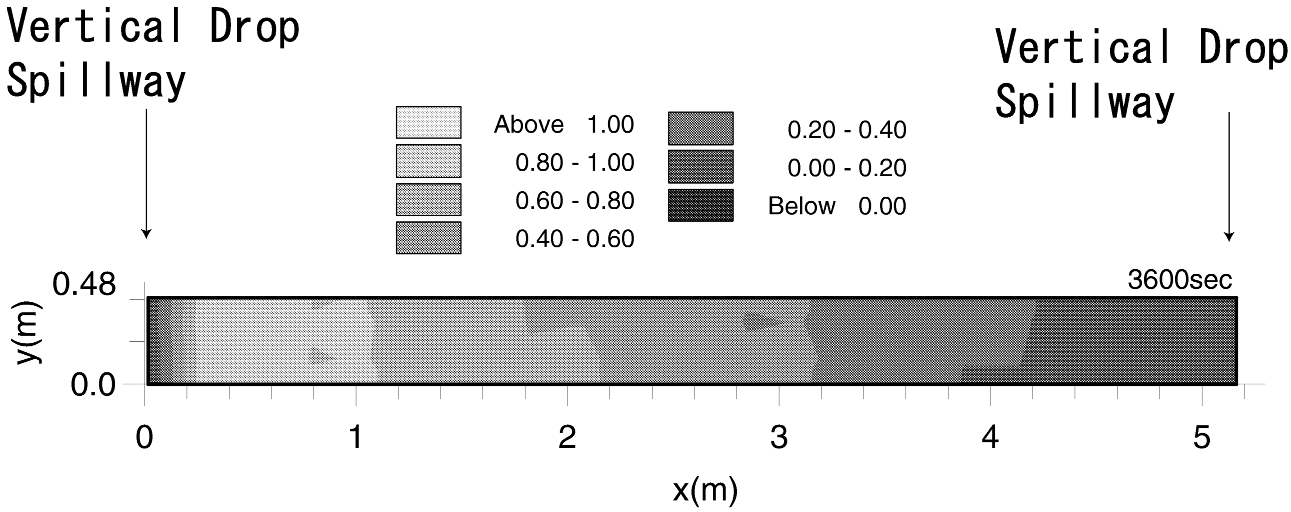

Figure 14: **Calculated bed elevation changes [Run2](with vertical drop spillways)**

with and without spillways, from 3,600 seconds to 5,400 seconds in the calculation. Figure 12 shows that alternate bars are migrating downstream with a constant speed. The calculated wavelength of the alternate bars was about 12 times the width of the channel. Bar height was about 1.3cm about 1.5 times the average water depth, slightly smaller than the observed value obtained during RUN-1 and shown in Figure 4.

For the model with vertical drop spillways the well regulated bar migration was not reproduced, Figure 13. It was discovered that when the bar fronts approach the vertical drops during the computer simulation their migration speed decreased, and thus their regular migration was interrupted. The calculation overestimates the deposition rate downstream of the drops. Which may be caused by non-equilibrium sediment transport passing through the vertical drop spillways. This is not taken into account in the present model.

Calculation was also conducted for the conditions of RUN-2, and calculated bed contours are shown in Figure 14. Alternate bars are not developed in this case, which agrees with the experimental results presented in Figure 4.

Conclusion

Characteristics of flow and bed movement especially the migration of alternate bar formations were studied using flume experiments and computer simulation. It has been shown that the regime criterion for the formation of alternate bars is valid even where vertical drop spillways exist in the channel.

A numerical model was proposed to calculate the flow and bed deformation in a compound channel with vertical drop spillways. In the numerical model the CIP method was used to calculate flow conditions with vertical drops and a partially flooded compound channel. The applicability of the model was confirmed by comparison with the experimental results. The performance of the proposed model using the CIP method was very successful and extends the range of application of numerical models beyond that available with existing codes.