

**THREE-DIMENSIONAL NUMERICAL STUDY ON FREE-FLOW
FLUSHING FOR ENHANCING THE EFFICIENCY OF SEDIMENT
MANAGEMENT IN RESERVOIRS**

by

Taymaz Esmaeili

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Taymaz Esmaeili

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Graduate School of Engineering
Kyoto University

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Taymaz Esmaeili
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Dedicated to

my Wife

FAHIMEH

my Daughter

SONYA

and my Dad and Mom

HALIMBERDI & HAMIDEH

Abstract

Sediment flushing is one of the proposed methods for preserving the storage capacity of dam reservoirs. In free-flow flushing (i.e., flushing with complete drawdown), the incoming flood erodes a flushing channel into the deposited sediment in the reservoir. Free-flow sediment flushing in reservoirs involves several complex processes. During a free-flow flushing, bottom outlets should be opened to drawdown the water level abruptly and to generate and accelerate the unsteady flow towards the outlet. When water level is completely drawndown, shallow flows emerge. Shallow flows behavior, and sediment transportation process during the water level drawdown and when shallow flows emerge are all complex matters especially due to the existing dynamic interaction between the unsteady flow field and bed changes. Loss of the stored water beside the lack of knowledge about the amount of the flushed out sediment, location of the erosion-deposition areas in the reservoir, changes in the bed morphology of river channel in downstream areas of the dam and the environmental impacts are all drawbacks for choosing the free-flow sediment flushing. Therefore, it is important to have an initial assessment of the impacts of an upcoming drawdown flushing operation. Because of the expected reduction of time and cost, and also to avoid the scaling problems, numerical models can be utilized. Recently, application of three-dimensional (i.e., 3-D) numerical models is feasible for simulating the flow field and sediment transport process owing to the increase in the calculation power of computers.

This study, first, focuses on 3-D numerical modelling of the velocity field in shallow reservoirs with different geometries and also different bed conditions (i.e., flat and misshaped bed). The experimentally measured surface velocity in all cases and velocity profiles in one case were used to validate the model. Then, flushing channel formation procedure was investigated in corresponding shallow reservoir geometries under the free-flow condition. The simulated final bed topography after the flushing operation in considered shallow reservoir geometries was validated with the measured one in corresponding physical models. Afterwards, the sediment flushing processes was investigated in the steep and meandering channel of Dashidaira and Unazuki reservoirs using a 3-D numerical model. For this purpose, a sensitivity analysis was carried out on mainly empirical parameters and also on bed load sediment transport formulas in the Dashidaira reservoir. Results, then, employed for Unazuki reservoir study area. Measured bed levels before and after the flushing operation were used for model verification purpose over the whole Dashidaira and over the study area of Unazuki reservoirs although some uncertainties may appear because bed levels are not measured soon after the flushing. Finally, soft-measure scenarios correspond to manipulation of water level and discharge rates during a free-flow fushing operation and hard-measure scenarios implies the construction of flow guiding structures,

and auxiliary longitudinal channels in the reservoirs were utilized to assess their effect on increasing the flushing efficiency.

As for 3-D flow simulation in rectangular shallow reservoirs, numerical results showed that a slight disturbance in the inflow boundary condition (e.g., 2.5% difference of velocity in one side of inlet compared to the other side) results in a steady asymmetric flow pattern in reservoirs with a higher defined shape factor but it does not affect the flow pattern in reservoirs with a lower defined shape factor. The simulated and measured flow velocity fields are reasonably consistent in all cases. Nonetheless, some discrepancy exists between the numerical results and the measurement in the upstream vortex dimensions and flow field in this area on both bed types (i.e., flat and misshaped). This discrepancy is mainly because of the simulated longer flow and reverse flow jet pattern, and also due to the simulated concentrated flow and reverse flow jet pattern with lower flow diffusion in the numerical outputs.

Regarding the simulation of flushing channel formation in rectangular shallow reservoirs, it was found that progressive erosion (i.e., erosion pattern propagates from inlet towards the downstream) plays a more important role in both sediment erosion and then sediment transportation compared to the retrogressive one (i.e., erosion pattern propagates from outlet towards the upstream) when free-flow condition exists. When the defined shape factor is low and the flushing channel is formed along the centerline, difference between the measured and calculated Flushing Efficiencies varied between 18%-39% (i.e., absolute error less than 11%), showing a reasonable performance of 3-D numerical model in representing the inherently complex process of free-flow flushing.

In case of flushing simulation in Dashidaira reservoir, outcomes showed that both Meyer-Peter-Müller (i.e., MPM formula) and Van Rijn's formula have a reasonable performance to predict the bed variations in specific zones of the reservoir. Nonetheless, MPM formula had a generally better performance for representing the morphological bed changes caused by the flushing operation and more than 75% of the measured Total Volume of Flushed out Sediment (i.e., TVFS) can be reached using this bed load sediment transport formula. In case of study area in Unazuki reservoir, deposition was dominant as the result of flushing because it has not reached to the equilibrium condition yet, and a portion of flushed out sediment from Dashidaira reservoir is deposited in this area. Thus, another criterion that shows the average bed changes in the study area was used to compare the numerical outcomes with the measurements.

Amongst the applied soft scenarios, introducing an artificial additional discharge during the free-flow stage is more reasonable from different aspects for reservoirs located across the Kurobe River (e.g., practical feasibility and environmental implications). However, it was found that introducing the extra discharge in the form of scattered discharge pulses with a bigger discharge pulse placed in the second half of the free-flow stage would be more efficient to enhance the FE and meanwhile increase the bed degradation in upstream areas covered with coarser materials in case of Dashidaira reservoir. Regarding the study area of Unazuki reservoir, this approach was challenging owing to the shorter duration of free-flow stage that implies a very big discharge pulse in the second half of free-flow stage.

Numerical outcomes showed that approximately 20.5% increase in the used water volume, in the form of additional discharge during the free-flow stage in Dashidaira reservoir, results in 4%-13% increase in the FE depends on how this additional water is introduced. 11% increase in the used water volume compared to the reference case, in the form of a constant additional discharge during the free-flow stage, can change the dominant morphological process in the study area of Unazuki reservoir from deposition to erosion mode. In addition, high magnitude of extra water level drop for a short period during the second half of drawdown stage had a significant effect on sediment mobilization, including coarser fractions in case of Dashidaira reservoir.

Application of hard measure scenarios such as flow guiding structures or auxiliary flushing channels implies a high construction cost. While they may have a local favorable effect during the flushing, they exist in the reservoir during the sedimentation period that can be the cause of some adverse effects. Thus, it is necessary to check their influence during the sedimentation period first. Nonetheless, application of flow guiding structures and auxiliary longitudinal channels in Dashidaira reservoir has locally increased the erosion from the dead zone area and consequently FE increased about 9% compared to a reference case.

Keywords

3-D numerical model, free-flow sediment flushing, bed morphology, flushing efficiency, reservoir sustainability

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List of symbols and abbreviations

A = given surface area of a cell;

$A1$ = asymmetric flow pattern in a shallow reservoir;

A_n = area of the side wall on the north side of cell p ;

A_e = area of the side wall on the east side of cell p ;

A_s = area of the side wall on the south side of cell p ;

A_w = area of the side wall on the west side of cell p ;

ADD = a proposed scenario implies the introduction of an Additional constant Discharge during the Drawdown stage;

ADF = a proposed scenario implies the introduction of an Additional constant Discharge during the Free-flow stage;

a = reference level set equal to the roughness height;

a_i = weighting coefficient for a neighbor cell;

a_p = weighting coefficient in cell p ;

a_n = weighting coefficient of the neighbour cell located in the north side of cell p ;

a_e = weighting coefficient of the neighbour cell located in the east side of cell p ;

a_s = sediment concentration of the neighbour cell located in the south side of cell p ;

a_w = sediment concentration of the neighbour cell located in the west side of cell p ;

b = inlet channel width;

B = reservoir width;

BCI = Bed Change Index;

C_{bed} = sediment concentration close to bed;

c = sediment concentration over time t and spatial geometries (i.e., x and z);

c_f = conversion factor between the sediment flux and bed level movement;

c_p = sediment concentration in cell p ;

c_n = sediment concentration in the neighbour cell located in the north side of cell p ;

c_e = sediment concentration in the neighbour cell located in the east side of cell p ;

c_s = sediment concentration in the neighbour cell located in the south side of cell p ;

c_w = sediment concentration in the neighbour cell located in the west side of cell p ;

c_μ = empirical constant;

$c_{1\epsilon}$ = empirical constant;

$c_{2\epsilon}$ = empirical constant;

d = sediment particle diameter;

d_i = diameter of the i th fraction of sediment particles;

d_{50} = mean size of sediment;

d_{90} = sieve size that 90% of the sample mass is finer than this size;

E = empirical value equals to 9.0;

F_n = total flux through the north side of cell p ;

F_e = total flux through the east side of cell p ;

F_s = total flux through the south side of cell p ;

F_w = total flux through the west side of cell p ;

FE = Flushing Efficiency;

Fr = Froude number;

g = acceleration of gravity;

h = flow depth;

I = slope of the energy line;

i = index for three spatial directions;

K = reduction factor for original value of the critical shear stress;

k = turbulence kinetic energy;

k_s = roughness equivalent to a diameter of particles on the bed;

k = turbulence kinetic energy;

k_s = equivalent bed roughness;

KCM = Kilo Cubic Meter;

L = reservoir length;

$LSPIV$ = Large Scale Particle Image Velocimetry technique;

$Mea.$ = Measured;

MAE = Mean Absolute Error;

MPM = Meyer-Peter-Müller bed load sediment transport formula;

n = number of grid cells in the vertical direction;

n_{max} = maximum number of grid cells in the vertical direction;

Pe = Pechlet number;

P = pressure;

P_k = production of turbulent kinetic energy;

p_p = pressure in the cell;

p_i = pressure in the i th neighbour cell;

po = parameter for the number of grid cells that is defined by the user;

PDF = a proposed scenario implies the Introduction of a Pulse of Discharge during the Free-flow stage;

Δp = pressure difference;

Q = flow discharge;

Q_1 = average discharge during the free-flow stage in a reference case when an ADF scenario is not implemented;

Q_2 = average discharge during the free-flow stage under an ADF scenario;

Q_s = sediment transport capacity;

Q_f = flushing discharge;

$q_{b,i}$ = sediment transportation rate for the i th sediment fraction of bed load per unit width;

R = particle Reynolds number;

r = hydraulic radius;

$\vec{r}$ = direction vector pointing from the center of a cell to the centre of the neighbour cell;

RMSE= Root Mean Squared Error;

S = bed slope;

S_0 = symmetric flow pattern in a shallow reservoir;

Sc = Schmidt number;

SF = Shape Factor;

S_i = sediment inflow quantity;

S_o = sediment outflow quantity;

$Sim.$ = Simulated;

t = time;

TVFS= Total Volume of Flushed out Sediment from a reservoir;

U_{bed} = velocity at the bed cell;

U_j = averaged flow velocity;

u_i = flow velocity in i direction;

u^* = shear velocity;

u = velocity fluctuations;

$\vec{u}$ = velocity vector of a cell;

UVP= Ultrasonic Velocity Profiler;

V_{in} = actual streamwise velocity at the inlet, which is affected by a slight disturbance;

V_0 = reference streamwise velocity at the inlet;

V_{out} = volume of the flushed out sediment;

V_{dep} = volume of the deposited sediments after deposition phase;

W = flushing channel width;

w = fall velocity of sediment;

WDS= a proposed scenario implies the Increasing of Water level Drawdown Speed;

x = coordinate in longitudinal direction;

x_i = spatial geometric scale;

y = coordinate in transversal direction and distance from the boundary;

z = coordinate in vertical direction;

z_p = water level elevation in the cell;

z_i = water level elevation in the i th neighbour cell;

$z_{i,ms}$ = measured or simulated bed level after flushing operation at each grid node;

$z_{i,reference}$ = measured or simulated reference bed level at each grid node;

∂z = elevation difference;

α = magnitude of linear variation of streamwise velocity along the inlet channel width;

ε = dissipation of k ;

δ_{ij} = Kronecker delta ;

ρ = density of fluid;

ρ_w = density of water;

ρ_s = density of sediment;

ν = viscosity of the water;

ν_T = eddy viscosity;

κ = constant equal to 4.0;

ε = dissipation of k ;

Γ_T = diffusion coefficient;

Γ_p = turbulence diffusion in cell p ;

Γ_n = turbulence diffusion in the neighbour cell located in the north side of cell p ;

Γ_e = turbulence diffusion in the neighbour cell located in the east side of cell p ;

Γ_s = turbulence diffusion in the neighbour cell located in the south side of cell p ;

Γ_w = turbulence diffusion in the neighbour cell located in the west side of cell p ;

τ_o = bed shear stress;

τ_{bed} = induced bed shear stress;

τ_c = critical bed shear stress;

$\tau_{c,i}$ = critical shear stress for the i th fraction of sediment particles;

τ^* = dimensionless shear stress;

τ'^* = Shields parameter related to the skin friction;

$\tau_{c,i}^*$ = reduced value of the critical shear stress for the i th fraction of the sediment;

$\tau_{c,i,0}^*$ = original value of the critical shear stress obtained from the Shields curve;

σ_ε = empirical constant;

σ_k = empirical constant;

Δ = *bed form hight;*

Δh_1 = *original water level drop at the beginning of the target segment in a WDS scenario;*

Δh_2 = *extra imposed water level drop at the beginning of the target segment in a WDS scenario;*

δz_0 = *vertical bed level movement;*

β = *angle between streamline and direction of near bed flow;*

α = *bed slope;*

φ = *slope parameter similar to the angle of repose;*

ψ = *angle between the near bed flow direction and sediment transport on the laterally inclined slope;*

$\partial h/\partial n$ = *transversal slope of the bed;*

Chapter 1 Introduction

Reservoirs are artificial lakes created by impounding a natural water course (e.g. a river) and are subject to some degree of sediment inflow and deposition. Reservoirs are used for different purposes such as water supply for irrigation, industry, or drinking water, flood control, discharge regulation for navigation, and hydropower generation. When a river enters a water reservoir and the flow velocity reduces, all bed load and coarser fractions of suspended load are deposited at the head of the reservoir whereas the finer fractions are transported further into the reservoir. Sediment deposition is the principal problem affecting the useful life of the reservoirs. The loss of reservoir storage volume due to the sediment deposition reduces the dam's effective life time and also diminishes the reservoir function for flood control purpose, hydropower generation, irrigation and water supply which represents a substantial economic loss (Morris and Fan, 1998). Thus, preserving the storage capacity of the existing reservoirs is a high priority issue for the authorities since building the new reservoirs is difficult owing to the strict environmental regulations, high cost of constructions, and lack of the suitable sites for new dams (Lai and Shen, 1996).

In Japan, generally the sediment yield by rivers is high compared to other countries due to the geologically young mountains, steep slope, and flashy flow regimes as well as wide spread landslides (Sumi, 2006 ; Kantoush et al., 2010). Consequently, incoming sediment loads to the reservoirs fed by these rivers are high (e.g. Kurobe River located in Toyama prefecture). Therefore, a free-flow flushing operation in two consecutive reservoirs located across the Kurobe River (i.e. Dashidaira and Unazuki) is performed each year for preserving the storage capacity.

In this study, first, features of flow field and free-flow flushing operation in a set of rectangular shallow reservoirs were numerically investigated. Then, flushing operation in both Dashidaira and Unazuki reservoir were simulated. Finally various type of measures suggested for each reservoir to increase the sediment flushing efficiency.

1.1 Overview of sedimentation issue in dam reservoirs and mitigating measures

The current estimate of total reservoir storage worldwide is around 7000 km³ (Palmieri et al., 2003). Between 0.5 and 1% of the storage volume of reservoirs is lost per year because of the sedimentation (White, 2001). The annual loss per year is also higher in the specific areas such as middle east and China (i.e. 1.5% and 2.3% respectively) (Palmieri et al., 2003). This can be attributed to the population pressure on fragile upland ecosystem that causes the accelerated soil erosion rates (Atkinson, 1996). Table 1.1 shows the sedimentation rate in different locations of the world. Using an average rate of sediment loss, the total volume of storage loss would be about 45 km³ per year. If the average volume

of a reservoir assumed to be 150 million m³, then, 300 large dams should be constructed annually solely to keep the current total worldwide storage (Palmieri et al., 2003). This implies a significant cost just for replacing this lost storage volume without considering the environmental and social costs of new dams. Beside this cost, the reservoir sedimentation associates with other direct costs such as less hydropower production, less irrigated land for food production, and decreased flood control function. In addition, if a reservoir is fully filled with sediments, it would be a hazard and owners are responsible for that while there is no benefits to pay for the reservoir maintenance (Palmieri et al., 2003). Meanwhile, decommissioning of fully filled reservoirs imposes both direct and indirect costs and there is no experience of large dams decommissioning. It should be also taken into account that most of the appropriate sites for dam construction have been already used. Besides, there is now a background of environmental and political pressures regarding the construction of large dams, which may result in a fewer new dam constructions in the future (Atkinson, 1996).

Due to the importance of sustainable sediment management for conservation of the existing reservoirs, different approaches such as upstream sediment trapping, sediment routing, density current venting, bypassing, sediment flushing and sluicing, and dredging have been developed and used. The following paragraphs, briefly introduce the mentioned approaches and their general features.

The upstream sediment trapping approach can be classify in the category of sediment yield reduction from the water shed, conducted through building a large artificial structure or more frequent structures across the stream. This framework can also includes the erosion control tasks from the watershed (e.g. bank erosion control in the stream, reforestation, and implementing vegetative screens). The aforementioned approaches can be an effective method as a long-term countermeasure against reservoir storage loss while associates with high costs and uncertainties that may lead into a limited benefit. In addition, the sediment inflow into the reservoir could not be stopped completely. Thus, this approach is not appropriate for already reduced storage reservoirs (Morris and Fan, 1998).

Region	Number of large dams	Annual sedimentation rate
Worldwide	45571	0.5-1
Europe	5497	0.17-0.2
North America	7205	0.2
South and Central America	1498	0.1
North Africa	280	0.08-1.5
Sub Saharan Africa	966	0.23
Middle East	895	1.5
Asia (Excluding China)	7230	0.3-1
China	22000	2.3

Table 1.1 Worldwide annual sedimentation rate (White, 2001).

The sediment routing (e.g. sediment sluicing) and density current venting approaches classify in the category of sediment deposition minimizing in reservoirs. In sediment routing approach, a flood with high sediment load is passed through the reservoir or around the reservoir (e.g. by using a sediment bypass system) to minimize the sediment settling and deposition. The techniques used in pass-through approach are based on increasing the flow velocity and turbulences during a flood event by opening the bottom outlets and drawing down the water level which keep the sediment particles in suspension without settling. In density current venting, as a special type of sediment routing approach, the water level could be kept in a high level. The bottom outlets are opened and density current which traveled along the submerged thalweg and reached the dam can be passed through the dam. The coarser fractions of the inflowing sediment and also already deposited sediments may not be removed using pass-through approach. Practically, both pass-through and density current venting approaches are complex to apply and mainly applicable at hydrologically small reservoirs where enough water is available for sediment release. In a bypass system, as another alternative, the major portion of sediment-laden flows will not be allowed to enter the reservoir. The bypass system with entrance at the upstream of the reservoir, is opened during a flood and inflowing water with a high sediment load can be bypassed into an off-stream reservoir or led back into the main stream at downstream area of the dam (Morris and Fan, 1998).

The most challenging task for sustaining the effective storage capacity is removing the already deposited sediment from the reservoir. Mechanical excavation (e.g., dredging) is a conventional method which usually associates with high costs and economically not feasible in many cases. Dry excavation is a special type of mechanical excavation and applicable mainly for coarser materials that will dewater fast. To do this, the reservoir should be emptied to excavate the emerging coarser sediments that have been deposited in the reservoir. Despite the high costs, sometimes mechanical excavation is the only available option for sediment management in a reservoir. Recently, a new method of dredging called siphon dredge has been developed in a physical model scale for small reservoirs to reduce the costs while having a reasonable efficiency (Chen et al., 2010).

Alternatively, sediment flushing in reservoirs offers the only means of recovering storage volume without incurring the dredging or mechanical excavation expenditure (Atkinson, 1996). Reservoir flushing is an approach where the transport capacity increases in the reservoir by the water flow itself and without using an external energy (Scheuerlein, 1990). Drawdown flushing involves a complete lowering of the water level by opening the bottom outlets to temporarily establish riverine flow (i.e. free flow) towards the outlet. The accelerated flow erodes a channel through the deposits and flushes out both fine and coarse sediment (Morris and Fan, 1998 ; Shen, 1999). Of all the mentioned measures, flushing plays a major role in reservoir storage capacity restoration since it is an efficient hydraulic technique for sediment removal (Lieu et al., 2004). During the flushing operation wide range of deposits from clay to gravel can be flushed out. However, the sediment flushing approach is not appropriate for all reservoirs and success of flushing depends on the reservoir geometry, sediment characteristics, the hydraulic and hydrological conditions (Esmaeili et al., 2014). For instance, the flushing operation in wide reservoirs results in formation of a narrow flushing channel through the deposits which forms a

channel and flood plain type geometry. Deposits in the channel can be eroded in the subsequent flushing operation, but deposited sediment in the flood plain during the impounding period have a low chance of erosion. Furthermore, large amount of water should be used and reservoir is required to be emptied (Sumi & Kantoush, 2010). Thus, this approach is mainly feasible for hydrologically small reservoirs which are able to refill quickly after the flushing operation. Besides, a large amount of deposited sediments are flushed out during a free-flow flushing operation, which increase the sediment concentration and thereby can result in negative impacts on the aquatic animal life and their habitat structure. Therefore, environmental impacts should be assessed and mitigating measures should be considered (Sumi et al., 2009).

1.2 Motivation and objectives

In order to conduct a sustainable sediment management in Dashidaira and Unazuki reservoir, deposited sediment in these reservoirs should be flushed out. Flushing out the accumulated sediment in these reservoirs is necessary because of the following reasons:

First, the sediment load that is transported by the Kurobe River is very high while the reservoirs capacity is almost hydrologically small. If suitable countermeasures are not implemented, the storage loss results in a decreased function of hydropower electricity production at Dashidaira dam and also decreased function of flood control, water supply, and electricity generation at Unazuki dam. Beside a direct economic loss through reduction of electricity production, the massive torrential floods threaten the safety of residents living in an alluvial fan located in lower reaches of Kurobe River which implies huge direct and indirect economic and social costs.

Second, the termination of natural sediment load because of several existing dams along the river course is the cause of bed degradation in middle and lower reaches of the river. This degradation threatens the safety and stability of river embankments. Thus, it is necessary to supply the sediment load in a near-natural condition as far as possible. This issue is more highlighted in downstream reaches where the alluvial fan is located. In this region, the amount of transported coarser fractions of sediments is lower than finer fractions although the free-flow flushing operation is performed. Figure 1.1 depicts the river mouth area, and alluvial fan located in lower reach of the Kurobe River close to the Toyama bay that is one of the largest ones in Japan.

Third, protection of the coastline formed by the alluvial fan of the Kurobe River from the severe wave induced erosion is necessary. The Shimo-Niikawa coastline was subject to erosion historically. The coast line has been receded about 40 m during the 30 years period from 1966 to 1996 (Kurobe Construction Office). As a countermeasure, series of off-shore concrete breakwaters have been installed in different locations. The location of Shimo-Niikawa coastline along with the aerial photo from the Ashizaki area under the effect of raging waves that causes a serious induced erosion has been illustrated in Figure 1.2. Figure 1.3 demonstrates installed offshore breakwaters to protect the coastline area. However, it is necessary to secure the inflow of enough sediment loads from the Kurobe River into the estuary zone. This can be performed through flushing out the accumulated sediments in the

reservoirs located across the Kurobe River. Finer materials that can be transported further to the coastal area, during a sediment flushing operation, plays the main role for this purpose. Figure 1.4 shows the inflow of sediment loads from Kurobe River into the estuary zone after conducting a flushing operation in reservoirs located across the river (i.e., Dashidaira and Unazuki reservoirs) and their contribution in protecting the coastline from wave induced erosion.



Figure 1.1 Photo from the estuary zone of the Kurobe River and alluvial fan formed by this river in the lower reach area (Photo from Kurobe River Work Office).

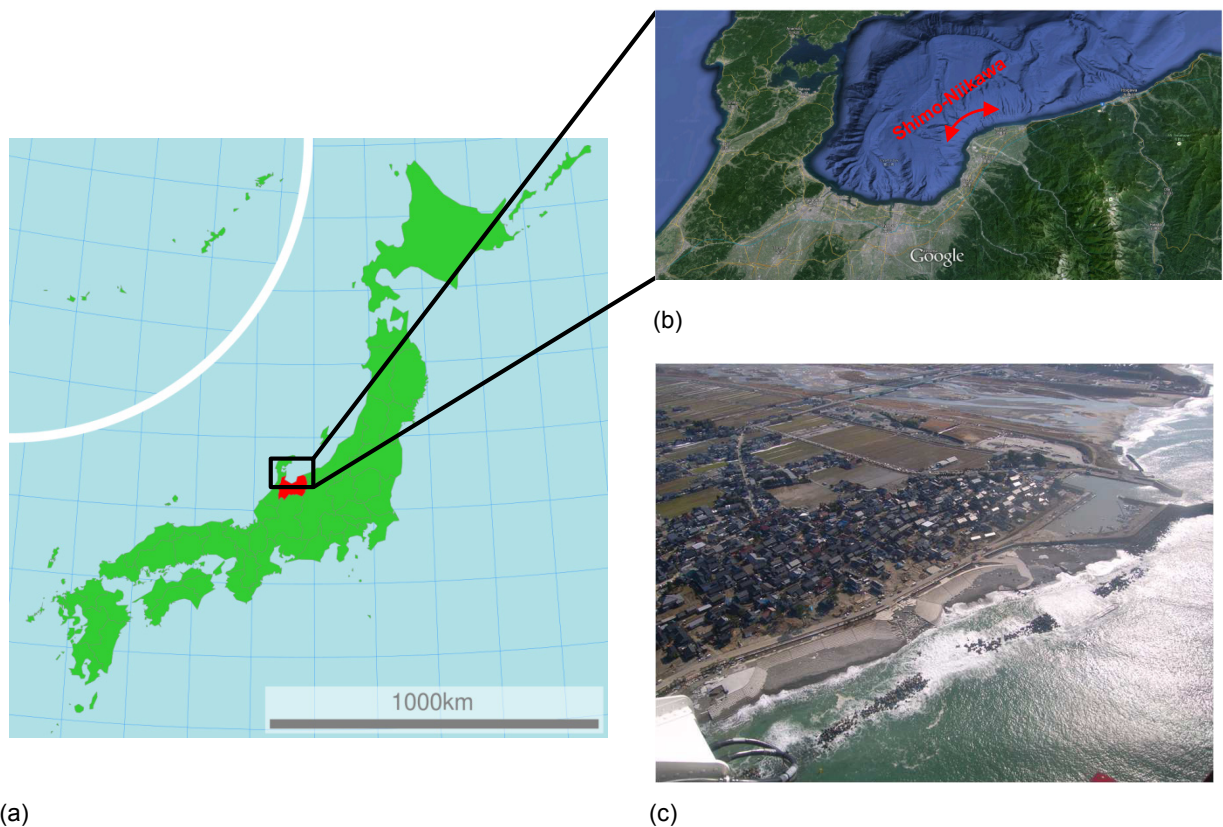


Figure 1.2 (a) Location of the coastal area of Toyama bay where Shimo-Niikawa coastline has been located (Figure from www.wikipedia.org); (b) location of the Shimo-Niikawa coastline under the serious wave induced erosion (Figure from <https://maps.google.co.jp>); (c) aerial photo shows the raging waves attacking the the coastline in Ashizaki area (Ranasinghe et al., 2011).

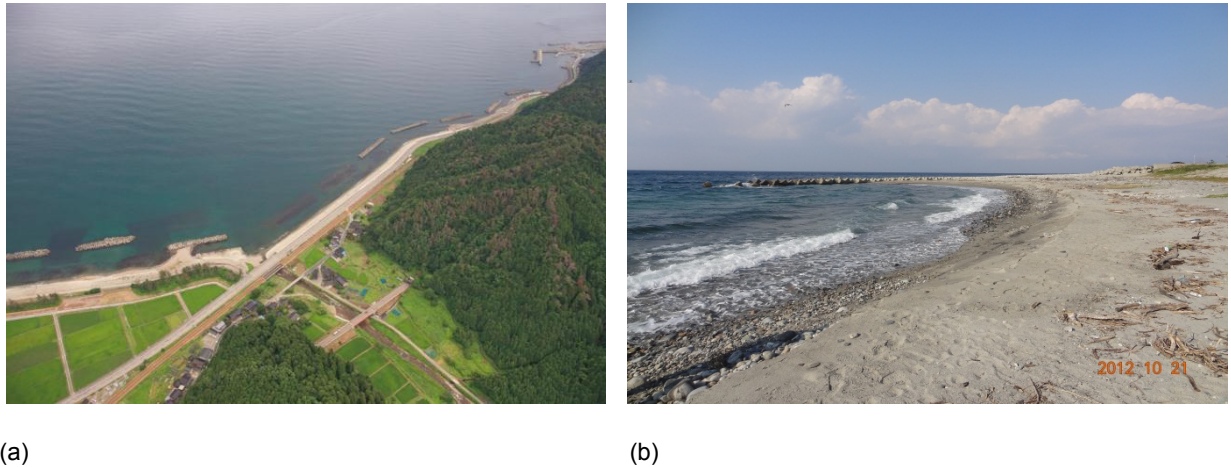


Figure 1.3 Installed offshore breakwaters for protecting the estuary from erosion: (a) aerial view (Photo from Kurobe River Work Office; (b) view at the coast line (Photo taken by author).

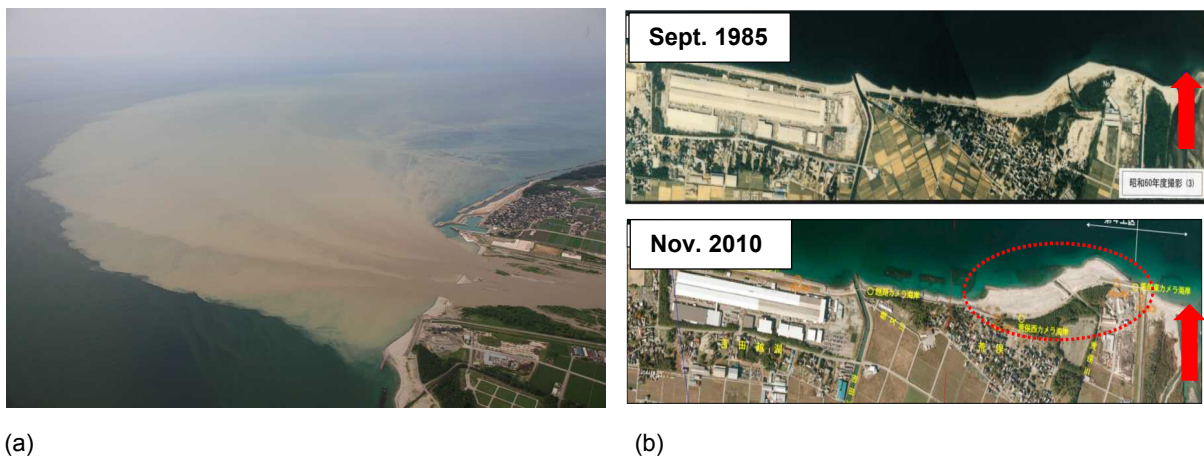


Figure 1.4 (a) diffusion of the released sediment loads after a flushing operation in the estuary zone of Kurobe River (Photo from Kurobe River Work Office); (b) effect of sediment load supply on coastline protection. Upper photo has been taken in 1985 when the free-flow sediment flushing had not been implemented regularly and the lower one has been taken in 2010 after implementing the regular free-flow flushing several times (Photo modified from Kurobe River Work Office).

Finally, it is vital to implement a comprehensive sediment management from the river source to estuary area in a more natural way with respect to the environmental concerns. Flushing out the accumulated sediment from reservoirs during a flood represents a more similar condition to the natural sediment transportation pattern in a river course because a higher sediment load is naturally transported during a flood.

The necessity of sustainable sediment management strategy in Dashidaira and Unazuki reservoirs with respect to the environmental concerns implies an environment-friendly sediment flushing operation. As a solution, a coordinated sediment flushing operation is being performed since June 2001 in both Dashidaira and Unazuki reservoirs. In this type of sediment flushing operation, the water level is abruptly drawdown during the latter half of a flood to generate a higher velocity and induced bed shear

stress which initiates a flushing channel formation along the reservoir and increases the sediment entrainment. Then, the bottom outlets are fully opened for a scheduled period which forms the free-flow condition with a low water head while inflow and outflow water discharge is almost equal. Consequently, the initial flushing channel develops further and eroded sediments will be transported. Finally, bottom outlets are closed and the water level is restored. This operation is fully linked and conducted in both reservoirs during a flood event. Spillways of both dams are kept open after the flushing operation to provide a continuous water discharge which can facilitate the recovery of downstream environmental structure. Another coordinated sediment sluicing (i.e. sediment routing) in both reservoirs is performed with the same procedure of water level drawdown during a successive flood to prevent the new sediment deposition after the flushing operation. The coordinated sediment sluicing operation is very similar to a natural flood condition since the coming flood pass through the reservoirs. The Kurobe River sediment flushing committee that is made by scholars and the Kurobe River sediment management council that is representative of towns and concerned organization in this area, and local farming and fishing communities are involved in monitoring the flushing operation in these reservoirs. Intense environmental surveys also are carried out before and after the flushing operation to monitor the environmental impacts. The results are made public and discussed by the mentioned committee and council. Figure 1.5 demonstrates the concept of coordinated sediment flushing operation in Dashidaira and Unazuki reservoirs.

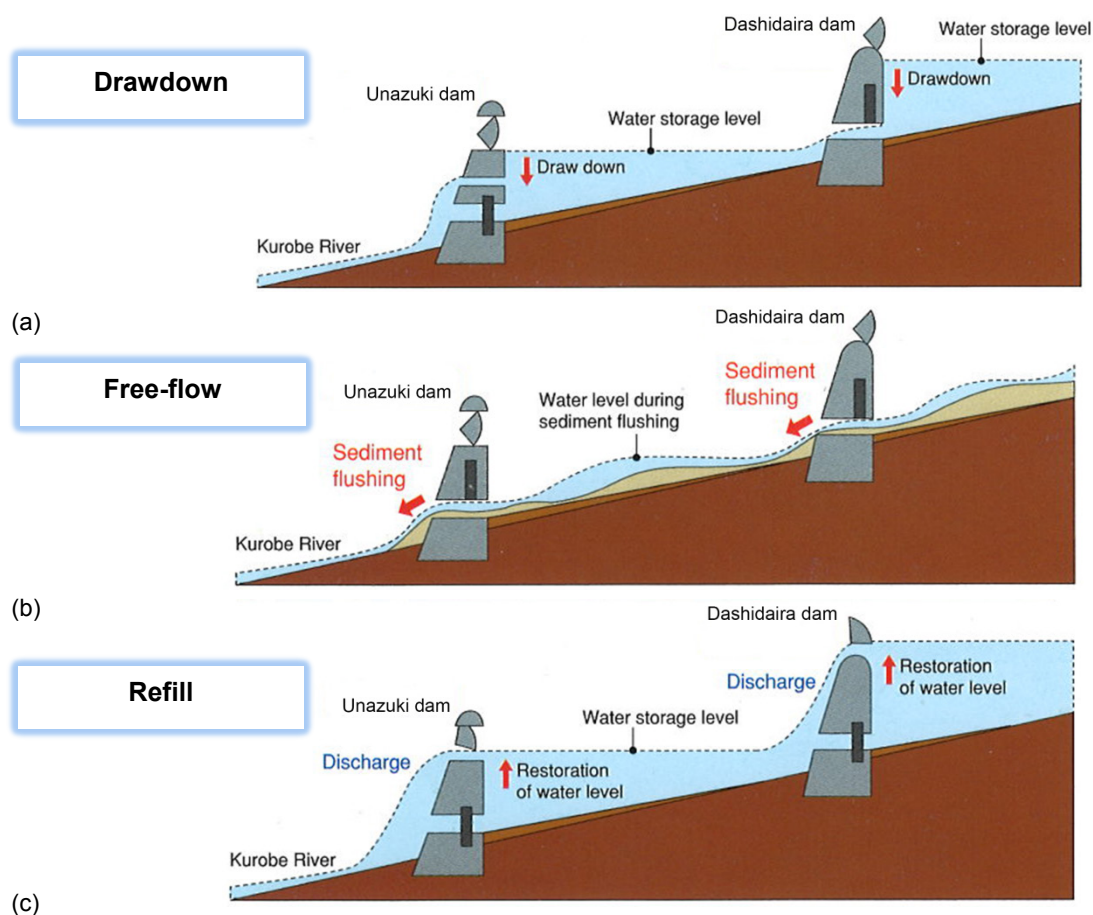


Figure 1.5 Coordinated sediment flushing outline in both Dashidaira and Unazuki reservoirs: (a) drawdown stage; (b) free-flow flushing stage; (c) Refill (i.e., recovery) stage (Figures modified from Kurobe River Work office).

Evaluation of reservoir flushing operation is a prerequisite to conduct a robust and also sustainable sediment management policy. Insight into the physical process of the flushing operation enables us to implement a more efficient sediment flushing operation from different aspects. However, the physical process involved is complex and there is a limited theoretical explanation of flushing channel formation is available in reservoir delta. For instance, location, shape, width and depth of flushing channel have a great influence on the flushing efficiency (Kantoush et al., 2010). Meanwhile, reservoir shape, volume and grain size of deposit materials, Discharge and water level variations affect the initiation of flushing channel formation.

In case of Dashidaira and Unazuki reservoir, owners of both reservoirs are interested in enhancing the flushing efficiency. In case of Dashidaira reservoir, there is a wide middle area in the reservoir where the flushing channel develops close to the right bank, during the flushing, and deposited sediments in area close to the left bank have no chance for erosion. Any type of optimization for increasing the erosion over this dead zone, contributes in enhancing the flushing efficiency. In addition, increasing the coarser material erosion from the upstream area of the reservoir during the flushing operation may overallly increase the amount of transported coarser materials to Unazuki reservoir first and then to the lower reaches of the Kurobe River after achieving the equilibrium state in the Unazuki reservoir. In Unazuki reservoir, its wider shape compared to Dashidaira reservoir implies a more challenging morphological bed changes during the flushing operation. There is a wide area in the entrance of this reservoir and deposits could not be removed efficiently from this area. Subsequently, a significant portion of this area remains as a dead zone. Likewise the wide middle area of Dashidaira reservoir, increasing the erosion over this dead zone may enhance the flushing efficiency and also increase the coarser materials supply for lower reaches of the Kurobe River. Figure 1.6 shows the flushing channel formed in the wide middle section of Dashidaira reservoir during the flushing operation in 2012. The dead zone which mainly includes the finer sediments can be seen from this figure.



Figure 1.6 Developed flushing channel during 2012 free-flow flushing operation in Dashidaira reservoir (Photo from Kurobe River Work Office); Red arrows show the flow direction and red oval illustrate the wide area of the reservoir with a dead zone along the left bank.

This study, therefore, generally focuses on explanation of the physical process involved in the free-flow flushing operation that was implemented in the mentioned reservoirs. Afterwards, the study was further developed with the insight of increasing the flushing efficiency by implementing some potentially practical measures. To achieve these goals, a 3-D Computational Fluid Dynamic (CFD) program was employed which implements the coupled computation of flow field and sediment transportation patterns (i.e. erosion and deposition). Taking into account the aforementioned targets, main objectives of this study are:

- Numerical investigation and analysis on hydrodynamic features of low-head waters (i.e. shallow flows) in a set of rectangular shallow reservoirs with different geometries and bed floor condition (i.e. flat and deformed bed) because of dominant role of shallow flows in sediment entrainment, flushing channel formation, and its development during the free-flow flushing operation.
- Numerical modeling of flushing channel formation and development in rectangular shallow reservoirs during a free-flow flushing operation and analysing the governing physical process since it would be easier to understand the process in these simple experimental cases.
- Numerical modeling of free-flow flushing operation of year 2012 in both Dashidaira and Unazuki reservoirs using the hydraulic, geometric and morphological features of each case study for representing the morphological bed changes during and after the flushing operation.
- Providing a general insight on 3-D numerical modeling of free-flow flushing operation in prototype scale reservoirs for presenting some recommendations to set up the numerical model.
- Conducting a sensitivity analysis for figuring out the effect of the mainly empirical parameters on the morphological bed changes caused by a free-flow flushing operation and consequently on the Total Volume of Flushed out Sediment (TVFS).
- Examining several approaches for increasing the flushing efficiency in both Dashidaira and Unazuki reservoirs through modifications in discharge and water level rates (i.e. soft measures), and through implementing some construction works (i.e., flow guiding or longitudinal auxiliary channel) to affect the flow pattern (i.e. hard measures).

1.3 Project outline

Dashidaira and Unazuki reservoirs are located across the Kurobe River. Dashidaira is a hydropower dam constructed in 1985. Both are the first cases in Japan designed with sediment flushing facilities. The height of Dashidaira dam is 76.7 m and produces 124 Mega Watt of electricity. The gross and effective storage capacities are 9.01 and 1.66 million m³. Since 1991, the sediment flushing operation is carried out in this reservoir. The Multipurpose Unazuki gravity dam with 97 m height has been completed in 2000. Dam is mainly used for flood control purpose whereas water supply and power generation are other functions of the dam. A discharge of 700 m³/s can be regulated by the dam when a flood occurs with 6900 m³/s peak discharge. This dam has been built 7 km downstream of Dashidaira dam with gross and effective storage capacity of 27.7 and 12.7 million m³, respectively (Lieu et al., 2004). While Dashidaira dam is operated by Kansai Electric Power Co., Inc., Unazuki dam is controlled

directly by the Ministry of Land, Infrastructure and Transport. Figure 1.7 shows downstream face of Dashidaira and Unazuki dam.



(a)



(b)

Figure 1.7 Photo taken from downstream face of (a) Dashidaira dam by author; (b) Unazuki dam (<http://commons.wikimedia.org>).

Kurobe River originates from the Washiba Mountain with 3000 m height, located in the northern Japanes Alps and flows into Toyama bay in Japan Sea. The Catchment area of the river is 682 km² and the length of the river is 85km. The bed slope is steep and varies between 1% and 20%. The average rainfall and total sediment yield per year are 4000 mm and 1.4×10^6 m³/year, respectively, that are both one of the highest in Japan. Almost 7000 landslide sites in upper reaches of Kurobe River make it a river with a high ratio of landslide area to the mountainous area in Japan. Figure 1.7 illustrates the location of Dashidaira and Unazuki reservoir across the Kurobe River (Minami et al., 2012 ; Kurobe Office Report).

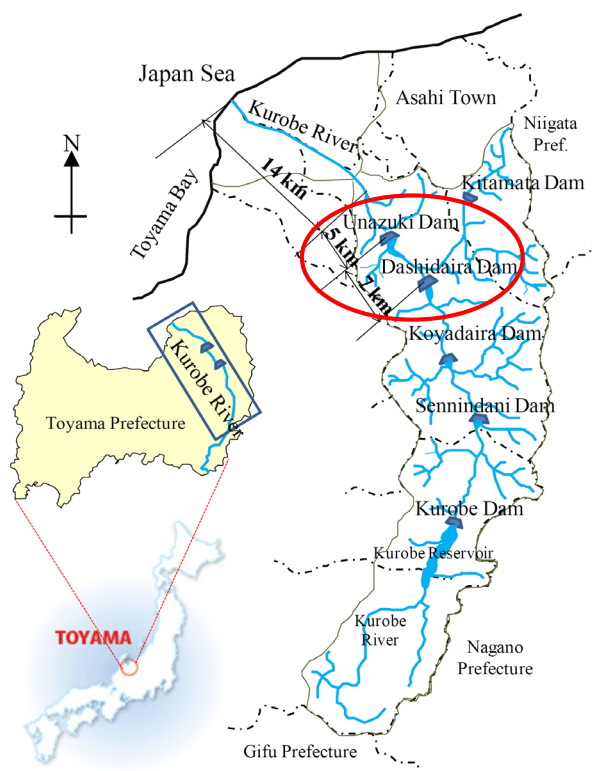


Figure 1.8 Site map of Kurobe River and location of the study area (Kantoush et al., 2010).

In 1991, the first flushing operation in Dashidaira reservoir was conducted that resulted in a severe environmental impacts on the downstream area of the dam. While the organic matters in deposited sediments were deteriorated over six years, deposits were released during the flushing operation without sufficient water discharge. Since June 2001, a coordinated sediment flushing operation with water level drawdown (i.e. free-flow flushing operation) is performed in the first major flood during the rainy season (i.e. June to August) to decrease the negative environmental impacts. Until July 2014, 24 coordinated sediment flushing operation has been conducted in Dashidaira and Unazuki reservoirs (Minami et al., 2012). The sediment sluicing operation is carried out at the successive bigger floods by the same reservoir operation to prevent additional sediment deposition in the reservoir (Sumi, 2006).

1.4 Research structure and methodology

According to the research objectives, first, a literature review is performed about the sediment transportation pattern (i.e. erosion and deposition) during the free-flow flushing operation. Consequently, related theoretical background about the physical process governing the morphological bed changes in prototype reservoirs (i.e. Dashidaira and Unazuki) would be highlighted. Besides, representative 3-D numerical simulations of free-flow flushing in other reservoirs around the world were discussed to use the relevant outcomes in this study.

Secondly, the theoretical and mathematical features of a CFD program called SSIM 2 are described in detail which implements a 3-D numerical model of flow field in three dimensions for turbulent flows and perform the sediment transport computation for simulating the morphological bed change.

Afterwards, to gain a more detailed insight on the physical process governing the morphological bed changes caused by shallow waters, the velocity field in shallow reservoirs with different geometries and also different bed conditions (i.e. fixed and equilibrium deformed bed) is simulated. Then, the flushing channel formation and evolution are simulated in the aforementioned reservoirs. Numerical outcomes are compared with the available experimental measurements (i.e. surface velocity and bed topography) and 3-D CFD code is verified. Relation between the reservoir geometry, flow pattern, and the flushing efficiency is figured out and through the achieved insight the study is expanded to the prototype case studies (i.e. Dashidaira and Unazuki reservoirs). Consequently, the flushing process during 2012 flushing operation is modelled, first, in Dashidaira reservoir which is narrower and steeper, and then in Unazuki reservoir, which is wider and bed slope is milder. Measured bed levels before and after the flushing operation are used for model calibration purpose. Moreover, a sensitivity analysis is carried out for the case of Dashidaira reservoir to disclose the effect of mainly empirical parameters on simulating the morphological bed changes, and to offset the limitations of the model for accurate simulation of all governing processes in the flushing operation. Taking into consideration of conducted sensitivity analysis and model calibration for Dashidaira reservoir, validation is performed for Unazuki reservoir using the achieved range of empirical parameters through the mentioned sensitivity analysis.

Based on the insight obtained through the simulation of flushing process in both physical and prototype scale reservoirs, a few scenarios (i.e. soft and hard measures) are numerically implemented in the verified model of Dashidaira and Unazuki reservoir to increase the flushing efficiency. Soft measure scenarios include modifications in water levels and discharge rates, and hard measure scenario focuses on changing the flow pattern utilizing flow guiding structures to affect the flow pattern and morphological bed changes. Results are compared to discover the advantage and disadvantage of each type of measures.

Finally, a summary for setting up the numerical model of study cases using SSIM 2 program, for describing the morphological process governing the flushing process, and for optimizing the future flushing operation is provided. Furthermore, recommendations for further development of this study on specific categories are supplied.

1.5 Outline of the thesis

The present study is divided into 7 chapters summarized as follows :

- **Chapter one** : presents an introduction to the study framework, describing the major concerns and motivations to conduct the study, showing the methodology to organize each step, and objectives of the current study.

- **Chapter two** : Includes a brief literature review on the reservoir sedimentation as the main cause for performing the flushing, on different types of flushing and their feasibility domain and their significance, and also on the highlighted feature deal with the flushing. Also contains the limited background on 3-D numerical studies conducted on free-flow flushing worldwide and lessons that can be learned from each case.
- **Chapter three** : Contains the governing theories and equations used in the 3-D CFD model for computing the hydrodynamics and sediment transport according to the objectives of this study.
- **Chapter four** : Includes 3-D simulation of flow velocity field in the physical model of rectangular shallow reservoirs with different geometries and also different bed conditions (i.e., flat and misshaped bed) and presents also the flushing channel formation and development in these reservoirs under the movable bed condition.
- **Chapter five** : Contains the numerical simulation results corresponds to the free-flow flushing in both Dashidaira and Unazuki reservoirs using the boundary condition of performed operation in June 2012 ; including sensitivity analysis, model calibration, and assessment of surface velocity field and morphological bed changes.
- **Chapter six** : proposing some measures and scenarios for increasing the efficiency of sediment flushing to scour a higher volume of deposited sediment from the reservoir.
- **Chapter seven** : covers the conclusions obtained from this study and provides the recommendations for future development of the current study.

Chapter 2 Sediment flushing: state-of-the-art and background review

This chapter provides the theoretical background which is available from the literature about the sedimentation process and features of sediment flushing in dam reservoirs. Although the main focus of this chapter is on free-flow sediment flushing from dam reservoirs, it is necessary to have an insight on the sedimentation process in reservoirs since sediment deposition is the main reason of sediment flushing. In addition, available literature about 3-D numerical modeling of flushing operation in prototype case studies has been reviewed. The characteristics and outcomes obtained from numerical modeling of each case have been highlighted to finally use for setting up and verification of free-flow flushing simulation in Dashidaira and Unazuki reservoirs.

2.1 Overview of sedimentation processes in dam reservoirs

When a natural stream enters the upstream zone of a reservoir, the cross-section area increases that causes the increase of flow depth and decrease of velocity. Consequently, the sediment transport capacity of the flow decreases and sediment load begins to deposit and forms a delta. Coarser materials are deposited first near the headwater area of the reservoir while finer ones transported to the deeper areas nearby the dam. Morris and Fan (1998), divided the sediment deposition into three distinguished zones including topset, foreset, and bottomset as shown in Figure 2.1. Relatively coarser materials are mainly transported in the form of bed load and are deposited along the topset. Topset beds correspond to delta deposition of rapidly settling sediment. Finer materials are transported downstream in the form of suspended load or density currents and forms the bottomset beds.

Foreset deposits represent the advancing front of delta into the reservoir. As more sediment enters the reservoir, the bottom set gradually increases in thickness and the foreset moves forwards (Palmieri et al., 2003). While topset beds may contain both coarse and fine sediment, the bottomset is mainly covered by fine sediment. However, tributary inflow, reservoir drawdown, slope failure and large floods may deliver coarser fractions of sediments to the bottomset zone.

Fan & Morris (1992a) also concluded that reservoir deltas have the following properties :

- 1- The bed slope is significantly different along the foreset compared to the topset so that can be differentiated based on the existing slope.
- 2- Deposited bed material type along the topset is considerably coarser than along the foreset.
- 3- The elevation difference in transition zone between the topset and foreset is a function of reservoir operation rules and maximum operation level.

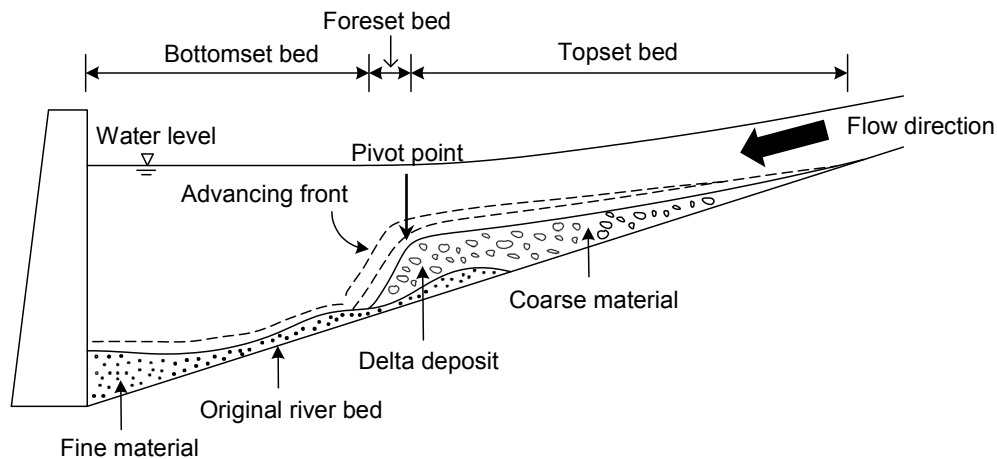


Figure 2.1 A Schematic view of longitudinal deposition in a reservoir (Modified from Morris and Fan, 1998 ; Sloff, 1991)

The reservoir longitudinal deposition pattern and subsequently the geometry of deposit is different case by case and depends on wide variety of factors including: watershed sediment production, land use, rate of inflowing sediment load, the volume of deposits, the bed material sizes and its distribution, flood frequency and discharge, geometry of reservoir (e.g slope of the valley and length of the reservoir), shape of structures in reservoirs, the capacity-inflow ratio of a reservoir, reservoir operation (Morris and Fan, 1998; Sloff, 1991). Morris and Fan (1998) classified the longitudinal sediment deposition pattern into four general types as shown in Figure 2.2. They also stated that combined patterns may exist together in different places of the reservoir.

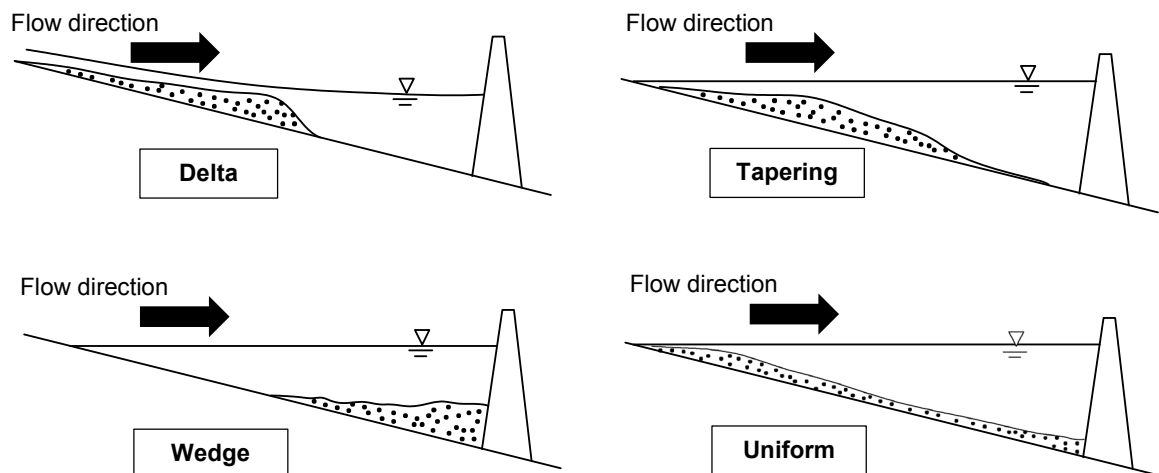


Figure 2.2 Four general longitudinal deposition pattern in reservoirs (Modified from Morris and Fan, 1998).

As mentioned before, Delta-shaped deposits are developed by coarser fractions of sediment which settle rapidly at the head of the reservoir. They can also contain the finer type of sediment such as silt. The Wedge-shaped deposits are caused by the transport of finer fractions of sediment by turbidity currents to nearby area of the dam. This type of deposition pattern can be found in small reservoirs with a large inflowing sediment load and also in large reservoirs operated with a low water level during flood events that carries most of the sediment load close to the dam site. The Tapering-type is prevalent in

long reservoirs which are usually operated at a high reservoir level. Uniform deposits are not common but occasionally occur in narrow reservoirs with frequent water level fluctuation and small load of inflowing fine sediments.

Beside the longitudinal deposition pattern along the reservoir, the lateral deposition pattern also should be considered. Morris and Fan (1998) stated that lateral deposition pattern in cross-sections of the reservoir usually initiates by sediment deposition in the deepest part of each cross-section. Concentration of deposition pattern in the deepest part of each cross-section finally creates a near-horizontal bed floor. Three diverse processes play a role in sediment focusing into the deepest part of each cross-section: 1- turbid density currents that transport and deposit the sediments along the thalweg. 2- logarithmic vertical concentration profile within the water column which concentrates the suspended sediment in the deepest part of the water column. 3- If suspended sediment is uniformly distributed throughout the cross section and settles vertically, sediment deposition thickness will be proportional to water depth. However, many 1D models assume the uniform spreading of the sediment deposition over the reservoir bed floor (Sloff, 1991). Amongst the mentioned processes, the lateral deposition pattern which is developed by turbid density currents play a significant role in explaining the lateral sediment deposition pattern in reservoirs compared to other processes. Figure 2.3 demonstrates the uniform spreading of deposits over the bed floor and also proportional deposit pattern with respect to the water depth

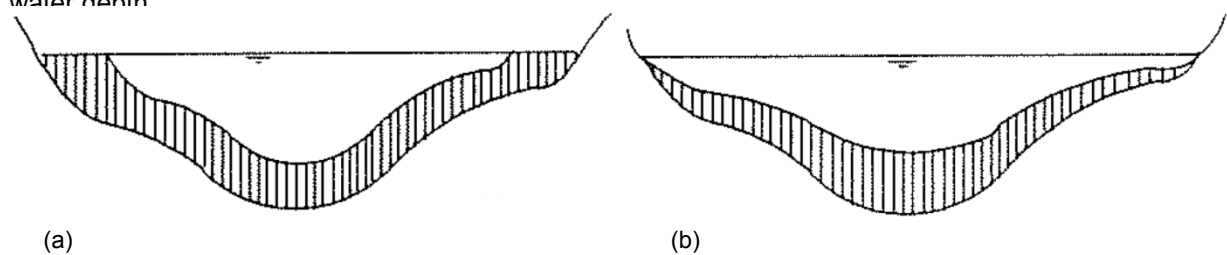


Figure 2.3 Different lateral deposition patterns over the cross-sections : (a) Uniform spread of deposits and (b) proportional to the water depth deposits.

2.2 Types of sediment flushing with water level drawdown

Hydraulic flushing involves the water level drawdown by opening the bottom outlets of the reservoir to establish the riverin flow along the reservoir, which erodes a channel through the deposited sediments in the reservoir and flushes out both fine and coarse sediments (i.e. size ranged between clay to gravel) (Morris and Fan, 1998). When the water level is dropped down, the flow velocities in the reservoir are increased to a magnitude that deposited sediment are re-mobilized and then transported through the bottom outlets (Palmieri et al., 2003). Sediment flushing through dam reservoirs is classified as a hydraulic measure to mitigate the reservoir sedimentation. Density current venting and sediment routing (i.e. sluicing) are also fall in this type of measures. The purpose of sediment flushing is to scour and release the sediment after it has been accumulated while sediment routing (i.e. sluicing) and density current venting try to prevent additional deposition during a flood. Two types of sediment flushing with water level drawdown can be performed : flushing with complete drawdown or free-flow

flushing and partial drawdown flushing. Properties and features of each type are described in subsections below.

2.2.1 Free-flow flushing (flushing with complete drawdown)

This type of sediment flushing can be performed in flood or non-flood season. Free-flow flushing in flood season is more effective since a higher magnitude of discharge would be available which can erode a higher volume of sediment and meanwhile the new sediment load coming with flood is also released (Morris and Fan, 1998). Also, from environmental point of view it is preferred since the sediment concentrations in a unit volume would be reduced. The free-flow flushing operation is mainly feasible in hydrologically small reservoirs where ratio of capacity to inflow is less than 0.3 (e.g. Dashidaira and Unazuki reservoirs in Japan) since the small capacity allows the reservoir to be refilled rapidly after closing the bottom outlets to terminate the flushing. Palmieri et al. (2003) and Atkinson (1996) stated that during the free-flow condition (i.e. riverin flow), the water level should be kept close to the bed for a significant length of time with small fluctuations to activate the sediment mobilization and increase the total volume of flushed out sediment. Schematic illustration of flushing with complete drawdown (i.e. free-flow flushing), presented by White (2001), has been shown in Figure 2.4. Development of free-flow condition and formation of flushing channel during the free-flow flushing operation in 2003 at Dashidaira reservoir has been shown in Figure 2.5.

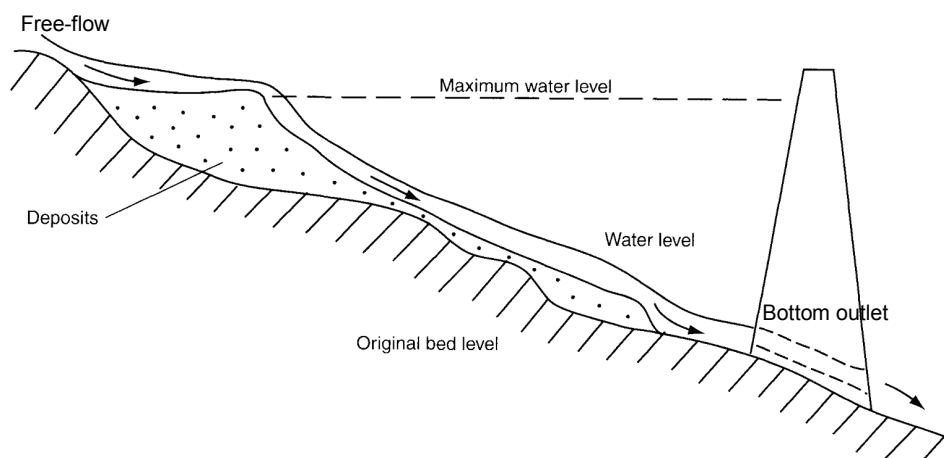


Figure 2.4 Schematic figure of flushing with complete drawdown (White, 2001).

According to Morris and Fan (1998), three different and successive stages can be distinguished in each flushing event namely: drawdown, erosion (i.e. free-flow or riverin flow), and refill as illustrated in Figure 2.6. Drawdown stage may last few weeks or days in case of large reservoirs and few hours in case of small reservoirs. This stage, itself, consists of preliminary drawdown stage where the water level is lowered up to the minimum operational level and final drawdown stage where the water level decreased more through complete opening the bottom outlets. During the drawdown stage, deposited sediments in the head area of the reservoir can be mobilized and transported to the downstream area where they will be redeposited. The main part of erosion, sediment transport, and flushing out through dam occurs during the free-flow stage with low water head along the whole reservoir length due to the

increase in flow velocity and induced bed shear stress. As soon as bottom outlets are closed, refill stage starts and rising backwater causes the suspended sediment to deposit within the reservoir.

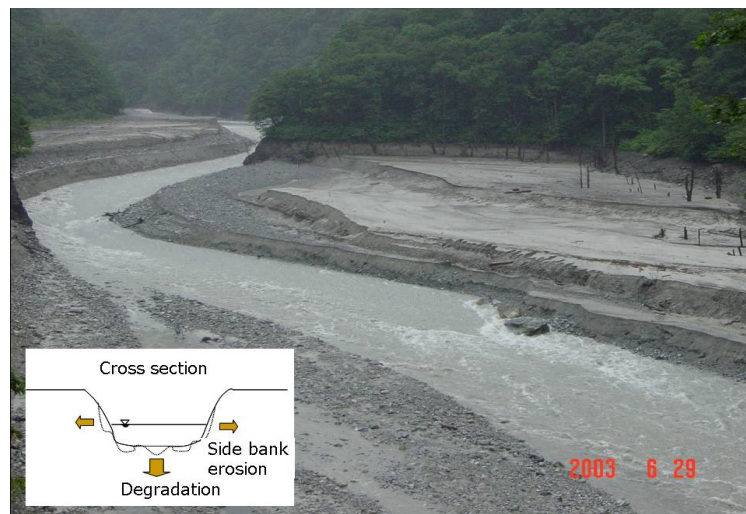


Figure 2.5 Free-flow condition and flushing channel development during the flushing operation in 2003 at Dashidaira reservoir (Sumi, 2006).

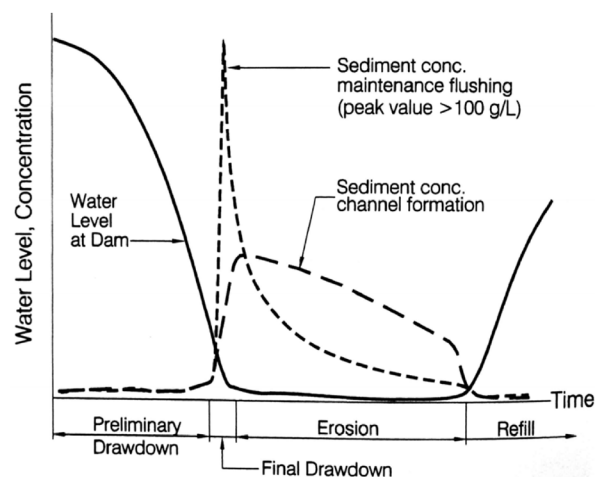


Figure 2.6 Water level rates and expected sediment load (i.e. concentration) in different stages of each flushing event (Morris and Fan, 1998).

2.2.2 Partial drawdown flushing

Partial draw-down flushing occurs when the reservoir level is drawn down only partially. Although, the flow velocity increases and subsequently sediment transport capacity becomes bigger in the reservoir, it is usually only enough to mobilize the deposited sediment within the reservoir, i.e., sediment is moved from upstream locations in the reservoir to locations further downstream and closer to the dam. Flushing with partial draw-down may be used to clear more live storage space and locate the sediment in a more favorable position for future complete draw-down flushing (Palmieri et al., 2003). Other effect would be formation of a conical scour hole in the vicinity of dam since the higher velocities are locally concentrated at the outlets. However, this hole may be refilled with coming sediments from upstream (Morris and Fan, 1998). To release the annual sediment load entering the reservoir, frequent partial-draw down should be implemented. Thus, it is not an effective approach for evacuating the annual

sediment deposition from a reservoir. Figure 2.7 shows longitudinal bed profile variation within a long-term partial drawdown flushing operation (White, 2001).

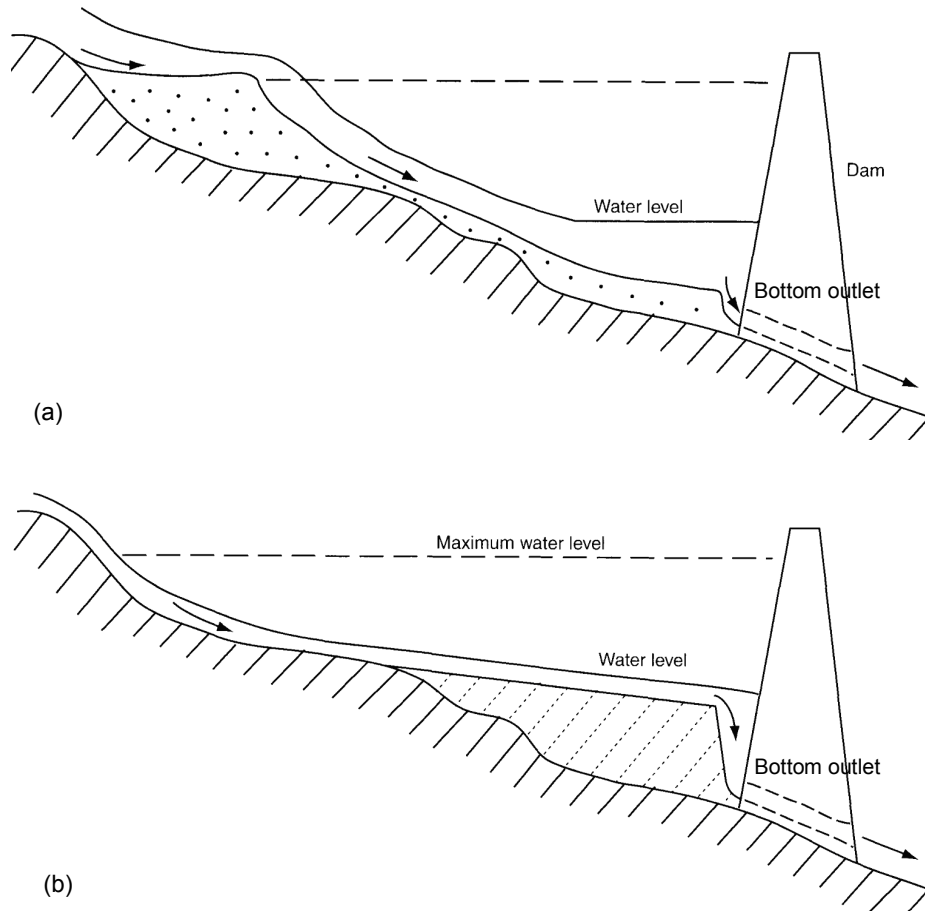


Figure 2.7 Longitudinal bed profile changes during different stages of partial drawdown flushing : (a) partial lowering of water level and initiation of local conic scour hole in the vicinity of the outlet ; (b) sediment transport from upstream reaches to areas closer to the dam (White, 2001).

2.3 RESCON model and feasibility of flushing

RESCON model is a computer model which applies RESCON approach and has been developed as a demonstration tool. The main algorithm that was implemented in the model is an economic optimization function with engineering relationships that make it possible to quantify the basic parameters. The general goal of the RESCON model is to select a sediment management strategy that is technically feasible in the target reservoir and also can maximize the economic benefits. Model considers the sediment flushing, hydrosuction, traditional dredging, trucking, and “do-nothing” scenario as different options for sediment management strategy in a reservoir (Palmieri et al., 2003).

Model assumes that traditional dredging and trucking is feasible in all reservoirs and assess the technical feasibility of flushing and hydrosuction, and then proceeds with economic optimization function. In addition, environmental and social considerations are taken into account in the model. However,

model does not assess the watershed management, sediment bypass, and operating rules of the reservoir (Palmieri et al., 2003).

Basson and Rooseboom (1997) established a diagram based on the observations from many dams to suggest which operation model can be more effective for a sustainable management of a reservoir. Horizontal axis represents the hydrological size of the reservoir, which is an indicator of the available amount of water to use in sediment management and vertical axis represents the ratio of sedimentation to storage capacity, which is an indicator of reservoir life period. The diagram has been shown in Figure 2.8.

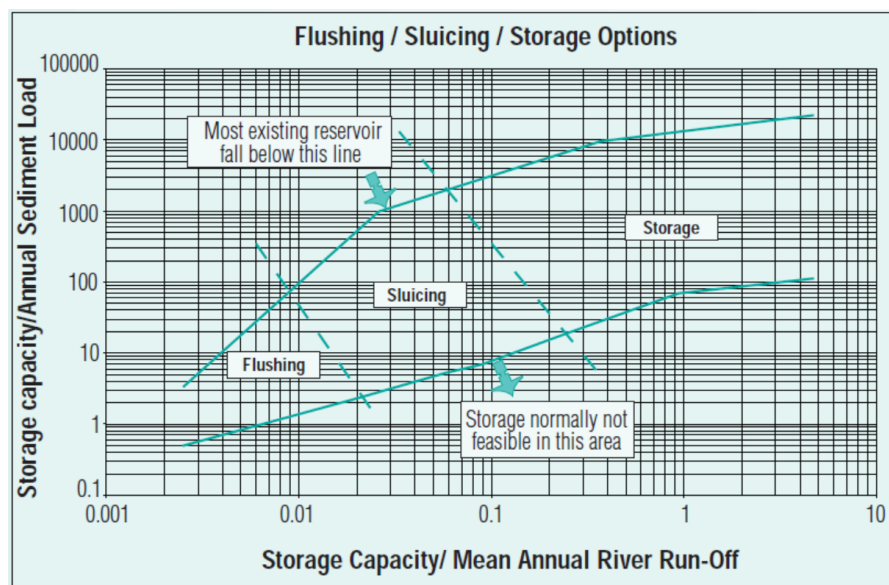


Figure 2.8 Diagram for preliminary checking the reservoir operation type (Palmieri et al., 2003)

Opposite to the design life approach which assumes that project costs are recovered by achieved benefits just during its service life, RESCON model employs life cycle management approach to conduct a sustainable use of reservoirs. If sustainable use is not possible, then, decommissioning is planned at the optimum time, and decommissioning fund is allocated. Design life approach does not consider what happens to project after its life ends and of course, in case of reservoirs, there would be no any instruction for the subsequent issues generated by the remaining life-ended reservoir. The program structure in RESCON model has been outlined in Figure 2.9.

The flushing module in the RESCON model uses the approach introduced by Atkinson (1996), first, to develop the criteria for assessing the feasibility of flushing, and then to calculate the volume of flushed out sediment from the reservoir. More details about the feasibility criteria can be found in Atkinson (1996).

Atkinson (1996) uses the sediment transport capacity formula developed at Tsinghua University in China, which has been reported in IRTCES (1985) :

$$Q_s = \psi \frac{Q_f^{1.6} S^{1.2}}{W^{0.6}} \quad (2.1)$$

where Q_s denotes sediment transport capacity in t/s, ψ is an empirical sediment parameter, Q_f is the flushing discharge in m^3/s , S is bed slope and W is the flushing channel width in m.

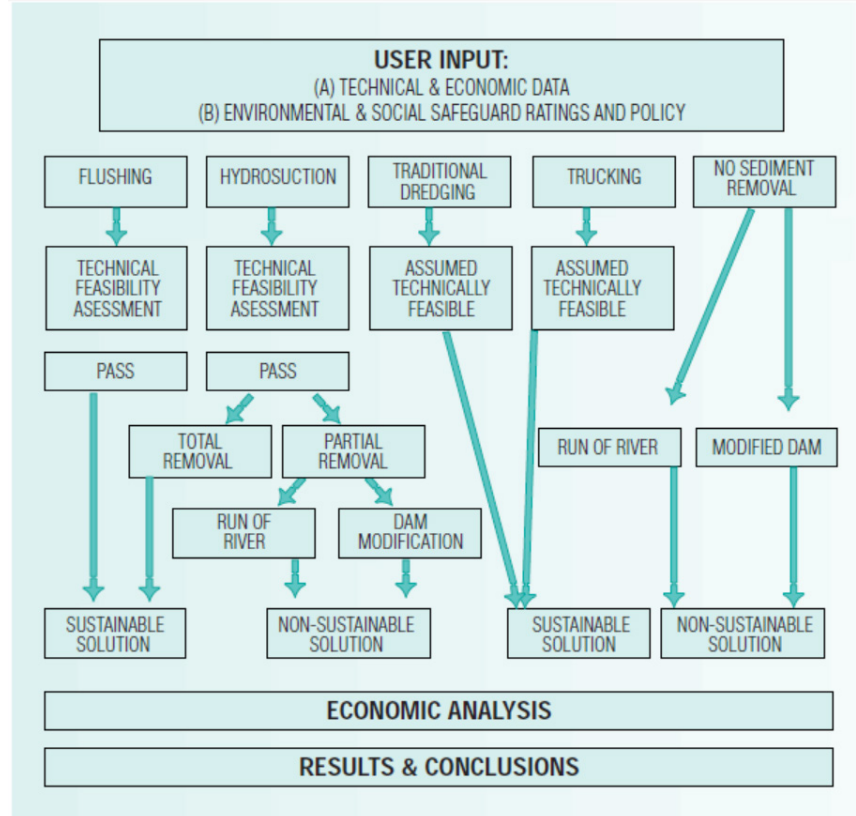


Figure 2.9 The structure of RESCON program (Palmieri et al., 2003).

When this formula is used for a reservoir, for the first time, the flushing channel width would be unknown. Thus, the flushing channel width should be estimated first. Due to the positive correlation between flushing discharge and flushing channel width in prototype cases, a line has been fitted to the observed points and an equation has been established :

$$W_f = 12.8 Q_f^{0.5} \quad (2.2)$$

where W_f denotes the flushing channel width in m and Q_f is the flushing discharge.

However, used data for developing equation (2.2) were collected from reservoirs in China where flushing techniques and sediment characteristics may not be representative for other regions in the world. For instance, the flushing channel width of Dashidaira reservoir in Japan is overestimated when equation (2.2) is used (Kantoush et al., 2010a). Thus, formula should be verified for other regions (e.g., for Japanese reservoirs).

RESCON model uses limited options for reservoir geometry and floodborne sediment inflow to a reservoir as input data. In addition, employed algorithms in flushing module just gives the volume of flushed out sediment. Thus, RESCON model can not provide outputs such as flow field distribution pattern and also flushing channel evolutionary pattern during a flushing operation. Both mentioned outputs are necessary for assessing the consequence of a flushing operation in a reservoir. Although, the volume of flushed out sediment provided by RESCON model is a useful information, it is not sufficient for monitoring the impacts of an upcoming sediment flushing operation in a reservoir.

2.4 Highlighted features governing the free-flow flushing operation

Influence of dominant factors on feasibility and success of a flushing operation has been outlined by Atkinson (1996), Morris and Fan (1998), Shen (1999) and White (2001). A brief summary about highlighted factors, which are directly involved in sediment flushing, and also about inherent features of flushing operation have been reviewed herein.

2.4.1 Reservoir geometry and development of flushing channel

Geometrical features of a reservoir (e.g., shape and size) have a significant effect on the feasibility of the flushing operation and its success. In wide reservoirs (e.g., with a large ratio of reservoir width to length), during a free-flow flushing operation, a narrow distinctive channel with a limited lateral erosion is scoured through the deposits, that produces a channel and floodplain-type geometry. While the deposited sediment in the already formed flushing channel can be eroded during a successive flushing operation, sediment deposited on the floodplains during impounding periods will continuously accumulate. In a narrow reservoir where the flushing channel occupies virtually the entire pool width, it is possible to restore and maintain most of the original storage volume (Morris and Fan, 1998). Subsequently, the free-flow flushing operation is more efficient in narrow reservoirs. Figure 2.10 illustrates the flushing channel formation in both narrow and wide reservoirs.

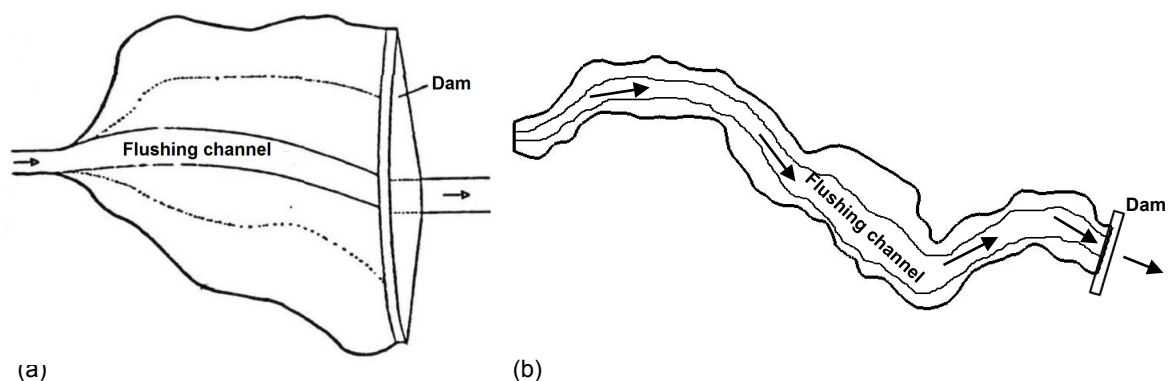


Figure 2.10 Plan view of established flushing channel in (a) a wide reservoir (Shen, 1999) ; (b) a narrow reservoir.

The flushing channel location in wide reservoirs is determined by the flow pattern, upstream and downstream condition that controls this pattern. However, it is difficult to predict the flushing channel

location due to the sensitivity of the flow pattern to the initial boundary condition and also the reservoir geometry that can be changed during the time (Sloff et al., 2004). Application of auxiliary flushing channels in addition to the main flushing channel can be considered as a measure for removing more deposits from the floodplain area in wide reservoirs.

2.4.2 Development of shallow flows domain

Shallow flows are described as a flow condition in which the vertical dimension of the fluid domain is noticeably smaller than its horizontal dimensions (Yuce and Chen, 2003). As shown in Figure 2.6, during the free-flow stage of the flushing operation, the water level decreases noticeably and therefore the water depth in flushing channel would be small. Subsequently, shallow water flow condition emerges and plays a significant role in removing the deposited sediment from the reservoir. The shallow flow pattern may become unstable, due to the effect of geometric and hydraulic boundary conditions, which contributes in development of large-scale transverse motions and turbulent coherent structures. As a result, the sediment transportation pattern (i.e. erosion and deposition) is significantly affected by the flow velocity field and different morphological processes can be developed in the reservoir.

Kantoush (2007) presented a comprehensive review of experimental tests in a series of shallow reservoirs with transverse flow motions in the symmetric channel expansions. His study figured out 3-D features of the flow structure in shallow reservoirs (i.e. secondary flows and 3-D stretching vortices). Although 2D depth-averaged numerical models can reasonably reproduce the surface velocity pattern, they cannot directly simulate the secondary current effects and velocity variations over the flow depth and particularly on deformed beds of shallow reservoirs. The complexity of 3-D flow patterns is further magnified over the existing bed forms, on the deformed bed, that is obtained after flushing and lowering the water level. Figure 2.11 illustrates the shallow flow condition during the free-flow stage at Dashidaira reservoir.



Figure 2.11 Shallow flow condition during the free-flow stage of flushing operation at the middle reach of Dashidaira reservoir (Photo taken by author).

2.4.3 Erosion patterns during the sediment flushing

There are two major types of erosion patterns that usually occur during a flushing operation : 1- retrogressive and 2- progressive. Retrogressive pattern propagates towards upstream, along a channel, from a zone with high slope to a zone with lower slope and lower erosion rate. The highest rate of erosion occurs along the steep drop at the downstream end of the deposit, causing this area of maximum erosion to move upstream through a headcutting process. The point where bed slope changes is called pivot point or nickpoint and sometimes it is observed visually (Morris and Fan, 1998). Figure 2.12 shows the schematic retrogressive erosion pattern in a flume experiment with unconsolidated materials and characteristics of this erosion type in a prototype reservoir.

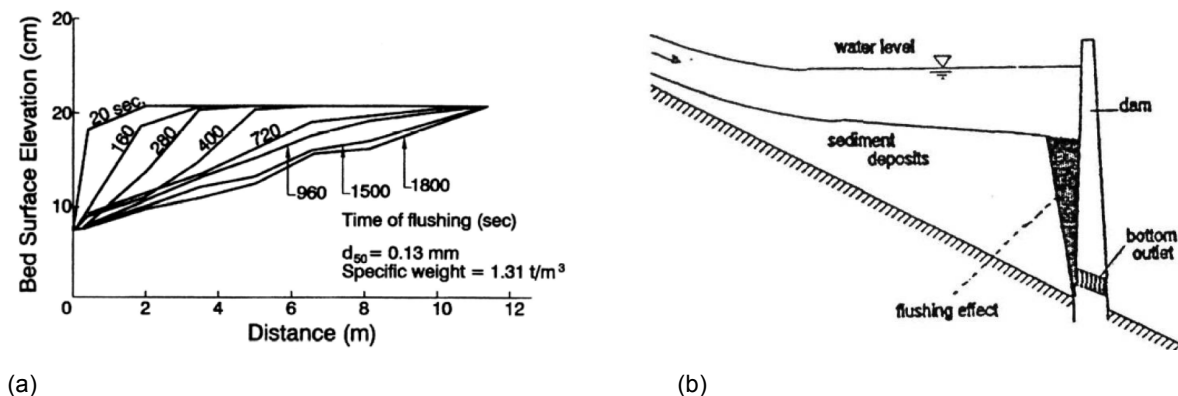


Figure 2.12 Longitudinal profile of a retrogressive erosion pattern in : (a) a flume experiment (Morris and Fan, 1998) ; (b) a real scaled reservoir (Shen, 1996).

When deep outlets are opened, a flow pattern develops across the deposits nearby the dam which initiates the retrogressive erosion pattern, creates a pivot point moves upstream rapidly depends on the nature of deposited sediment (e.g., cohesive or non-cohesive, consolidated or unconsolidated) and erosional forces (Morris and Fan, 1998). For example in case of Sefid-Roud Reservoir in Iran, the erosion rate towards upstream was 100 m/day in cohesive materials (Tolouie, 1993). In large and long reservoirs the retrogressive erosion pattern towards upstream may last few months along the reservoir (e.g., 2 months in Sanmenxia reservoir across the Yellow River, China) while it may take few in hours in small reservoirs (e.g., 24 hours in Hengshan Reservoir, China). The main reason for initiating the retrogressive erosion pattern is change in hydraulic energy cause by discontinuity in the longitudinal profile. In prototype reservoirs, retrogressive erosion initiates with a serious erosion in area with the highest slope (i.e., foreset area) and pivot point moves upstream that causes the slope to decrease in foreset area. Meanwhile, channel erosion causes the slope to increase in the topset area. This process continues until a unified slope is achieved. Afterwards, the retrogressive erosion pattern stops and another progressive erosion pattern starts.

In progressive pattern, erosion occurs uniformly along the whole length of channel instead of concentrating at the downstream area nearby the outlets. Overall, when the suspended sediment concentration in flowing water is smaller than the sediment transport capacity, flow will start to erode the bed materials. Initially, the erosion rate of bed material is high since the clear water has a high

sediment transport capacity. As the flow moves downstream and erodes sediment, the water flow capacity to erode and transport additional sediment will decrease, and becomes zero. In such a condition, progressive erosion pattern erodes a high amount of bed material from the upstream end and less amount from the downstream end. Opposite to retrogressive pattern, the bed erosion develops from upstream towards downstream in progressive pattern. If the upstream end area is covered with coarser materials, the progressive pattern can not be effectively established. Figure 2.13 illustrates the schematic progressive erosion pattern in an experimental study with unconsolidated materials.

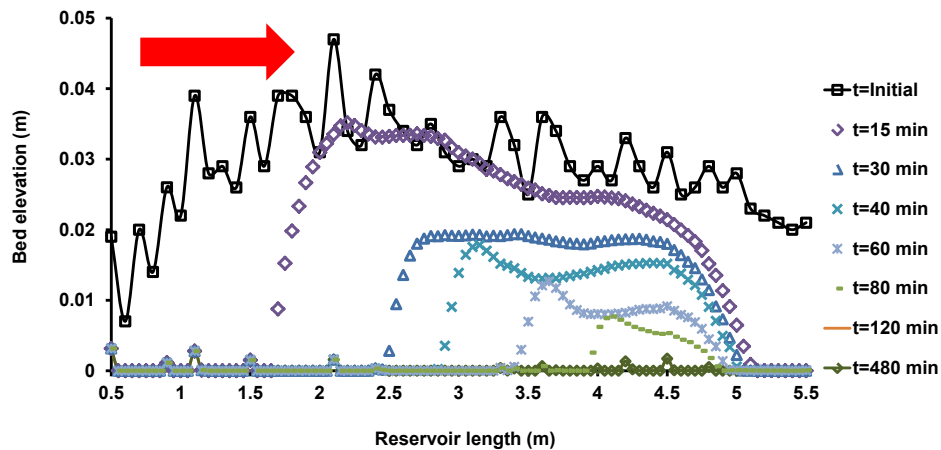


Figure 2.13 Longitudinal profile of a flushing channel thalweg showing the progressive erosion pattern in an experiment study (Esmaeili et al., 2014a).

During the progressive erosion pattern, lateral (i.e. widening) and vertical (i.e. deepening) erosion pattern can be distinguished. Both lateral and vertical erosion pattern were observed in experimental studies and also in real free-flow flushing operations (Esmaeili et al., 2014a ; Kantoush et al., 2010). The experimental and numerical study on flushing channel formation in shallow rectangular reservoirs conducted by Kantoush & Schleiss (2009) and Esmaeili et al. (2014a), respectively revealed that widening and deepening rate decreases remarkably after formation of initial flushing channel along the whole reservoir length. It was also found that slow widening and deepening stage initiates after formation of initial flushing channel due to the concentration of accelerated shallow water flow just in the already formed flushing channel (Esmaeili et al., 2014a). In prototype reservoirs, the sediment size and consolidation rate may increase as flushing channel is further deepened, that makes it difficult to erode the existing bed material from the bottom of the channel. Morris & Fan (1998) stated that channel may be deepened by using either smaller or larger flows, but widening is achieved by large flows only. Toluie (1993) and Randle & Lyons (1995) observed that using large flows contributes in widening the flushing channel in fine sediment deposits and also non-cohesive coarse delta deposits, respectively. Figure 2.14 demonstrates the widening and deepening pattern of a flushing channel in a physical model study along with concentration of shallow and accelerated flow in the flushing channel during a free-flow condition.

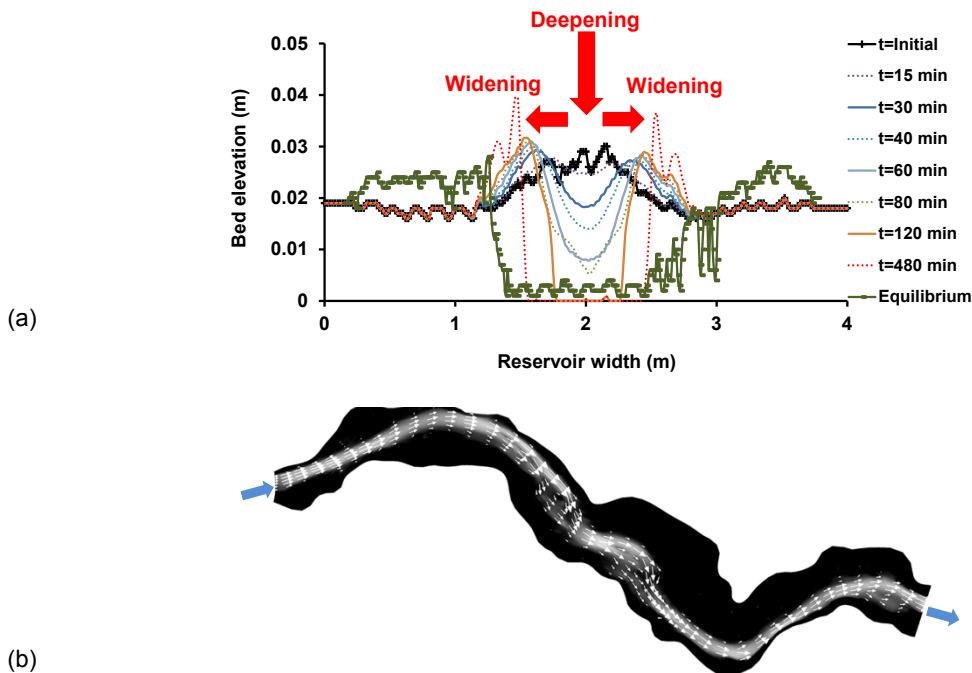


Figure 2.14 (a) Simulation of widening and deepening evolutionary process in a cross-section of the flushing channel in a physical model (Esmaili et al., 2014a); (b) shallow flow concentration in already formed flushing channel at Dashidaira reservoir.

2.4.4 Sediment characteristics and transportation during the free-flow flushing

Many sediment transport formulas utilizing different concepts exist for sediment transport computations, but most of them are based on some assumptions for simplification such as assuming steady uniform flow condition in prismatic channels. Moreover, they have been developed mainly for cohesionless sediments and assume that equilibrium exchange condition between fluid and bed has been satisfied. But, the mentioned assumptions and conditions are not the case in reservoirs (Morris & Fan, 1998).

Usually, cross-sections are not prismatic in reservoirs and flow is also not steady and uniform. The entering flow into the reservoir deposits the sediment load, and therefore the equilibrium exchange assumption between fluid and bed is not true. In many reservoirs, large amount of deposited sediments are cohesive, which presents difficulty in sediment transport and subsequently the erodibility of due to the existing cohesive forces.

Sediment transportation quality and quantity during a flushing operation strongly depends on the sediment characteristics. Small sized sediment such as sands can be easily transported and flushed out from a reservoir. Coarser materials such as cobbles and gravels are eroded sometimes from reservoir head areas but they are usually deposit again in further downstream areas (i.e., close to the dam) where the flow velocity reduces and depth is higher compared to other areas (e.g., Unazuki and Dashidaira reservoirs in Japan or Alpine reservoirs in Europe). This process can be observed during a free-flow flushing operation, and therefore it is obvious that a partial drawdown flushing is not a suitable option for eroding the coarse materials from reservoir head areas. Figure 2.15 demonstrates the sediment size distribution in head and tail area of Dashidaira reservoir. As can be observed, coarse

materials exists in area very close to dam, that remains from last flushing operations. In case of fine materials such as silts and clays, the cohesive forces plays a major role in their erosion and transportaion since these forces can be larger several orders of magnitude from the gravitational forces. Consolidation of fine materials also make them more difficult to be eroded since their resistance against erosion increase. Thus, sediment cohesion escalate the resistance against erosion and may reduce the flushed out sediment volume noticably (Morris & Fan, 1998).

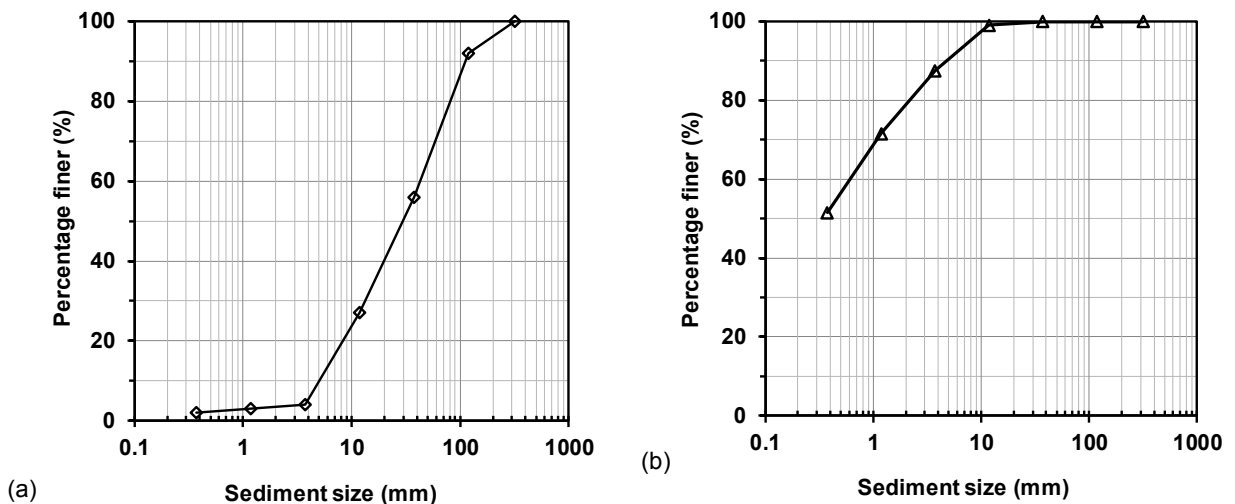


Figure 2.15 Sediment size distribution extarcted using onsite bed samples in (a) head area of Dashidaira reservoir and (b) nearby the dam.

2.4.5 Design issues of bottom outlets for free-flow flushing

The main concept in a free-flow flushing operation is lowering the water level mainly by opening the bottom outlets to develop a free-flow condition, which increases the induced bed shear stress abd initiates the erosion process in already deposited sediments in a dam reseervoir. Subsequently, it is important to have a low water level as it is possible during the flushing operation which can be acheived through installing the bottom outlets with sediment flushing facilities in an appropriate position.

In general, flushing sluices should be located as deep as possible and should be as wide as possible. Two side-by-side arrangement of bottom outlet is prefered since makes a lower backwater (Morris and Fan, 1998).

The bottom outlets, itself, is a limiting factor for implementing the free-flow flushing in many existing reservoirs. If the bottom outlets have not enough capacity, first, the full drawdown condition can not be established to develop a free-flow condition and, second, the large amount of water-sediment mixture can not be flushed out. Installing the bottom outlets in a suitable position with an enough capacity on an existing dam can play a major role in recovering the lost storage capacity as it was experienced in Mangaho River project in New zealand where 75% of the accumulated sediment within 25 years of resservoir operation could be flushed out (Jowett, 1984). Figure 2.16 shows the position of bottom outlets at Gebidem and Dashidaira reservoir.

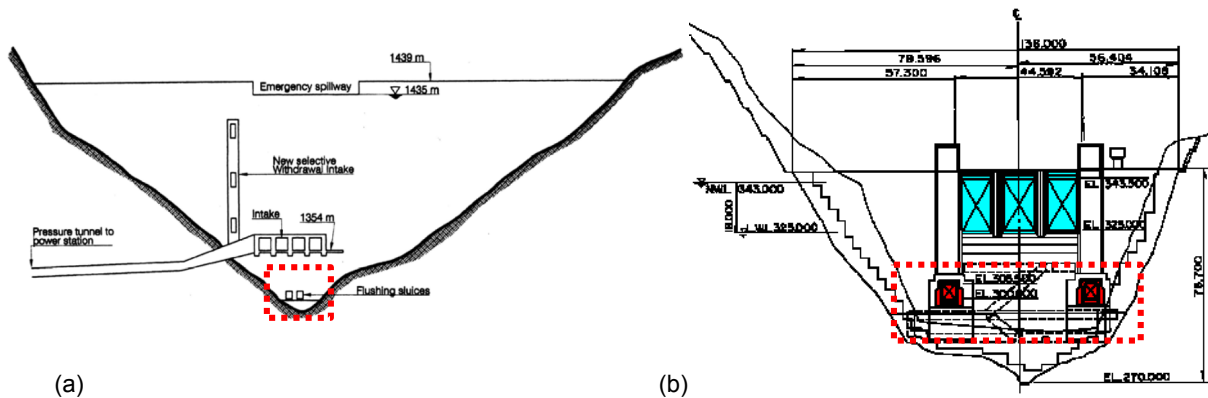


Figure 2.16 Location of bottom outlets, shown in dashed-lined rectangle, used for flushing purpose at (a) Gebidem dam in Switzerland (Morris & Fan, 1998) ; (b) at Dashidaira reservoir in Japan.

2.4.6 Hydrological conditions required for free-flow flushing

Hydrological conditions (e.g., available runoff discharge) during and after the flushing are other important factors with major influence on the feasibility and success of a flushing operation. Enough water discharge should be available during a free-flow flushing operation to onset the erosion process of deposited materials and also to flush out the eroded materials from the reservoir. White (2001) and Palmieri et al., (2003) stated that flushing discharges two times bigger than the mean annual flow may be required for a successful flushing. Using a higher amount of discharge during a flushing operation specially in the free-flow state, not only contributes in a higher erosion of deposited material, but also tends to decrease the sediment concentrations, which modulates the negative environmental impacts of flushing on downstream areas of the dam.

Reservoirs with big annula run-off compared to the volume of reservoir (i.e., more than 3 times), are more suitable for implemntng a regular annual flushing operation. In other words, flushing could be an option for hydrologically small reservoirs such as Gebidem reservoir in Switzerland with capacity/inflow ratio of 0.021, Unazuki reservoir in Japan with capacity/inflow ratio of 0.03. Figure 2.17 depicts the catchment of Kurobe River where Dashidaira and Unazuki reservoir along with other reservoirs have been located across this river and a regular flushing operation is being implemented in both Dashidaira and Unazuki reservoirs. Regular flushing operation would be effective during flood seasons in monsoon areas or during late spring or summer months in areas with flood event due to the snow melt. In other words, in areas with distinct wet and dry season, required hydrological condition is acheived during the wet season when flood happens. When flood hapens in wet season, it is also possible to route the sediment laden flow through the reservoir during a flushing operation as a side advantage (Fan & Morris, 1992b). Other advantage of regular flushing during the wet season is reducing the required time for reservoir refill especially for larger reservoirs. However, arid araes with a low percipitation (i.e., desert environments) are not suitable for flushing operation, although a wet and dry season can be distinguished.

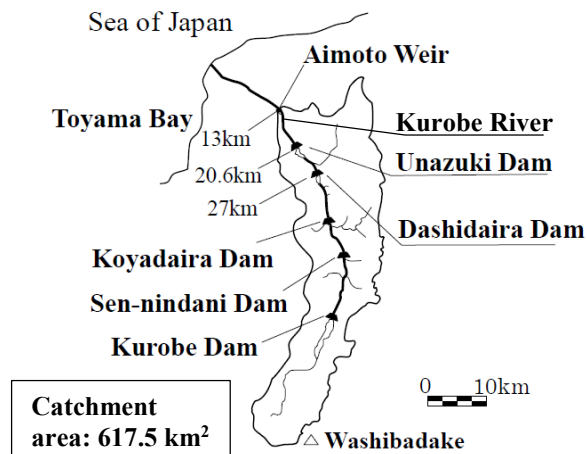


Figure 2.17 Catchment area of Kurobe River and located reservoirs across this river (Modified from Minami et al., 2012) .

The flushing operation process also can be adjusted due to the available amount of water source. In case of limited available water, a short period of flushing operation with a high discharge tends to be more effective than long period with a low discharge (Palmieri et al., 2003). When discharge is higher, a wider flushing channel may be formed and a higher amount of sediment can be eroded while a lower discharge over a longer period develops a narrow and deep flushing channel (Guo & Li, 1984).

2.4.7 Flushing efficiency definition

Flushing efficiency is defined here as the ratio of eroded volume of deposited sediment to the volume of water used during flushing over any specified time interval (Morris & Fan, 1998). The flushing efficiency depends on topographic position, outlet capacity, outlet elevation, characteristics of the inflow sediment, the mode of operation, time duration of flushing and flushing discharge (IRTCES, 1985).

From topography considerations, long, narrow reservoirs are better suited for flushing than short, wide and shallow reservoirs. Furthermore, reservoirs in the upper and middle reaches of a river are better suited for flushing, because the bed slope is small for the lower reaches and limiting the sediment transport. Other reason can be attributed to the larger flood plains of reservoirs in the middle and downstream reaches that flushing channel can not be developed over there and accumulated sediments are not able to be flushed out (White, 2001).

In free-flow flushing operation, the riverin flow condition with low water level has a remarkable effect on sediment mobilization and erosion. The riverin flow condition (i.e., free flow stage) should last long enough to erode the bed materials and, then, transport them, and finally flush them out from the bottom outlets. When the water quantity is enough, a higher flushing efficiency is expected, if the flushing period is longer. To do so, the bottom outlet capacity should be enough (e.g. as wide as possible) to develop the free-flow condition and the position of bottom outlet is preferred to be close as possible to the thalweg.

A short flushing period may be effective for entraining the fine sediments and on the other hand, longer flushing period with a higher discharge may be effective to remove coarser materials, which are accumulated in the reservoir head. In addition, erosion of coarser materials depends on the water level during a large discharge event. Deposition area of coarser materials in the reservoir head can be affected by the former delta dedeposition, which elevates the bed level in this area and reduces the bed slope. This process, itself, decreases the induced bed shear stress in this area and further decreases the erosion and transportation of coarser materials, that increase also the flood hazard (Morris & Fan, 1998). Finer materials could be eroded easily during the flushing operation, but coarser materials in head area of the reservoir can not be removed as easy as finer materials. It should be noticed that finer materials, which are deposited on the floodplain of a reservoir, outside of the flushing channel, have a less chance for erosion and flushing from the reservoir. Esmaili et al. (2015) concluded that discharge of flushed out coarser materials during the free-flushing operation is more sensitive to water level changes than variation in discharge rates.

Kantoush et al. (2009) and Esmaili et al. (2014) concluded that flushing efficiency is high at the beginning stage of flushing channel formation due to the widening and deepening procedure, but efficiency decreases after formation of the initial flushing channel because of a slow widening procedure. Any strategy that contributes in widening the flushing channel or increasing the number of flushing channels in flood plains (e.g., auxiliary channels in longitudinal or lateral direction) can increase the flushing efficiency. If additional water is available from an upstream reservoir, introducing pulses of flow discharge also may contribute in a higher erosion of coarser materials from the reservoir head and further transport to areas closer to the dam. This process may increase the chance of erosion for coarser materials that are already deposited in areas closer to the dam.

2.4.8 Economical features of free-flow flushing

Flushing and sluicing are efficient hydraulic techniques for sediment removal and restore the storage capacity (Liu et al., 2004). By using flushing operation, the lost storage capacity can be recovered and useful capacity of storage can be maintained without expenditure of dredging or mechanical removal (Atkinson, 1996). However, a large amount of water should be used, which decreases or stops the water supply for electricity generation and represents an economic loss. If the hydropower plants are become offline during the drawdown and refill stage, in flushing operation, the economic loss scale would be increased. Also, emptying of a large reservoirs which takes long time for refilling can impose huge economic losses by disturbing or stopping the various functions of a reservoir (e.g., electricity generation, irrigation and drinking water supply). In addition to these constraints, for some existing reservoirs, it is very costly to install the sediment flushing facilities since a major structural modification should be conducted. However, in some cases such as Gebidem reservoir in Switzerland, economical cost analysis (i.e., capital cost, unit cost of sediment removal, and water use) showed the advantage of flushing operation compared to other sediment management strategies especially from the water use amount point of view. This is because of already planned engineering work without need to an additional structural modification (Morris & Fan, 1998).

It is basically assumed that costs of flushing from various aspects should not exceed the benefits. This can be assessed by RESCON model which suggests a sustainable sediment management strategy in a reservoir with a special emphasis on the economic assessment. However, it is a challenge to maximize the economical benefits without conflicting with technical feasibility requirements and also social and environmental impacts.

2.4.9 Ecological impacts of free-flow flushing

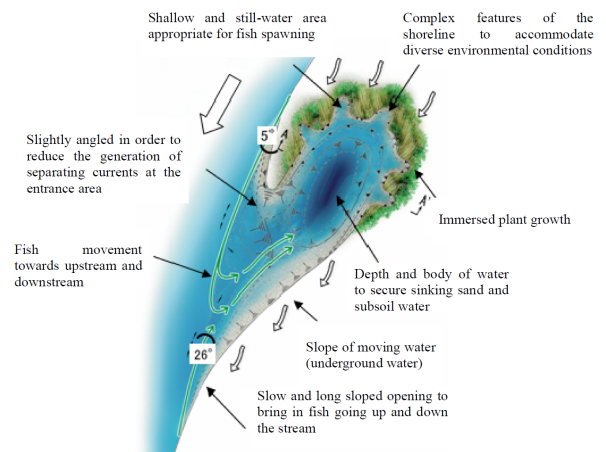
Beside the benefits of free-flow flushing for preserving the reservoir storage capacity, some adverse ecological and environmental impacts may happen if flushing is performed without considering proper mitigating measures. During a flushing operation, a noticeable amount of sediment flushes out from the reservoir. The flushed out sediment may be deposited in the stream channel, a reservoir, a water intake, irrigation channels etc, and the necessary discharge for preventing this excessive and localized deposition could be more complex to determine compared to the discharge needed for flushing out the sediment from a reservoir itself (Morris & Fan, 1998). The sediment deposition not only has an adverse effect on the proper function of river stream in downstream area of the dam through capacity reduction of the river channel, but also sediment deposition clogs the river bed and damages the habitat structure of aquatic animals (i.e., fish and benthos) (Schneider et al., 2006). In addition, the sediment concentration increases during a flushing operation and actually it would be much more than what occurs in a natural fluvial system, even when a natural flood occurs. Records reveal that sediment concentration can exceed 1000 g/L in some cases that will smother the aquatic animals by clogging their gills. Anoxia due to the depressed magnitude of dissolved oxygen caused by a high sediment concentration during a free-flow state also can potentially kill all the organisms. Other impacts on the downstream environment could be: changing the structure and population of benthos, reducing the available food source and changing the food chain for benthos and incorporating the deteriorated and toxin materials into the food source of benthos if the flushed out sediments contain agricultural, chemical and other type of toxins (Morris & Fan, 1998). If deposited sediments in downstream area after the flushing operation and the covered banks and river bed with fine materials are not washed out, the environmental impacts would be more severe. Thus, enough water discharge should be released to scour and wash downstream area after the flushing. This can be applied with releasing a high discharge magnitude during the reservoir refill period to rinse the fine materials (Sumi et al., 2009). Moreover, free-flow flushing can be done when a natural flood happens in the stream in order to represent a more natural condition as a mitigating measure. Because of high discharge value, deposited sediments can be eroded easier while the upper limit for Suspended Sediment Concentration (i.e., SSC) can be satisfied (e.g., free-flow flushing case in Kurobe River). It should be noted that flushing of coarser materials from the reservoir, during a free-flow flushing, would be beneficial for downstream ecosystem since gravel sized materials are used by Salmons for spawning and also used as the habitat for a wide variety of aquatic animals. However, other mitigating measures to save the aquatic animals life during the draw-down and free-flow stage, when the SSC is high, would be useful to reduce the negative environmental impacts.

Another practical approach for decreasing the negative environmental impacts is to assure that sediment concentration in downstream area does not exceed the upper limit of the concentration during a natural flood event (Scheuerlein, 1995). Considering this point, some regulations for upper limit of SSC can be defined by the authorities to reduce the negative environmental impacts of a flushing operation.

In case of Kurobe River in Japan, some evacuation facilities have been constructed artificially in lower reaches of the river (i.e., called Wando in Japanese) for aquatic animals so that fish and benthos can use them as a safe place to survive from the high sediment concentrations during a flushing operation (Sumi et al., 2009). Such kind of innovative and environmental friendly ideas beside of performing the flushing operation during a flood, and introducing a high discharge to downstream during the reservoir refill, can decrease the negative environmental impacts of a flushing operation on downstream areas. Figure 2.18 demonstrates the flushing out sediment laden flow during a flushing operation from the bottom outlets of Unazuki reservoir in Japan along with a schematic figure from an existing Wando.



(a)



(b)

Figure 2.18 (a) High sediment concentration during the flushing operation of Unazuki reservoir in June 2013 (Photo taken by author) ; (b) A conceptual illustration showing a wando structure in the Kurobe River (Sumi et al., 2009).

2.5 Simulation of sediment flushing process using CFD

Although physical models are useful tools to obtain a better understanding of the process during a flushing operation, they suffer from scaling effect of the study case and sediment particle sizes. Through a significant development in Computational Fluid Dynamics (i.e., CFD), numerical models are widely used as an alternative to avoid scaling problems of physical models and also because of an expected time and cost reduction (Chandler et al., 2003). Moreover, Fletcher (1996) concluded that CFD models have five main advantages compared to the physical models as reduction in design and development time, possibility to simulate a complex flow condition that could not be reproduced in physical models, availability of comprehensive details and information, cost-effectiveness, and lower

energy consumption. Depending on the characteristics of the problem domain (i.e., boundary conditions) and also inherent constraints of each model, a suitable numerical model should be selected for simulation purpose. Regarding the sediment transport modeling during a flushing event, Peng and Niu (1987) and Ju (1990) were of the first, who used a 1D diffusion model for calculating the flushed out volume of sediment and bed profile change during a flushing process. For simplification purpose, a constant discharge and channel width was used during the flushing process. Kitamura (1995) and Chang et al. (1996) introduced the flushing channel width, as an initial geometrical boundary, that is developed in the wide study case reservoirs. More advanced 2D modelling of flushing process was conducted by Olsen (1999) in a physical model study and by Harb et al. (2012). 3-D models are still underdevelopment in this study area and recently just a few studies have been carried out by Haun and Olsen (2012a), Haun and Olsen (2012b) in the physical and prototype scale, respectively, and by Harb et al. (2014) in the prototype scale. 3-D simulation of sediment transportation process during a flushing process in a physical model is easier than in real prototype models because almost all parameters are known or idealized in physical models (e.g., uniform sediment size, known sediment discharge rate coming to the reservoir).

2.5.1 Advantage of 3-D CFD models compared to 1-D and 2-D models

When it is necessary to assess the morphological bed changes in rivers and reservoirs, application of numerical models other than 1D is essential because usually 1D models provide a profile and it is difficult to capture the morphological evolutionary process (Fukuoka et al., 2013). 2D depth-averaged numerical models can not directly simulate a complex 3-D flow field, including secondary flows in channel bends and transverse water level inclination, while they have strong contribution in natural sediment transportation processes. The complexity of 3-D flow patterns is more highlighted over the existing bed forms, on the deformed beds, during the free-flow state in a sediment flushing operation from a reservoir (Esmaeili et al., 2014a; Haun and Olsen, 2012a). Haun and Olsen (2012a) clearly showed the weakness of a 2D model in replicating the bed changes in a cross-section located at a channel bend because of the lack of ability to reproduce the secondary currents in channel bends. Artificial mathematical extensions should be included in 2D models to simulate the secondary currents in channel bends. Thus, application of 3-D numerical models are beneficial in complex reservoir geometries, when the velocity variation over the flow depth plays a major role (e.g., in channel bends). Recently, application of 3-D numerical models is feasible for simulating the sediment transport process owing to the increase of the calculation power of computers (e.g., Olsen & Kjellesvig, 1999; Ruether et al., 2005; Fischer-Antze et al., 2008; Dehghani et al., 2012; Esmaeili et al., 2013). Examples for the application of 3-D numerical models in reservoirs are the sediment transport simulation in Three Gorges project (Fang and Rodi, 2003), the simulation of flushing process in rectangular reservoirs (Esmaeili et al., 2014b), the Bodendorf reservoir (Haun et al., 2012), the Angostura reservoir (Haun and Olsen, 2012b), and also in Fischening reservoir (Harb et al., 2014).

2.5.2 3-D numerical modeling of a sediment flushing event in HPP Faching reservoir in Austria

HPP Faching is a small reservoir located in Alpine area of Austria with the approximate length of 4.5 km and the initial storage capacity of 1.4 million m³ in 1994. The storage volume is small compared to the annual inflow and sedimentation is a common problem in many Alpine reservoirs including Faching HPP reservoir. A large portion of suspended sediment is transported through the reservoir, but deposition of bed load fractions represents the main sedimentation problem (Harb et al., 2014). In these reservoirs the head area of the reservoir is usually covered by coarser materials and tail area by finer materials. Thus, a wide range of sediment size distribution exists in the reservoir area. Deposition of coarser materials in the head area of the reservoir results in the bed level aggradation and subsequently the water level raising that causes the flood protection problems (Harb et al., 2014 ; Haun et al., 2012). More specifically, gravel sized sediments are deposited in the head area while silt and clay are deposited in areas close to the weir. Figure 2.19 shows the aerial view of the reservoir with the sediment sampling points along the reservoir. In addition, Figure 2.20 illustrates the corresponding grain size distribution obtained from the sediment samples.



Figure 2.19 Plan view of the Faching HPP reservoir with the sediment sampling points (Harb et al., 2014).

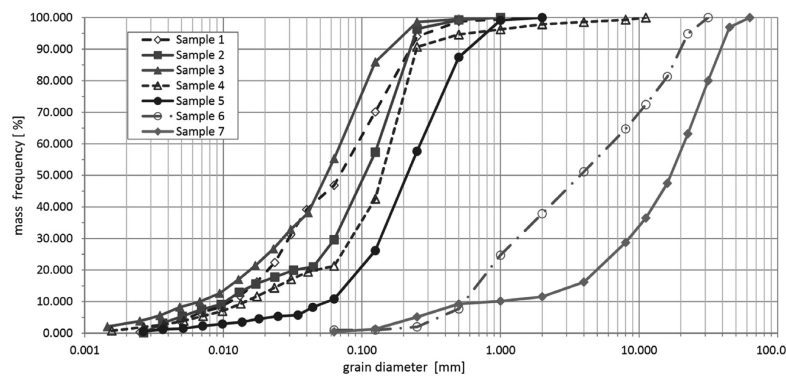


Figure 2.20 Grain size distribution of taken sediment samples from the reservoir (Harb et al., 2014).

Based on the conducted echo-soundings in 2007, it was found that approximately 0.89 million m³ of sediments have been already deposited in the reservoir. In the last years, the annual sedimentation

was approximately 85000 m³ that represents 6.1% annual sedimentation rate in this reservoir (Harb et al., 2014).

Harb et al (2014) used an open source 3-D numerical model, named Telemac-3-D, which solves the Navier-stokes equations in three dimensions for computing the turbulent flows. Detailed description of the model can be found in Hervouet (2007). Model was internally coupled with a morphological module SISYPHE to simulate the sediment transport process during the reservoir flushing operation in 2009. In the flushing operation, the water level was lowered about 1.6 m for 37 hrs. Furthermore, 8 sediment sizes with the spatial varying fractions were used according to the sediment distribution in the reservoir. k- ϵ turbulence model was used in their study and computed morphological bed changes were validated with the measurements taken by sonar in this reservoir. The hydrodynamic model verification was already performed utilizing the the measured surface velocity by ADCP in the prototype (Harb et al. 2013).

Their study revealed that it is a challenging task for the numerical model to use empirical sediment transport formulas and simulate the morphological bed changes accurately since the mentioned formulas were valid for the hydraulic domain for which they were developed. On the other hand, it was not possible to take into account the effect of complex developed bed forms on the bed roughness and subsequently on the sediment transport process. More specifically, there were standing waves induced by the antidune bed forms just upstream of the weir that could not be modelled properly and subsequently there was very low erosion in these areas. Figure 2.21 depicts the standing waves, which were observed during the flushing operation. However, it was found that both Meyer-Peter-Müller and Van Rijn sediment transport formula can provide the reasonable morphological bed changes. Application of Englund-Hansen total load formula could not provide reasonable morphological bed changes in their case may be because of its inability to reflect the different sediment transport behaviour of the bed load and suspended load fractions when three-dimensional effects exist.



Figure 2.21 Standing waves during the flushing operation of 2012 observed upstream of the weir (Harb et al., 2014).

2.5.3 3-D numerical modeling of a free-flow sediment flushing operation in Angostura reservoir, Costa Rica

Angostura reservoir is located on Reventazon River, about 50 km far from the San Jose in the central Costa Rica and its storage capacity is 17 million m³. The Reventazon River is located approximately 3000 m above the sea level and flow into the Caribbean ocean. The total catchment area of the river is 2953 km² with steep slopes and average annual precipitation of 3500 mm that can climb to 6000 mm locally (Janson & Rodriguez, 1992; Jimenez et al., 2004). Annual sediment yield is high in the river basin and it is about 2600 ton/km²/year. Thus, tributaries transport a noticeable amount of sediment into the river and subsequently into all the reservoirs along the river. As a consequence, the sediment flushing is necessary to decrease the storage loss in these reservoirs. Hydro power generation from Reventazon River provides about 25% of the total used electricity in Costa Rica. The catchment area of the reservoir itself is about 1463 km² and the annual sediment entrainment into the reservoir is about 3.5 million ton. The rockfill hydropower dam has been built in 2000 with sediment flushing facilities and produces 177 million watt electricity (Jimenez et al., 2004). Since 2006, free-flow sediment flushing operation is conducted in November, which is followed by a partial drawdown flushing together with upstream Cachi reservoir in September. Through this type of sediment management strategy the annual sedimentation rate decreased to 2.5% and during the 10 years operation period only 4 million m³ of the initial storage volume was lost (Haun and Olsen, 2012b). Figure 2.21 shows the Angostura reservoir with the location of dam and upstream inflow boundaries.



Figure 2.22 Downstream view from Angostura reservoir, Costa Rica in November 2010 (Haun & Olsen, 2012b).

Haun & Olsen (2012b) used a 3-D numerical model, named SSIM, for simulating the free-flow flushing operation of Angostura reservoir in November 2010. SSIM model solves Navier-stokes equations in three dimensions for computing the turbulent flows and uses finite volume approach as discretization method. In the morphological module, the suspended and bed load sediment transport process can be modeled using Van Rijn (1984a and 1984b) formulas. Bedforms are also taken into account using

another empirical formula introduced by Van Rijn (1984c). During the simulation, only the water body was modeled through the application of a wetting/drying algorithm and cells with a shallow water head were removed from the computational domain. The effect of wetting/drying algorithm during the simulation of flushing operation has been illustrated in Figure 2.23. Model also used an implicit and iterative algorithm for computing the free water surface which enables the model to utilize large time steps. The sand slide algorithm that is used in the model was generally developed for non-cohesive materials and bank failure algorithms were not considered completely. Detailed description of the model can be found in Olsen (2014). The Discharge and water level rates that were recorded at the weir during the free-flow flushing operation were also used as the hydrodynamic boundary conditions. Moreover, non-uniform distribution of the bed materials with 4 sediment sizes was used according to the available bed samples. The sediment inflow into the reservoir was also introduced to the model using the rating curves developed from measurements at a gauging station in Reventazon River. The standard k- ϵ turbulence model was utilized in their study and computed morphological bed changes were verified with the measurements taken by sonar after the flushing in this reservoir.

Their numerical study showed that in the wide Angostura reservoir, with a large ratio of width to length, a distinctive flushing channel is formed similar to the observations along the thalweg of the Reventazon River. The water level over the tail area of the reservoir was slightly higher compared to the observed one that results in a lower bed shear stress and consequently lower erosion in this area. In addition, moving waves towards upstream were observed in the real operation that implies the development of bedforms in the form of dunes and antidunes especially over the tail area of the reservoir. Since bedforms were taken into account through an empirical formula that was originally developed for small Froude numbers, they concluded that it may possible to have difference in the simulated bedforms characteristics in the mentioned area. They reported a notable difference between the simulated and measured bed levels in the tail area of the reservoir, where bedforms emerged. Uncertainties in prediction of bed levels were also suggested to increase when a combined effect of bedforms and secondary currents exist. Haun & Olsen (2012b) also suggested that a second order upwind scheme (SOU) provides a higher quality outputs compared to the power law scheme (POW) since it can handle a more complex flow field through a better prediction of secondary current effects in the computation. The total volume of flushed out sediment volume was equal to 259500 m³ based on the measurements while simulation results reached to 258500 m³.

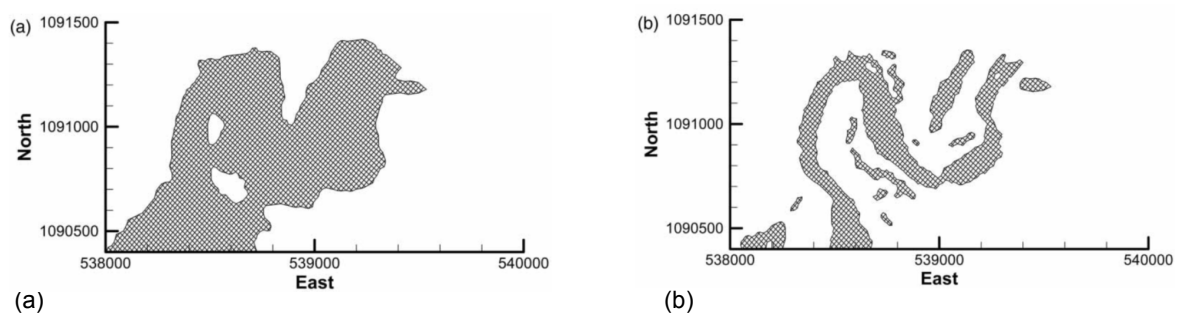


Figure 2.23 Computational grid in the downstream part of the reservoir (a) at the beginning of flushing simulation ; (b) during the free-flow condition.

2.5.4 3-D numerical modeling of a free-flow sediment flushing operation in Bodendorf reservoir, Austria

Bodendorf reservoir is located in Styria province, across the Mur river in Alpine area of Austria. It was designed as a run-of-river hydropower plant with the capacity of 7.5 million m³ and a drainage area of about 1360 km². The reservoir shape is narrow since the reservoir length is almost 2.5 km and the average reservoir width is about 40 m, and its average depth is also about 3 m (Knablauch et al., 2005 ; Schneider et al., 2006). The effective storage capacity is equal to 0.9 million m³ in 1982, but about 0.6 million m³ of the reservoir was filled with sediments 12 years after starting the operation that implies the 4% annual sedimentation rate (i.e., 35000 m³/year). Also the measure volume of the flushed out sediment during 2004 flushing operation proved that a significant amount of the flushed out sediment (i.e., about 70%) was transported as the bed load (Badura et al., 2006). Likewise Fischening HPP reservoir, the grain size distribution showed a large variation from the head area of the reservoir to the tail area. While the median sediment size was 32 mm in the head area of the reservoir, it decreased to 1.5 mm in area close to the weir (Badura, 2007). Sediment deposition in this reservoir not only decreased the reservoir storage capacity, but also causes the bed aggradation in the head area that increases the water level and consequently the flooding risk. Sediment deposition in this reservoir also causes the bed degradation and coarser sediment shortage in downstream area, which has a negative impact on the downstream ecology (e.g., lack of suitable habitat structure for aquatic animals). An aerial photo from Bodendorf reservoir along with another photo from the area close to the weir of Bodendorf reservoir, during a flushing operation has been illustrated in Figure 2.24.



(a)



(b)

Figure 2.24 (a) An aerial photo from Bodendorf reservoir (Badura et al., 2007); (b) A photo from Bodendorf hydropower plant during a flushing operation (Schneider et al., 2006).

Haun et al. (2012) used 3-D numerical model SSIIM, as already described in part 2.5.3, and simulated the free-flow flushing operation of 2004 in this reservoir. The flushing operation in 2004 last for 31 hours with 17 hours of free-flow condition and total volume of 47300 m³ was flushed out through the reservoir. The bathymetry data before the flushing operation, spatial varying sediment distribution, and the

hydrodynamic boundary condition (i.e., discharge rates and water levels) of 2004 free-flow flushing operation were used to set up the model.

In the conducted study by Haun et al. (2012), both Van Rijn (1984b) and Meyer-Peter-Müller (1948) bed load sediment transport formula were tested and, then, computed morphological bed changes were compared with the measured one. It was concluded that both Van Rijn and Meyer-Peter-Müller bed load transport formula can provide an acceptable prediction of the total volume of the flushed out sediment. The computed total volume of the flushed out sediment was equal to 23400 m³ when Van Rijn's formula was used while it was equal to 31200 when Meyer-Peter-Müller's formula was utilized. Moreover, application of Meyer-Peter-Müller formula showed a better prediction of morphological bed changes compared to application of Van Rijn formula since Meyer-Peter-Müller formula is recommended for rivers with steeper slopes that carries the coarser sediments mainly as the bed load. However, both formula underestimated the sediment transport capacity and subsequently an irregular bed erosion pattern with a low erosion magnitude emerged in downstream areas of the reservoir (i.e., close to the weir). Besides, eroded coarse materials redeposited nearby the weir that caused the bed elevations to raise. The secondary current effects were also overestimated, which resulted in a higher erosion in the outer part of the channel bend and also a higher deposition in the inner part. For this case also the observed bedforms such as dunes and antidunes influenced the sediment transportation, which can not be measured during a flushing operation. The influences of these bedforms just are modeled by an empirical formula that is mainly developed for low Froude numbers. However, the model can properly predict the total volume of the flushed out sediment that enables the authorities to have a preliminary assessment of the upcoming flushing event.

Chapter 3 Theoretical background of 3-D CFD model SSIIM

SSIIM program has been initially developed by N.R.B Olsen in 1990-1991 at Norwegian University of Science and Technology (NTNU) and is still under development. SSIIM is an abbreviation for Sediment Simulation In Intakes with Multiblock option. The program has been made for research purpose by academic communities and the initial motivation was to simulate the sediment transportation in rivers and channels especially when it was difficult to conduct the physical model study. Later, the application of the program was further developed to various hydraulic engineering topics. However, the main application domain is sediment transport in rivers, reservoirs and around hydraulic structures (Olsen, 2014).

The main advantage of SSIIM compared to other CFD programs is simulation of sediment transport with movable bed in complex geometries. Moreover, model is able to take into account the multiple sediment sizes transportation, sediment sorting process, bed load and suspended load transportation, and bed slope. Model also computes the time dependent bed changes on almost non-orthogonal grid that moves with changes in water and bed levels. The recently added wetting/drying modules, that works with unstructured grids, enables the model to deal with more complex geometries and morphological bed changes (e.g., lateral movement of channel), and also more complex hydraulic boundary conditions (e.g., abrupt water level drawdown) (Olsen, 2014).

In this chapter, the governing theories and equations for computing the hydrodynamics and sediment transport in SSIIM program will be described. More details about technical instructions for applying the corresponding algorithms to the governing theories can be found in (Olsen, 2014). Herein, descriptions mainly focuses on the theories and equations utilized in SSIIM program according to the objectives of this study.

3.1 3-D numerical modeling of flow field in SSIIM

In SSIIM program, the Reynolds-averaged Navier-Stokes equations are solved using different turbulence closure schemes on a general three dimensional grid to compute the water motions for turbulent flows. An implicit solver is used and computed velocities are then utilized for solving the convection-diffusion equation for different sediment sizes (Olsen, 2014). The mentioned equations and corresponding descriptions are presented in the following sections.

3.1.1 Governing equations for flow field modeling

Continuity equation together with the Reynolds-averaged Navier-Stokes equations are solved to determine the turbulent flow field for a 3-D geometry as follows (Versteeg & Malalasekera, 2007):

$$\frac{\partial U_i}{\partial x_i} = 0 \quad \text{with } i=1,2,3 \quad (1)$$

$$\frac{\partial U_i}{\partial t} + U_j \frac{\partial U_i}{\partial x_j} = \frac{1}{\rho_w} \frac{\partial}{\partial x_j} \left(-P \delta_{ij} - \rho_w \overline{u_i u_j} \right) \quad (2)$$

where $i=1, 2, 3$ is the representative of three directions; where U_j is the averaged flow velocity, x_i is the spatial geometrical scale, ρ_w is the water density, P is the Reynolds-averaged pressure, δ_{ij} is the Kronecker delta, and $-\rho_w \overline{u_i u_j}$ is the Reynolds stress term. u is the velocity fluctuations over the time in one time step Δt .

The first and second terms on the left hand side of Reynolds-averaged Navier-Stokes equations are transient and convective terms, respectively. On the right hand side, the first and second terms are also the pressure and Reynolds stress terms (Volume and surface forces). The finite-volume approach is applied as a spatial discretisation method to transform the partial equations into algebraic equations. The convection term in the Navier-Stokes equation can be solved using second-order upwind scheme. However, it is also possible to use other schemes. More detailed explanations are provided in the corresponding parts of this section.

3.1.2 Discretization schemes

There are different spatial discretization methods such as finite difference, finite element, and finite volume method to transform the partial differential equations into algebraic equations where the variable in one cell would be the function of variables in neighbour cells. The finite difference method is utilized directly on the differential equations using the Taylor series expansions. Although higher order derivatives can be computed easily using this approach, it can be just utilized on structured grids and its application to curvilinear grids is not straight-forward (Blazek, 2001). Consequently, simple grids should be used and it would not be a suitable option for complex geometries. In finite element method triangular or tetrahedral elements are employed that makes this method applicable on unstructured grids (i.e., grids with different number of cells in longitudinal and lateral direction). Thus, this method is more suitable for complex geometries. However, much more numerical computations should be performed compared to the finite volume method (Blazek, 2001).

The finite volume method is applied on the integral form of the equations over the control volume. The surface integral then would be approximated as sum of the fluxes crossing the individual faces of the control volume. The finite volume method has two important advantages. First, the spatial discretization is done over the physical space, which means transformation between different coordinate systems is not required. Therefore, finite volume method can be flexibly used for both structured and unstructured grids. Second, the method is conservative since the integral form of the equations is used and the fluxes entering a given volume are identical to that leaving the adjacent volume. Due to the advantages of this method, it is used in the SSIIM program.

3.1.3 Turbulence modeling shemes

An appropriate turbulence model should have a minimum complexity and can also capture the essence of the relevant physics (Wilcox, 1995). As the default turbulence closure scheme in SSIIM program, the standard k- ε turbulence model with constant empirical values is used for modelling the Reynolds stress term as shown in Eq. (3) and (4) (Rodi, 1980). This is due to the strength and wide applicability of k- ε turbulence model, which makes it suitable for general use in CFD programs (Versteeg and Malalasekera, 2007).

$$-\overline{u_i u_j} = \nu_t \left(\frac{\partial U_i}{\partial x_j} + \frac{\partial U_j}{\partial x_i} \right) - \frac{2}{3} k \delta_{ij} \quad (3)$$

$$\nu_t = \frac{c_\mu k^2}{\varepsilon} \quad (4)$$

where ν_t is the turbulent eddy-viscosity, k is the turbulent kinetic energy, and ε is the dissipation rate of k . The equations for k and ε are as follows:

$$\frac{Dk}{Dt} = \frac{\partial k}{\partial t} + U_j \frac{\partial k}{\partial x_j} = \frac{\partial}{\partial x_j} \left[\left(\nu + \frac{\nu_t}{\sigma_k} \right) \frac{\partial k}{\partial x_j} \right] + P_k - \varepsilon \quad (5)$$

$$\frac{D\varepsilon}{Dt} = \frac{\partial \varepsilon}{\partial t} + U_j \frac{\partial \varepsilon}{\partial x_j} = \frac{\partial}{\partial x_j} \left[\left(\nu + \frac{\nu_t}{\sigma_\varepsilon} \right) \frac{\partial \varepsilon}{\partial x_j} \right] + c_{1\varepsilon} \frac{\varepsilon}{k} P_k - c_{2\varepsilon} \frac{\varepsilon^2}{k} \quad (6)$$

where c_μ , $c_{1\varepsilon}$, $c_{2\varepsilon}$, σ_k and σ_ε are the constant empirical values by (Lauder & Spalding, 1972):

$c_\mu=0.09$, $c_{1\varepsilon}=1.44$, $c_{2\varepsilon}=1.92$, $\sigma_k=1.00$ and $\sigma_\varepsilon=1.30$.

Here, P_k denotes the production of kinetic energy as is calculated as follows:

$$P_k = \nu_t \frac{\partial U_i}{\partial x_j} \left(\frac{\partial U_j}{\partial x_i} + \frac{\partial U_i}{\partial x_j} \right) \quad (7)$$

The turbulent eddy viscosity is the property of the flow field not the fluid and consequently varies in space and time. In all equations above, the turbulent eddy viscosity is assumed as a scalar quantity while it is in contrast to anisotropic nature of turbulence. However, there are some turbulence models that use various eddy viscosities in different directions (Rodi, 1984). In SSIIM program, also k- ω turbulence model can be utilized. Both can predict the properties of the turbulent flow without prior information about the turbulent structure. This is possible due to the prediction of both kinetic energy and turbulence length scale (Wilcox, 1995). Depends on the version of the SSIIM program, other turbulence models can also be used (Olsen, 2014).

3.1.4 Computation of pressure field and free-water surface

The free surface is computed using a fixed lid approach and location of this fixed lid and its movement is a function of time step and water flow field (Olsen, 2014). In SSIIM program the SIMPLE method introduced by Patankar (1980) is used by the 3-D solver for computing the unknown pressure field in the Reynolds-averaged-Navier Stokes equation. Then, pressure difference between a reference point, which is usually located at the downstream boundary, and any other surface cell is related to the elevation difference as follows (Rüther & Olsen, 2007):

$$\frac{\partial z}{\partial x} = \frac{1}{\rho g} \frac{\partial p}{\partial x} \quad (8)$$

where ∂z is elevation difference, ∂p is the pressure difference, ρ is the fluid density and g is the gravity acceleration. Thus, it is assumed that the water level is located at the pressure line with a lid that is fixed in each time step. This assumption will be correct as long as the energy loss can be calculated correctly by the CFD program. This implicit and iterative approach is a robust and stable method that can be also used in connection with the sediment transport and morphological bed changes under unsteady flow condition. As long as large number of inner iterations is set for each time step in 3-D Navier-stokes solver, for stability and convergence issues, large time steps can also be employed. This method was successfully used for number of cases with large time steps and unsteady flow condition such as Danube River (Fischer-Antze et al., 2008). This method was also used for 3-D computation of flood process in natural complex geometries (Tritthart & Gutknecht, 2007).

However, assuming the location of water level on the pressure line with a lid that is fixed in each time step is not applicable in some cases such as high Froude number flows. When the Froude number is around one, the algorithm can become unstable. In addition, the energy loss may not be computed correctly in a relatively coarse grid, which results in incorrect water elevation differences between the reference point and the other point. Thus, a new algorithm was developed by Haun & Olsen (2012b) that uses the local elevation difference between a cell and neighbor cells instead of computing the elevation difference between a cell and the reference cell. In the new algorithm, the elevation difference is also computed using the Eq (8). An iterative method would be also used since the water level is unknown in all cells. In the other word, an implicit free-water surface algorithm is used in the computations, based on the pressure gradient between the cell and the neighbour cell (Olsen and Haun 2010).

Because a number of neighbor cells would be used for computing the water elevation differences for each cell, different values will appear for water elevation difference depending on the number of neighbour cells used for computation. Therefore, a weighted average of these values was employed in the new algorithm. The weighting coefficient (i.e., a_i) for each neighbor cell is the function of Froude number, the flow direction and location of the neighbor cell (Eq. (9)). This coefficient is then used for descritizing the Eq. (11) (Haun & Olsen, 2012b).

$$a_i = \begin{cases} \min(2 - Fr, 1) & \text{for } w > -0.1 \text{ and } Fr < 2.0 \\ w^2 (Fr - 1) & \text{for } w < -0.5 \text{ and } Fr > 2.0 \\ 0.0. & \end{cases} \quad (9)$$

with:

$$w = \frac{\vec{r} \times \vec{u}}{|\vec{r}| \times |\vec{u}|} \quad (10)$$

where a_i is the weighting coefficient for the neighbor cell, Fr is the Froude number, w is the dot product of $\vec{r}$ and $\vec{u}$, where $\vec{r}$ is the direction vector pointing from the center of a cell to the centre of the neighbour cell aimed to take into account the upstream/downstream effect and $\vec{u}$ is the velocity vector of the cell.

$$\sum_{i=1}^8 a_i z_p = \sum_{i=1}^8 a_i \left(z_i + \frac{1}{\rho g} (p_p - p_i) \right) \quad (11)$$

where z_p is the water level elevation in the cell, z_i is the water level elevation in the i th neighbour cell, p_p is the pressure in the cell and p_i is the pressure in the i th neighbour cell.

The new free-water surface algorithm was successfully used for simulation of free-flow sediment flushing in prototype reservoirs such as Angostura reservoir in Costa Rica (Haun and Olsen, 2012b) Bodendorf reservoir in Austria (Haun et al., 2012). It has been also tested for physical model of Kaliga Gandaki reservoir in Nepal (Haun and Olsen, 2012a).

3.1.5 Types of used boundary conditions

In SSIIM program, the Dirichlet boundary condition for the water inflow (i.e., logarithmic velocity distribution) was used, whereas the zero-gradient boundary condition was specified for the water outflow and the sediment concentration calculation.

In zero gradient boundary condition, the derivative of variables would be zero. It means the value of the variable at the boundary would be equal to the value in the closest cell to the boundary. This is an straightforward approach in iterative solution procedure so that in each iteration the boundary value would be set equal to the value in the closest cell. Dirichlet boundary condition means the value of variables should be specified at the boundary. For instance, the zero sediment concentration is usually given at the water surface and also the sediment concentration at the upstream boundary is specified.

For rough boundaries of side walls and the bed, where there is no water flux, the empirical wall laws introduced by Schlichting (1979) are utilized (Eq. (12)) while for smooth boundaries, Eq. (13) and Eq. (14) are used. This empirical wall laws are used because the velocity gradient towards walls and bed is steep and many grid cells will be required. This means that the velocity profile in areas close to the bed and side walls follows an empirical function (Olsen, 2014).

$$\frac{U}{u^*} = \frac{1}{\kappa} \ln \left(\frac{30y}{k_s} \right) \quad (12)$$

where the shear velocity is denoted as u^* , κ is the von Karman constant which is equal to 0.4, y is the water depth (distance from the boundary), and k_s is the equivalent roughness.

$$\frac{U}{u^*} = \frac{1}{\kappa} \ln \left(\frac{Eyu^*}{\nu} \right) \quad \frac{yu^*}{\nu} > 11 \quad (13)$$

$$\frac{U}{u^*} = \left(\frac{Eyu^*}{\nu} \right) \quad \frac{yu^*}{\nu} < 11 \quad (14)$$

where E is an empirical value equals to 9.0.

Wall-laws are also used to transform the discretized source terms into the analytical expressions for predicting the velocity field in Reynolds-averaged Navier-Stokes equations and predicting the turbulence parameters as well (Olsen, 2014).

3.1.6 Computational grid and effect of wetting/drying algorithms

SSIIM program solves the continuity equation together with Reynolds-averaged Navier-Stokes equations over 3-D almost general-nonorthogonal grid. Depending on the version of the software, structured or non-structured grid can be utilized. While SSIIM 1 uses the structured grid, SSIIM 2 uses the unstructured grid. In the structured grid each cell has three indexes (i.e., i, j, k in three cardinal dimensions) while in unstructured one each cell has one index. The computational speed would be faster in structured grid since faster solvers are available and less memory is used per cell since the connection between cells, surface and geometry points are simpler. The main advantage of the unstructured grid is feasibility of modeling the complex bed geometries when wetting/drying condition occurs in the computational domain (Olsen, 2014). Thus, one can expect to have a more accurate simulation of time dependent morphological bed changes when an iterative algorithm is used. In both versions of the SSIM model, an adaptive mesh is used, which moves with the changes in the bed and water level in each time step during the computation.

In SSIIM 2, both tetrahedral and hexahedral cells are used and the water surface is recomputed after each time step. Then, a new grid is generated based on the magnitude of the recomputed water depth using a wetting/drying algorithm (Olsen, 2003). Thus, just the water body is simulated (Olsen & Haun, 2010). Using this approach makes it possible to have a dynamic grid that can move in the lateral direction similar to the procedure takes place in free-form meandering channel evolution, or in sediment flushing from reservoirs (Olsen, 2014). In other word, through the application of wetting/drying algorithm for simulating the water body, a varying number of grid cells in different spatial directions is possible during the computation process. A wetting/drying procedure that takes place during a flushing operation and how it is considered in SSIIM 2 are described in the following paragraph (Olsen, 2003):

Due to the water level draw down or the bed level raise during a flushing operation, the number of cells in vertical direction would be decreased and some areas may be dried up. In areas where drying condition occurs, cells that covers these areas will be eliminated from the computational domain to prevent emerging the distorted cells (i.e., cells with very small vertical dimension compared to the horizontal dimension). In contrast, when the water level increases during the refilling stage, the wetting condition occurs in dried up areas and new cells will be regenerated to cover again these areas. For implementing the wetting/drying condition in SSIIM 2, the lower and upper boundary of water depth (i.e., z_1 and z_2) should be introduced to the model. In areas where the water depth is smaller than z_1 (i.e., $y < z_1$), the drying condition will occur and cells in this area will be disappeared. In areas where the water depth is between the lower and the upper boundary (i.e., $z_1 < y < z_2$), just one cell will be generated and a 2-D computation of flow will be implemented while for areas with water depth bigger than the upper boundary (i.e., $y > z_2$), Eq. (15) is applied to calculate the number of cells in the vertical direction, and a 3-D computation will be performed.

$$n = n_{\max} \left(\frac{\text{depth}}{\text{depth}_{\max}} \right)^{po} \quad (15)$$

where n is the number of grid cells in the vertical direction, $n_{\max}$ is the maximum number of grid cells in the vertical direction and po is a parameter for the number of grid cells and is defined by the user.

3.2 3-D numerical modeling of morphological processes in SSIIM

Morphological bed changes in aluvial streams and reservoirs are the result of the sediment load transport process. Bed load and suspended load transport are two general types of the sediment load transport. The sum of the bed load and suspended load transport represents the total sediment load transport. Bed load is the portion of the total load, which is transported along the bed by sliding, rolling, or saltating (Van Rijn, 1984b). However, it is difficult to distinguish a specific borderline between the bed load and suspended load transport. One quantitative criterion that can divide the total sediment load transport to bedload and suspended load is the Rouse number, which is developed by Rouse (1937). The number is used for calculating the vertical distribution of the suspended load transport. If it is larger than the unity, bed load transport occurs and if it is smaller than the unity, then the sediment transport would be mainly as suspended load.

Thus, the sediment transport computation for simulating the morphological bed changes in SSIIM program is divided into the suspended and bed load transport. In this section, the governing equations for simulating the morphological bed changes process are presented and discussed.

3.2.1 Computation of suspended sediment transport

Suspended load transport is calculated by solving the transient convection-diffusion equation formula (Olsen, 2014) as shown in Eq. (16). For simulating the non-uniform sediment sizes in alluvial streams and reservoirs, the convection-diffusion equation is solved separately for each sediment sizes.

$$\frac{\partial c}{\partial t} + U_j \frac{\partial c}{\partial x_j} + w \frac{\partial c}{\partial z} = \frac{\partial}{\partial x_j} (\Gamma_T \frac{\partial c}{\partial x_j}) \quad (16)$$

Where U_j is the water velocity, w is the fall velocity of sediments, c is the sediment concentration over time t and spatial geometries (i.e., x and z), and Γ_T is the turbulent diffusivity picked from k - ϵ model and can be expressed by Eq. (17):

$$\Gamma_T = \frac{\nu_T}{Sc} \quad (17)$$

where Sc is the Schmidt number representing the ratio of eddy viscosity coefficient ν_T to diffusion coefficient and set to 1.0 as default.

Generally, there are two main transport processes for the suspended sediment transport, namely, the convection and the turbulent diffusion of steady sediment flow. The convection of sediment is a type of transport because of the average water velocity. Sediment transport because of the fall velocity of sediments is a type of convective sediment transport. When the sediment flux should be calculated through an area, Eq. (18) is employed. The turbulent diffusion of sediment is due to the turbulence mixing and concentration gradients. The turbulent diffusion can be modelled with a turbulent mixing coefficient (Eq. 19) (Olsen, 1999b).

$$F = cUA \quad (18)$$

where c is the average sediment concentration over the area, U is the average sediment velocity normal to the surface, and A is the given surface area.

$$\Gamma = \left(\frac{F}{\frac{A}{dc}} \right) \quad (19)$$

where, the sediment flux over the area A is divided by the concentration gradient.

The convective sediment transport is the more dominant suspended sediment transport process, which is mainly affected by the water flow and sediment fall velocity. However, in some cases such as sand traps or reservoirs, the diffusive sediment transport which is mainly affected by the turbulence mixing and the concentration gradient would be important.

In order to compute the equilibrium suspended sediment concentration in the cells close to the bed, Eq. 20 is used as the boundary condition (Van Rijn, 1984b). This sediment concentration is converted to the sediment entrainment rate later to perform the time-dependent computations.

$$C_{bed} = 0.015 \frac{d_i^{0.3}}{a} \frac{\left[\frac{\tau - \tau_{c,i}}{\tau_{c,i}} \right]^{1.5}}{\left[\frac{(\rho_s - \rho_w)g}{\rho_w \nu^2} \right]^{0.1}} \quad (20)$$

where a is the reference level set equal to the roughness height, d_i is the diameter of the i th fraction, τ is the shear stress, $\tau_{c,i}$ is the critical shear stress for d_i which was calculated from the Shield's curve, ρ_s is the density of sediment, ρ_w is the density of the water, g is the gravity acceleration and ν is the kinematic viscosity. The empirical parameters are also can be modified in SSIIM program.

3.2.2 Computation of bed-load sediment transport

In SSIIM program, bed load sediment transport is modelled using empirical formulas. These formulas are developed in experimental flumes under a certain condition of flow and bed sediment characteristics, which means each formula may have some limitations for application. Thus, all empirical formulas are not able to provide appropriate performance for each case. Bed load can be simulated by Van Rijn formula (Van Rijn, 1984a) or alternatively by Meyer-Peter-Müller formula that is called MPM hereafter (Meyer-Peter and Müller, 1948). The latter one is applicable just in SSIIM 2.

Van Rijn formula (Eq. 21) has been developed for sand particles and used to simulate a wide variety of sediment transport issues in both physical model and prototype scale whereas MPM formula (Eq.22) is more appropriate for steep rivers that mainly transport the coarse sediments close to the bed :

$$\frac{q_{b,i}}{d_i^{1.5} \sqrt{\frac{(\rho_s - \rho_w)g}{\rho_w}}} = 0.053 \frac{\left[\frac{\tau - \tau_{c,i}}{\tau_{c,i}} \right]^{1.5}}{d_i^{0.3} \left[\left(\frac{(\rho_s - \rho_w)g}{\rho_w \nu^2} \right) \right]^{0.1}} \quad (21)$$

where $q_{b,i}$ is sediment transportation rate for the i th fraction of bed load per unit width, ρ_s is the density of sediment, and ρ_w is the density of water. Other parameters were already defined in Eq. (20).

$$q_{b,i} = \frac{1}{g} \left[\frac{\rho_w g r I - 0.047 g (\rho_s - \rho_w) d_{50}}{0.25 \rho_w^{\frac{1}{3}} \left(\frac{\rho_s - \rho_w}{\rho_s} \right)^{\frac{2}{3}}} \right] \quad (22)$$

where d_{50} is the characteristic sediment size (median sediment size), r is the hydraulic radius and I is the slope of the energy line. Other parameters in the formula were already defined.

3.2.3 Bed forms modeling

In rivers and reservoirs, bed forms are usually developed and result in a significant effect on the bed roughness. While bed forms such as ripples and dunes are developed in rivers and reservoirs under a subcritical flow condition, antidunes appear when supercritical flows occurs during a free-flow flushing operation.

In SSIIM program, bed roughness in the form of dunes and ripples are taken into account using the sediment size distribution and the bed form height (Eq. (23)) (Olsen, 2014 ; R  ther & Olsen, 2007).

$$k_s = 3.0d_{90} + 1.1\Delta \left(1.0 - e^{\left(\frac{-25\Delta}{7.3y} \right)} \right) \quad (23)$$

where d_{90} is the characteristic sediment size, y is the water depth, and Δ is the bed form hight that is calculated as follows (Van Rijn, 1984c) :

$$\frac{\Delta}{y} = 0.11 \left(\frac{d_{50}}{y} \right)^{0.3} \left(1 - e^{-0.5 \left(\frac{\tau - \tau_{c,i}}{\tau_{c,i}} \right)} \right) \left(25 - \left(\frac{\tau - \tau_{c,i}}{\tau_{c,i}} \right) \right) \quad (24)$$

where d_{50} is the meadian sediment size, τ is the shear stress, and $\tau_{c,i}$ is the critical shear stress for the i the fraction.

The suggested formula by Van Rijn (1984c) to compute the bed form roughness is mainly suitable for fine and uniform size sediments under the small Froude number flows. These limitations would be a disadvantage for simulating the flushing process. Because when water level is completely lowered and free flow stage is developed over the reservoir, bed forms can play a major role in sediment transportation pattern (i.e., erosion and deposition). However, it is necessary to keep in mind that formulas for calculating the bed form hight and lenght have been rarely developed.

3.2.4 Numerical discretization schemes used for computations

For transforming the partial differential eqauations into an algebraic equation as a function of the variables in the neighbour cells, different numerical schemes are used. Amongst different schemes, both the first order Power Law scheme (POW) and Second-Order Upwind schemes (SOU) are widely used with the control volume as the numerical discretization method. Here, it is assumed that the unknown variable that we try to compute is the sediment concentration since fluxes through the cell surfaces are easier to imagine. The new function can be a weighted average of the concentration in neighbour cells and can be calculated using the following schemes (Olsen, 1999b) :

First Order Upwind (FOU) scheme

If the sediment concentrartion in cell p is considered in 2-D plan view (i.e., c_p), there were four concentrartion values for neighbour cells in north, east, south, and west direction, namely, c_n , c_e , c_s and

c_w respectively (Figure 3.1). In 3-D mesh, there are also two additional neighbour cells in top and the bottom direction. Then, corresponding weighting factors would be a_n , a_e , a_s and a_w . Consequently, the weighted average for sediment concentration in cell p would be:

$$c_p = \frac{a_w c_w + a_e c_e + a_n c_n + a_s c_s}{a_p} \quad (25)$$

where $a_p = a_w + a_e + a_n + a_s$

The weighting factors for the neighbour cells are also denoted a_{nb} and different discretization methods (i.e., POW and SOU) are used for calculating the a_{nb} (Olsen, 1999b).

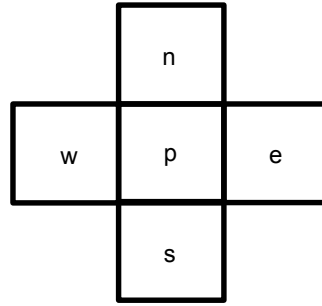


Figure 3.1 Calculation cells in the first order discretization schemes.

In the control volume method, continuity of the sediment is employed and the basis of the calculation is the fluxes on a cell surface. As already mentioned there are two types of fluxes :

- 1- convective flux that equals to $U \cdot A \cdot c$
- 2- diffusive flux that equals to $\Gamma \cdot A \cdot dc/dx$

where A , U , c and Γ denote the surface area, the velocity at the surface normal to it, the concentration at the surface, and turbulence diffusion at the surface, respectively. dc/dx also represents the concentration difference between cells on each side of the surface divided by the distance between the centers of the cells. The following figure explains the variables in a calculation molecule:

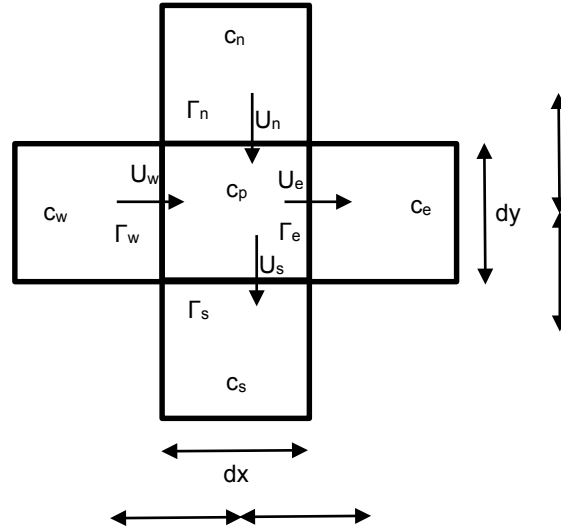


Figure 3.2 Calculation molecule in the first order discretization schemes with utilized variables for the computation.

Thus, the total flux through the west side of cell p can be calculated as follows :

$$F_w = U_w A_w c_w + \Gamma_w A_w (c_w - c_p) / dx \quad (26)$$

where A_w is the area of the side wall on the west side, dy times the height of the side wall.

Similiarly, the following fluxes are computed for the other sides :

$$F_e = U_e A_e c_e + \Gamma_e A_e (c_p - c_e) / dx \quad (27)$$

$$F_n = U_n A_n c_n + \Gamma_n A_n (c_n - c_p) / dy \quad (28)$$

$$F_s = U_s A_s c_s + \Gamma_s A_s (c_p - c_s) / dy \quad (29)$$

Sum of the fluxes should be zero as : $F_w - F_e + F_n - F_s = 0$. Comparing the summation result with Eq. (25), provides the weighting factors as follows :

$$a_e = \Gamma_e A_e / dx \quad (30)$$

$$a_w = U_w A_w + \Gamma_w A_w / dx \quad (31)$$

$$a_s = \Gamma_s A_s / dy \quad (32)$$

$$a_n = U_n A_n + \Gamma_n A_n / dy \quad (33)$$

$$a_p = \Gamma_e A_e / dx + U_e A_e + \Gamma_w A_w / dx + \Gamma_n A_n / dy + U_s A_s + \Gamma_s A_s / dy \quad (34)$$

The first order upwind schemes may include some inaccuracies in steep gradients and gives large false diffusions, although it can give stable simulation in some cases. To keep some stability and also decrease the false diffuions at the same time, a second order upwind scheme should be used.

POW discretization scheme

This discretization method is a first-order scheme and is used when the convective fluxes are dominating. In this scheme, turbulent diffusive terms are reduced using a factor f , which is assumed to have a value between 0 and 1. This discretization scheme, can be used instead of other first-order schemes to prevent negative weighting coefficients and false diffusions. The reduction is performed using the following power function (Olsen, 1999b):

$$f = \max\left[0, (1 - 0.1|Pe|)^5\right] \quad (35)$$

with the pechlet number (i.e., Pe) equals to :

$$Pe = \rho UL_c / \Gamma \quad (36)$$

where L is the cell length.

In SSIIM 1 the reduction of turbulent diffusive terms is performed in both directions but in SSIIM 2 the reduction would be just in vertical direction (Olsen, 2014).

SOU discretization scheme

In SOU scheme, a larger calculation molecule is considered as follows :

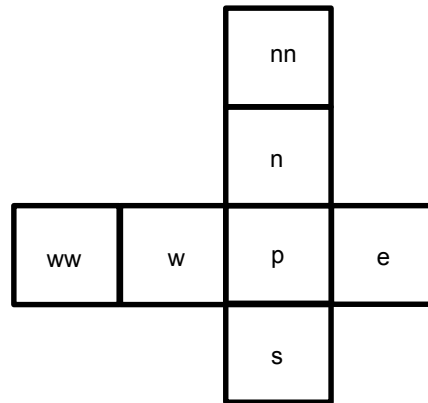


Figure 3.3 Calculation cells arrangement in the second order upwind (SOU) scheme.

Subsequently, weighted average of sediment concentration in cell p would be as follows :

$$c_p = \frac{a_w c_w + a_e c_e + a_n c_n + a_s c_s + a_{ww} c_{ww} + a_{nn} c_{nn}}{a_p} \quad (37)$$

where $a_p = a_w + a_e + a_n + a_s + a_{ww} + a_{nn}$. Therefore, flux through the west side of cell p can be calculated as :

$$F_w = U_w A_w \left(\frac{3}{2} c_w - \frac{1}{2} c_{ww} \right) + \Gamma_w A_w (c_w - c_p) / dx \quad (38)$$

The following fluxes are also computed for the other sides :

$$F_e = U_e A_e \left(\frac{3}{2} c_p - \frac{1}{2} c_w \right) + \Gamma_e A_e (c_p - c_e) / dx \quad (39)$$

$$F_n = U_n A_n \left(\frac{3}{2} c_n - \frac{1}{2} c_{nn} \right) + \Gamma_n A_n (c_n - c_p) / dy \quad (40)$$

$$F_s = U_s A_s \left(\frac{3}{2} c_p - \frac{1}{2} c_n \right) + \Gamma_s A_s (c_p - c_s) / dy$$

(41)

Finally, the weighting factors would be extracted as follows :

$$a_e = \Gamma_e A_e / dx \quad (42)$$

$$a_w = \frac{3}{2} U_w A_w + \frac{1}{2} U_n A_n + \Gamma_w A_w / dx \quad (43)$$

$$a_{ww} = -\frac{1}{2} U_w A_w \quad (44)$$

$$a_s = \Gamma_s A_s / dy$$

(45)

$$a_n = \frac{3}{2} U_n A_n + \frac{1}{2} \Gamma_n A_n / dy \quad (46)$$

$$a_{nn} = -\frac{1}{2} U_n A_n \quad (47)$$

$$a_p = \frac{3}{2} U_w A_w + \Gamma_w A_w / dx + \frac{3}{2} U_n A_n + \Gamma_n A_n / dy + \frac{1}{2} U_e A_e + \frac{1}{2} U_s A_s + \Gamma_e A_e / dx + \Gamma_s A_s / dy - \frac{1}{2} U_w A_w - \frac{1}{2} U_n A_n \quad (48)$$

The second order upwind scheme is a second order accurate scheme, which can better afford the steep gradients compared to the first order upwind schemes (e.g, POW scheme). This scheme also can reduce false diffusions during the computation. But, because of the existing negative weighting factors it may result in unphysical outcomes in some cases (e.g., overestimation of the eddies or secondary flows in the computational domain) (Haun, 2012).

3.2.5 Calculation of bed and critical bed shear stress

In SSIIM program, the bed shear stress is calculated using the velocity in the bed cell, roughness, and bed forms. To do this, the shear velocity that is calculated by wall laws (Schlichting, 1979) is used (Haun, 2012) :

$$\tau_{bed} = \rho u_*^2 = \rho \left(\frac{U_{bed} \kappa}{\ln\left(\frac{30y}{k_s}\right)} \right)^2 \quad (49)$$

where τ_{bed} is the shear stress at the bed, U_{bed} is the velocity at the bed cell. Other variables were defined in Eq. (12). If the $k-\epsilon$ turbulence model is employed, one can assume that production and dissipation of turbulence would be in equilibrium in areas close to the wall. Consequently, the shear stress at the bed (i.e., τ_{bed}) can be computed through the turbulent kinetic energy close to the bed (Olsen, 2014 ; Haun, 2012) :

$$\tau_{bed} = \sqrt{c_p} \rho k = 300k \quad (50)$$

For computing the critical bed shear stress, the empirical curve by Shields (1936) is used. Thus, the particle Reynolds number (i.e., R_*) is calculated using Eq. (51) for each sediment fraction. The particle Reynolds number then is used for parameterization of the Shields curve using the Eq. (52) (Haun, 2012). This number is also related to the dimensionless shear stress (i.e., τ_*). Subsequently, it can be concluded that critical shear stress is a function of both bed sediment size and hydraulic condition (Chanson, 2004).

$$R_* = \frac{u_* d_i}{\nu} = \frac{d_i \sqrt{\frac{\tau_o}{\rho_s}}}{\nu}$$

(51)

$$\text{If } R_* > 500 \quad \text{then} \quad \tau_* = 0.06 \quad (52)$$

$$\text{If } R_* < 500 \quad \text{then} \quad \log(\tau_*) = a (\log R_*) + b (\log R_*)^2 + c (\log R_*)^3 + d (\log R_*)^4$$

$$a = -0.99863612 ; b = -0.92539586 ; c = 0.54283631 ; d = -0.08454406$$

where u_* is the shear velocity, d_i is the size of the i th fraction of the bed material, τ_o is the shear stress on the bed, ρ_s is the density of the sediment particles, and ν is the viscosity of the fluid.

The dimensionless Shields parameter also is achieved using the following equation :

$$\tau_* = \frac{\tau_o}{g(\rho_s - \rho_w)d_i} \quad (53)$$

where τ_o is the shear stress on the bed, ρ_w is the density of the water, and g is the gravity acceleration. Other parameters were also defined.

3.2.6 Calculation of bed level changes

When the existing bed shear stress exceeds the critical bed shear stress, the erosion process would be initiated. Thus, bed level changes occurs. Then, the vertical bed level changes is calculated with respect to the continuity defect in the cell close to bed using the following equation (Haun, 2012) :

$$A\delta z_0 = c_f(S_i - S_o) \quad (54)$$

where δz_0 is the vertical bed level movement, A is the bed cell area in a horizontal plane, c_f is a conversion factor between the sediment flux and bed level movement, S_i is the sediment inflow, and S_o is the sediment outflow quantities.

If the bed material contains the cohesive fine particles, or bed forms occur, or the bed armoring effects exist, threshold condition for sediment erosion will be changed and the corresponding algorithms should be modified accordingly to take such kind of effects into account.

3.2.7 Bed slope effect on calculation of critical bed shear stress

The threshold condition for sediment erosion (i.e, critical bed shear stress) on sloping beds is different than on flat beds. Then, critical bed shear stress on sloping beds is a function of gravity and tractive forces beside the hydrodynamic forces. Thus, the critical bed shear stress that is obtained from the Shields curve should be corrected using a correction factor as shown in Eq. (55).

$$\tau_{c,i}^* = K\tau_{c,i,0}^* \quad (55)$$

where $\tau_{c,i}^*$ is the reduced value of the critical shear stress for the i th fraction of the sediment and $\tau_{c,i,0}^*$ is the original value of the critical bed shear stress obtained from the Shields curve and K is the reduction factor that can be obtained using different formulas.

In SSIIM program, Brooks (1963) formula is employed to calculate the K factor for taking into account the slope effect in reduction of the critical bed shear stress:

$$K = -\frac{\sin\alpha.\sin\beta}{\tan\phi} + \sqrt{\left(\frac{\sin\alpha.\sin\beta}{\tan\phi}\right)^2 - \cos^2\alpha \left[1 - \left(\frac{\tan\alpha}{\tan\phi}\right)^2\right]} \quad (56)$$

where β is the angle between streamline and direction of near bed flow, α is the bed slope, and ϕ is a slope parameter similar to the angle of repose (Rüther & Olsen, 2007).

3.2.8 Sand slide modelling

Brooks (1963) formula together with a sand slide algorithm is used for modelling the side bank erosion in SSIIM program. The sand slide algorithm corrects the bed slope when it exceeds a defined critical

angle of repose (i.e., φ) of the sediment during the excessive erosion. In fact, sand slide algorithm acts as a limiter for K when erosion continues and bed slope increases.

Moreover, side slope effect is taken into account on computing the deviation angle between the flow direction and sediment transport on the laterally inclined slope. This is done by introducing an empirically derived modified Shields parameter considering the skin friction of the sediments introduced by Engelund (1981) shown in Eq. (57) & (58) :

$$\tan \psi = \frac{0.6}{\sqrt{\tau'_*}} \cdot \frac{\partial h}{\partial n} \quad (57)$$

$$\tau'_* = 0.4 * \tau_*^2 + 0.06 \quad (58)$$

where ψ is the angle between the near bed flow direction and sediment transport on the laterally inclined slope, $\partial h / \partial n$ is the transversal (i.e., lateral) slope of the bed, τ'_* is the Shields parameter related to the skin friction and τ_* is the Shield parameter as define in Eq. (53) (Fischer-Antze et al., 2008). As long as bank materials are not influenced by cohesion force, mentioned method works suitably for modeling the bank erosion. Examples are 3-D CFD modeling of free-form meandering channel evolution (Rüther & Olsen 2007), morphological bed changes in Danube River (Fischer-Antze et al. 2008), flushing operation in Angostura reservoir (Haun & Olsen, 2012b) and also in an Alpine Reservoir (Haun et al., 2012). However, applying more comprehensive bank erosion approaches will result in a more accurate simulation of this geotechnical feature.

Chapter 4 Hydrodynamic features and sediment flushing in shallow reservoirs

Shallow flows are described as a flow condition in which the vertical dimension of the fluid domain is noticeably smaller than its horizontal dimensions (Yuce and Chen, 2003). Flows in wide rivers, lakes, coastal lagoons, estuaries and large reservoirs are examples of shallow waters in the prototype scale. A flow pattern in wide and shallow reservoirs with a sudden expansion of the inlet section may become unstable, which produces large-scale transversal motions and recirculation zones. This phenomenon can be attributed to the high sensitivity of the flow pattern to the initial boundary condition at the inlet section (e.g., small transverse disturbance) (Dewals et al., 2008). When large-scale transverse motions and turbulent coherent structures develop in shallow reservoir, the sediment transportation pattern (i.e., erosion and deposition) is significantly affected by the velocity field.

Concerning the sediment management issue in dam reservoirs (e. g., free-flow flushing operation), the shallow-flow condition emerges during the sediment-flushing operation with full drawdown and plays a significant role in removing the deposited sediment from the reservoir. When shallow flow emerges with symmetric or asymmetric patterns, the flow domain exhibits complex three-dimensional (3-D) features (e.g., helical flows). Thus, this chapter focuses on the numerical modelling of the velocity field in shallow reservoirs with different geometries and also different bed conditions (i.e., flat and misshaped bed). The experimentally measured surface velocity in all cases and over-the-flow depth velocity in two cases were used to validate the model. Afterwards, numerical model was utilized to simulate the flushing channel development pattern in shallow reservoirs. Outcomes will be useful to establish a better understanding on the physical process governing the flushing channel development. This understanding can be further utilized for predicting and managing the shallow-flow behaviour to increase the flushing efficiency when free-flow flushing operation is performed in real prototype reservoirs.

4.1 Introduction

Shallow flows are predominant in nature and also emerges in many engineering applications such as sudden expansions (Shapira et al., 1990), compound channels (Ghidaoui & Kolyshkin, 1999; Chu et al., 1991), storage chambers (Stovin & Saul, 1996; Adamson et al., 2003), settling tanks (Frey et al., 1993), shallow reservoirs sedimentation (Kantoush et al., 2008a & 2010; Dufresne et al., 2012; Camnasio et al., 2013) and sediment flushing (Kantoush & Schleiss, 2009; Esmaeili et al., 2014b).

The effect of geometric and hydraulic boundary conditions on the flow pattern of shallow reservoirs was clarified in the experimental tests of Kantoush (2008) and Dufresne et al. (2011). Kantoush (2008)

presented a comprehensive review of experimental tests in a series of shallow reservoirs with transverse flow motions in the symmetric channel expansions. His study revealed 3-D features of the flow structure in shallow reservoirs (i.e., secondary flows and 3-D stretching vortices). In addition, the experimental tests show that an asymmetric flow pattern emerges under a certain geometric and hydraulic condition despite the perfect symmetric geometry and hydraulic condition. Similar results were obtained by Stovin & Saul (1996) and Adamson et al. (2003) regarding storage chambers and storage tank sedimentation, respectively. Kolyshkin and Ghidaoui (2003) came to a similar conclusion about the development of an asymmetric flow pattern in the wake flows. Recently, Peltier et al. (2014a) presented a review on the experiments regarding the shallow reservoirs and suggested the domains for existence of different flow patterns, including meandering flows, in shallow reservoirs (Peltier et al., 2014b).

Dewals et al. (2008), Dufresne et al. (2011), and recently Peltier et al. (2014c) used 2D numerical models to investigate the turbulent flow patterns in rectangular shallow reservoirs. As long as secondary current effects and velocity variations over the flow depth are not significant, 2D models can be utilized. However, the geometry condition may be more complex in practical cases and a complex 3-D flow pattern can emerge. 2D numerical models cannot simulate the secondary current effects directly and particularly the velocity variations over the flow depth on misshaped beds of shallow reservoirs. Stansby (2006) concluded that 2D depth averaged models could not consider the flow curvature over the bed friction that emerged because of the vertical mixing, which is induced by horizontal strain rates. This inability may lead into a significant underestimation of the bed friction in some cases. The complexity of 3-D flow patterns is further magnified over the existing bed forms, on the misshaped bed, that is obtained after flushing and lowering the water level. Because completely different flow patterns may appear over the flow depth, knowledge about the vertical distribution of the streamwise and lateral velocity, which is reproduced using 3-D numerical simulations, can provide a more precise evaluation of morphological processes in shallow reservoirs. Knowledge about shallow flows leads to a more efficient sediment management strategies in reservoirs and settling basins (Kantoush et al., 2011a & 2011b). Nonetheless, 3-D numerical modelling of the symmetric and asymmetric turbulent flow field in shallow reservoirs with different geometries is scarce (Esmaeili et al., 2014a).

In this chapter, 3-D modelling of the velocity field on flat and misshaped beds of shallow reservoirs, without sediment transport condition, was performed using SSIIM 1 program. Outcomes were compared and validated with the experimental measurements. Moreover, the flushing channel formation and evolution trend were simulated in a series of shallow reservoirs to have a deeper insight regarding the sediment transportation processes (i.e., erosion and deposition) during a free-flow flushing operation.

4.2 Physical model setup, experimental conditions, and study cases

The experimental tests were performed at the Laboratory of Hydraulic Constructions of Swiss Federal Institute of Technology (EPFL) in a rectangular reservoir with a maximum inner length (L) of 6 m and width (B) of 4 m (Kantoush, 2007). The inlet and outlet rectangular channel width (b) and length (l) were

0.25 m and 1 m, respectively. Both channels were located at the center of the upstream and downstream side walls of the reservoir. Various shallow reservoir geometries could be achieved by adjusting a moveable PVC plate wall. The reservoir depth was 0.3 m, and both side walls and the bottom floor were hydraulically flat. However, small distortions in flat bed of reservoirs may exist in mm accuracy due to the fabrication of PVC plates and setup as well (Kantoush 2008). A movable frame with 4 m length was mounted on the side walls of the reservoir to install the measurement devices. Adjacent to the reservoir, a mixing tank was used to provide the water-sediment mixture that can be drained into the reservoirs by gravity force. Non-uniform crushed walnut shells were used as suspended sediment for this purpose. The median size of this non-cohesive light-weight and homogenous grain material was $50\ \mu\text{m}$ with a density of $1500\ \text{kg/m}^3$ and a standard deviation of 2.4. Figure 4.1 illustrates the schematic view of experimental setup described above.

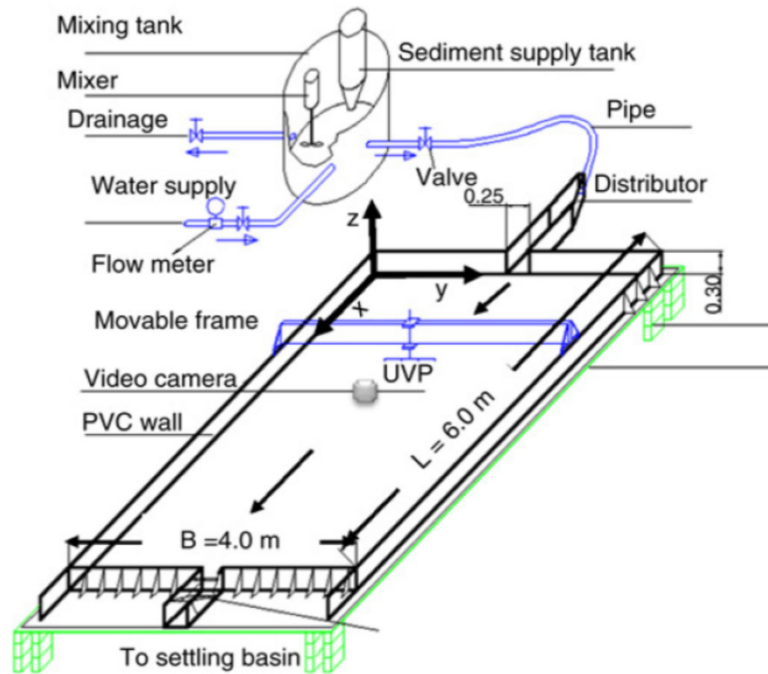


Figure 4.1 Schematic view of experimental facilities installation for shallow reservoirs tests (Kantoush, 2008).

In the overall frame work, the physical model study and measurements were carried out in three different phases with non-continues procedure. This chapter corresponds with the phase 1 and 3 of the frme work. In phase 1, the shallow reservoirs with flat beds were filled with clear water and after reaching the stable state the velocity measurements were performed. In phase 2, the mixture of water and non-uniform crushed walnut shells was drained into the reservoir for modeling the sediment deposition and in phase 3 the final bed topography from the second phase was used as the initial bed for performing the flushing with lowering the water level (i.e., free-flow flushing). After developing the flushing channel in the reservoir, when changes are negligible in flushing channel shape and size, the surface velocity was measured over the misshaped beds. The flow discharge rate (Q) and water depth (h) were fixed for all experiments as $0.007\ \text{m}^3/\text{s}$ and $0.2\ \text{m}$ in phase 1, respectively. Thus, in all examined configurations with the flat bed, the measured Froude number was small as $Fr=0.1$ and Reynolds

number was high as $14000 \leq Re \leq 28000$ to ensure that a turbulent flow was developed. For the flow field measurements on misshaped beds after flushing, the water level and discharge were 0.1 m and 0.007 m³/s, respectively.

Table 4.1 shows the geometrical attributes of five reservoirs used in the present study and corresponding shape factors (i.e., SF) described by Dufresne et al. (2011). According to Dufresne et al. (2011), in shallow reservoirs with flat beds, the flow pattern was symmetric (i.e., S0) when SF was approximately lower than 6.2 and flow pattern was asymmetric (i.e., A1) when SF was approximately bigger than 6.8. Thus, in the applied geometries, T11 and T13 have S0 flow pattern whereas T7, T8 and T9 have A1 flow pattern.

Case	b(m)	L(m)	B(m)	ΔB^*	SF(-)**
T7	0.25	6	3	1.375	8.63
T8	0.25	6	2	0.875	11.32
T9	0.25	6	1	0.375	18.82
T11	0.25	5	4	1.875	5.97
T13	0.25	3	4	1.875	3.58

* ΔB is equal to $(B-b)/2$

** SF is the shape factor introduced by Dufresne et al. (2011), which is defined as $L/\Delta B^{0.6} b^{0.4}$.

Table 4.1 Geometrical attributes of experimental cases employed by Kantoush (2008).

4.3 State-of-the-art facilities for experimental measurements

The water level in the reservoir was controlled using a 0.25 m wide and 0.3 m high flap gate at the end of the outlet channel. Large Scale Particle Image Velocimetry (i.e., LSPIV) technique was used to measure the surface velocity field, and Ultrasonic Velocity Profiler (i.e., UVP) devices were used to provide the 3-D flow velocity measurements (Kantoush et al., 2008b). Each UVP device, can instantaneously measure 1D velocity profile over the flow depth. A set of three UVP probes, which were inclined at 20° to the vertical axis were installed on the movable frame, allowed measuring the 3-D flow field. In each position of measurement, in each designated cross-section, four groups of three probes were installed for data acquisition. Depends on the cross-section width, number of UVP groups can vary accordingly. The first valid UVP measurements were located at 12.5 cm away from the side walls and 2.5 cm from the free water surface. Additionally, in the framework of the experimental study, the surface velocity was measured after sediment flushing from the shallow reservoirs. Plastic particles with a density of 960 kg/m³ and average diameter of 3.4 mm were used as seed for LSPIV measurements (Kantoush et al., 2008b). Non-uniform crushed walnut shells were used as fluid tracers to provide ultrasound reflection for UVP devices and also used as suspended material for modelling the sediment deposition. In addition, the evolution of the bed level and water level were measured with a miniature echo sounder and an ultrasonic probe, respectively. Figure 4.2 shows the schematic configuration of LSPIV and UVP devices for acquisition of velocity data.

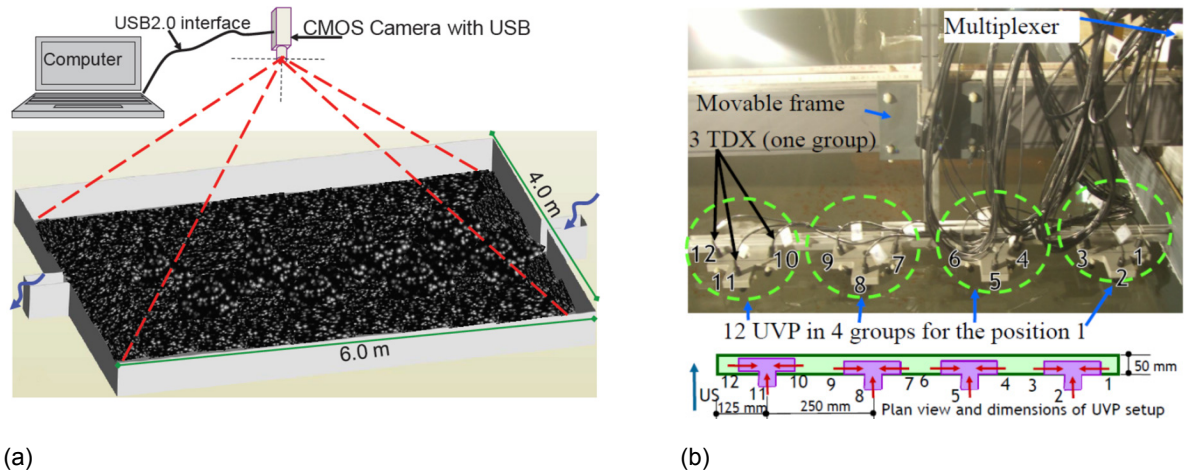


Figure 4.2 Schematic figure of (a) LSPIV configuration; (b): UVP configuration (Kantoush, 2008).

4.4 Flow fields domain in shallow reservoirs with flat bed

The assessment of the flow field is necessary to characterise the domain of the main jet flow, reverse flow and eddies in a shallow reservoir. It should be noted that flat bed condition is a simplified type of bed geometry in shallow reservoirs since the bed condition is usually misshaped in practical cases. However, analyzing the flow pattern distribution over the flat beds would be beneficial to establish a better understanding about the effect of initial boundary condition on development of a symmetric or asymmetric flow pattern in reservoirs.

Kantoush (2008) observed that in case T7 the issuing flow jet deviated to the right-hand side and developed an A1 flow pattern. The main eddy rotated anticlockwise in the centre part of the reservoir, and two smaller ones rotated clockwise in the upstream corners. Additionally, a S0 flow pattern with one main jet trajectory in the centreline and two circulation zones on each side was developed for case T13. Kantoush (2007) concluded that the deviation to the right-hand side occurred because of the random disturbance of the initial flow boundary condition, and a mirror situation would be easily established by slightly disturbing the initial boundary condition. The flow deflection to one side of the reservoir can be attributed to the difference in flow velocity along one side of the main jet compared with the other side and the consequential pressure difference. The local pressure difference deviates the flow towards one side of the reservoir and is called the Coanda effect (Chiang et al., 2000).

More detailed analysis about the simulated and observed flow pattern over shallow reservoirs with flat bed is provided in the following parts. Discussion and analysis was divided into three categories including the symmetric and asymmetric flows, 2-D surface velocity field, and 3-D velocity field.

4.4.1 Symmetric and asymmetric flow development

Because the S0 (i.e., symmetric) flow pattern was not observed in the physical model experiments for special geometries, Dewals et al. (2008) introduced a slight disturbance in the initial boundary condition for 2D numerical simulations. They used a non-uniform cross-sectional discharge in the inlet boundary

to examine the stability of the numerical model outputs. The identical concept of slight disturbance in the inflow boundary condition was implemented herein for all runs, and the non-uniform cross-sectional velocity distribution was used in the inflow boundary condition:

$$V_{in}(x) = V_0 \left[1 - \alpha \left(0.5 + \frac{y}{b} \right) \right] \quad (1)$$

where V_{in} is the actual streamwise velocity value, which is specified as the inflow boundary condition, V_0 is the reference value (i.e., total discharge divided by inlet cross section area), α measures the magnitude of the linear variation, b denotes the inlet channel breadth, and y is the coordinate along the transversal direction, which changes between $-b/2$ and $b/2$ (i.e., right and left sides of the inlet channel, respectively). Assuming that $\alpha=2.5\%$ for numerical modelling, the initial velocity magnitude differs by 2.5% at one side of the inlet channel compared to the other side. This type of disturbance is inevitable in the experimental setup. Nonetheless, a notably small perturbation of the inflow condition will significantly affect the A1 (i.e., asymmetric) flow pattern in the numerical results. Changing α between 1 and 4% shows a notably close flow field to that observed with the physical model except for case T7, in which there is a threshold value for α . For this case, the smallest value of α to reproduce the A1 flow pattern is 2.5% and this value is also used for other cases with A1 flow pattern. This type of results reveals the unstable nature of symmetric flows in such geometries and consequently the high sensitivity of the flow field to the inflow boundary condition. Depending on which side of the inlet has the higher velocity, the jet deflection to each side of the reservoir (i.e., right or left) can be obtained. A Limited number of runs also showed the contribution of the bed and side-walls roughness in the development of the A1 flow pattern in case T7 even with α of 1% when the roughness increases. A higher roughness can affect the initial flow condition, which is consistent with the findings of Chu et al. (1991) about the effect of friction on the velocity profile and consequently the flow pattern. However, an intensive numerical study should be performed in the future with a physical-model study on the interaction between slight disturbances in the inflow boundary condition and the side and bed roughness and their effect on the flow pattern.

Figure 4.3 shows the jet evolution pattern of case T7 after a jet was issued into the stagnant shallow water, which eventually established the steady asymmetric flow condition. First, when the main jet flow attaches to the downstream wall at $t=240$ s, it returns backward, which causes the energy dissipation and velocity reduction as shown in Figure 4.3(a). Because of the non-uniform velocity distribution and the consequential pressure difference (Coanda effect), the flow field is transversally unstable, and the main jet flow slightly deviates to the left side. In the subsequent stage of $t=240-480$ s, a transitional phase occurs, where the jet gradually deviates from the left side to the right side as shown in Figure 4.3(b). Similar to the first stage, the locally reduced pressure because of a higher velocity on one side than the other side tends to amplify the jet deflection. Meanwhile, upstream corner vortices are formed, and their size increases, which controls the centre anti-clockwise vortex. Afterwards, the main jet flow reattaches to the right side wall, and a steady A1 flow pattern finally emerges over the shallow reservoir

as illustrated in Figure 4.3(c). The simulated jet evolution pattern was consistent with the experimental observations. Furthermore, an identical jet evolution pattern was obtained for cases T8 and T9.

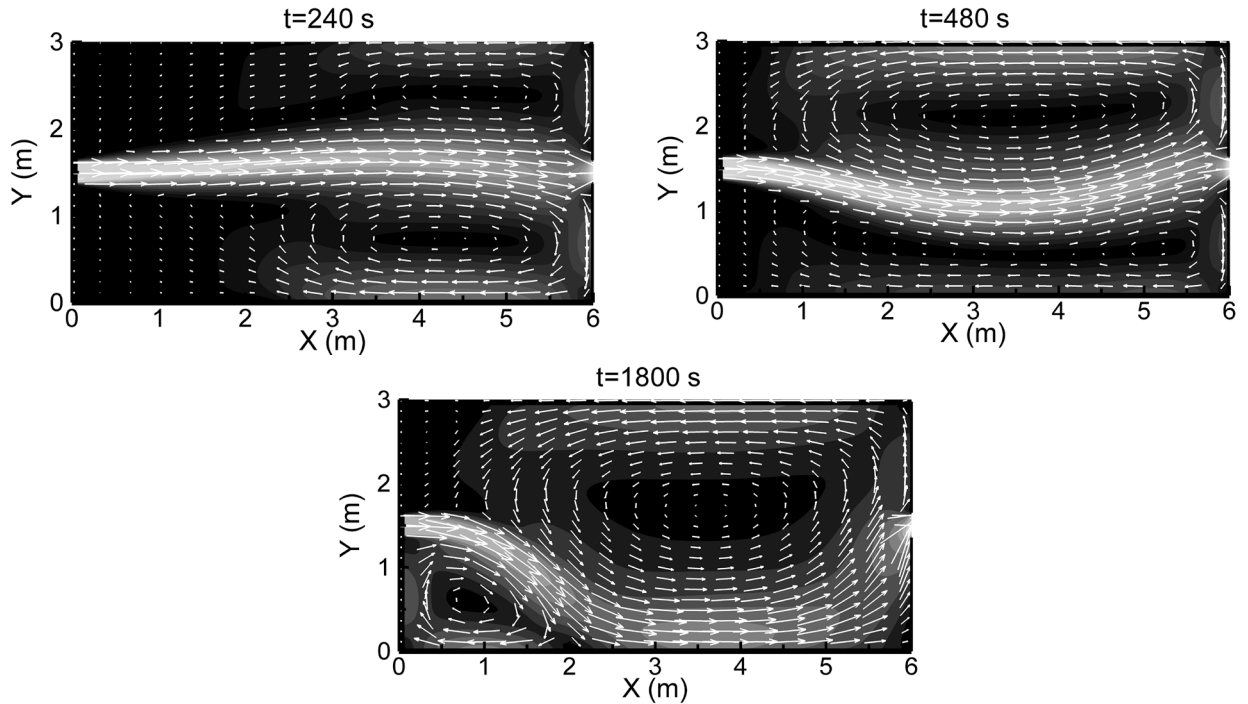


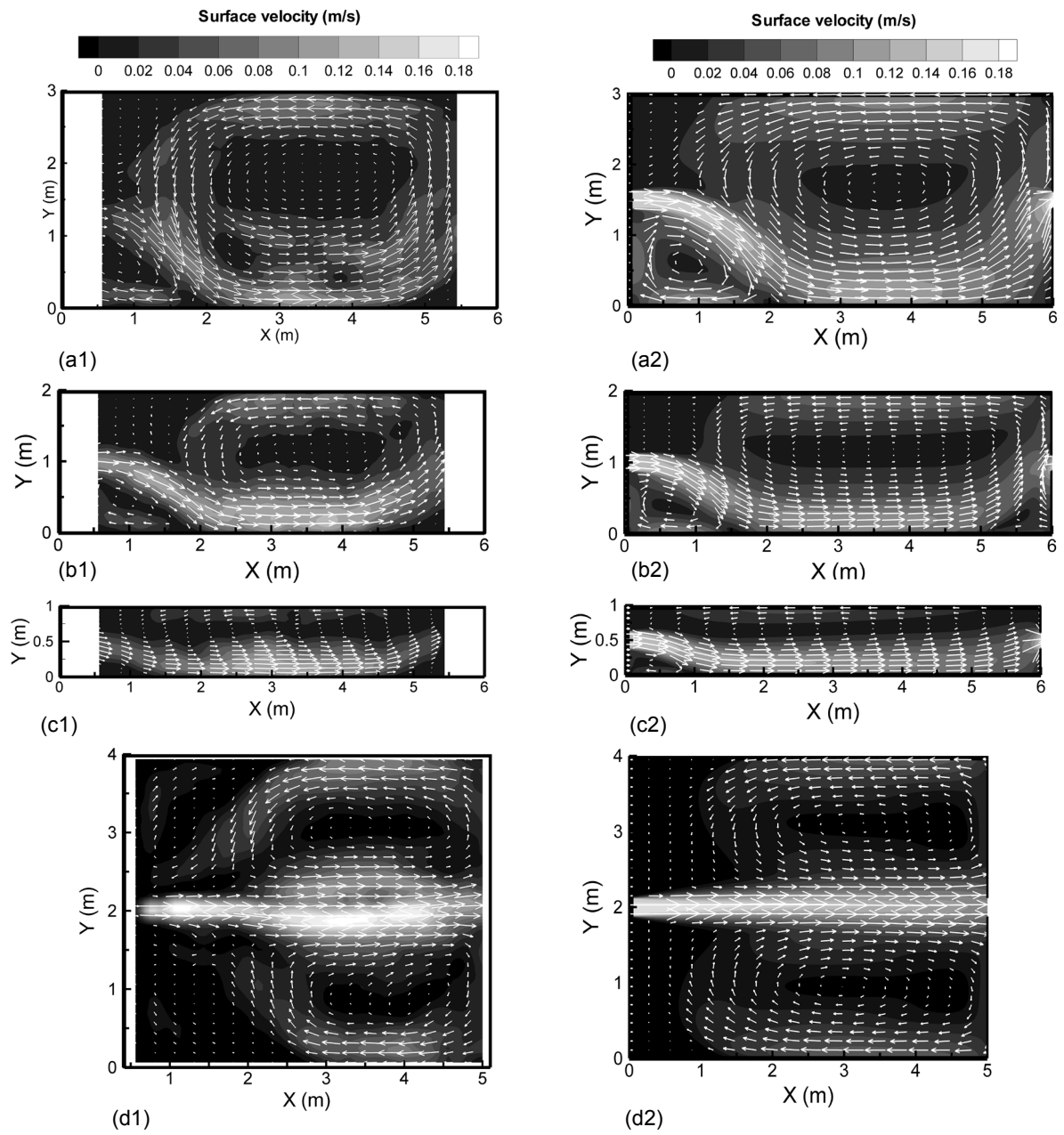
Figure 4.3 Various stages of asymmetric flow pattern development in shallow reservoirs with flat bed: (a) attachment of main jet flow to the downstream wall ($t=240$ s); (b) deviation of main jet flow from centreline to the right hand side during the transitional stage ($t=480$ s) and (c) attachment of main jet flow to the right side wall ($t=1800$ s).

4.4.2 Simulation of 2-D surface velocity

Figure 4.4 illustrates the measured and simulated surface velocity magnitudes and flow distribution patterns for five selected experiments. As shown in Figure 4.4, an A1 flow pattern develops in cases T7, T8 and T9, whereas an almost S0 flow pattern develops in cases T11 and T13. The model can simulate a surface flow velocity pattern that is similar to the measured pattern by reproducing the dominant aspects of the flow field, such as the main flow jet trajectory, location of the reverse flow, main vortices and corner gyres. Nevertheless, the numerical model results show a straighter and longer reverse flow trajectory than the observations and a concentrated main jet flow for all cases. Subsequently, the upstream corner gyres in the numerical outputs have a smaller size than those of the experimental measurements. This situation is predominant for T11 (Figures 4.4(d1) and 4.4(d2)).

In the numerical simulation, the time step was set as 2 seconds for the runs T8 and T9, whereas it was 0.5 second for T7, T11 and T13. Smaller time steps contributed to faster and more stable convergence of the computations in wider reservoirs. Simulations were performed using the geometry and inflow/outflow boundary conditions that were similar to the physical model. The $k-\epsilon$ turbulence model was used, and simulations were conducted until a steady-state flow condition was obtained. The simulations show that the model cannot reproduce an A1 flow pattern when the geometry configuration and hydraulic boundary condition are perfectly symmetric because the applied mathematical algorithms

were not intended to reproduce this type of artificial asymmetric numerical results when the input boundary condition is symmetric.



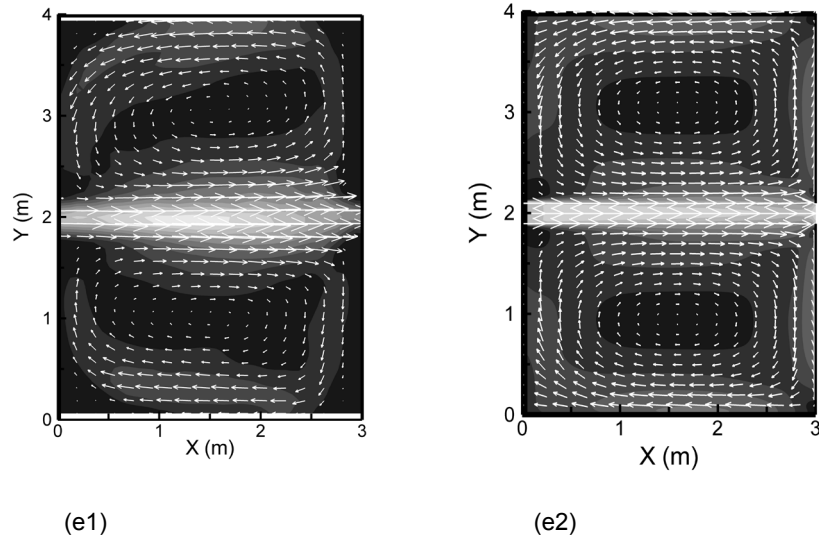
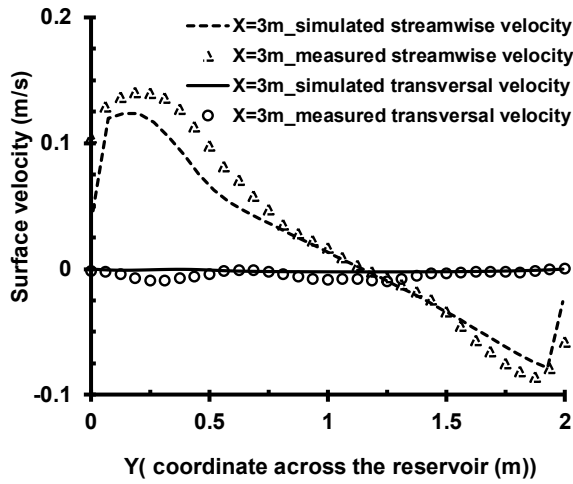
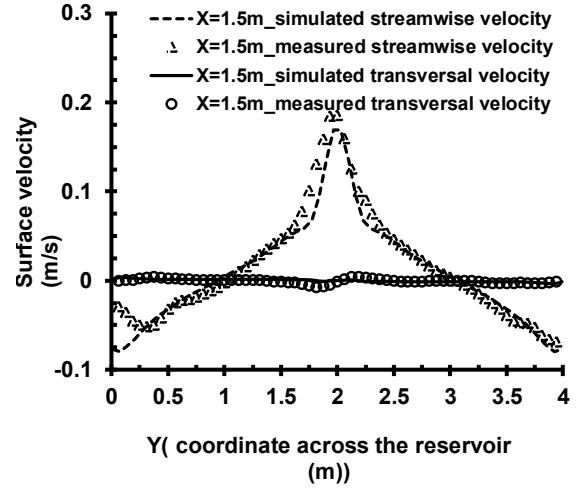


Figure 4.4 Left: the measured surface velocity field with velocity vectors over flat bed for case: (a1) T7; (b1) T8; (c1) T9; (d1) T11; (e1) T13, respectively and right: corresponding simulated velocity field for each case

Figure 4.5 quantitatively demonstrates the simulated streamwise and transversal surface velocity distribution versus the measured one at the middle cross-section of the reservoirs for cases T8 and T13. The numerical model results are consistent with the measurements in case T8, which has an A1 flow pattern as shown in Figure 4.5(a). Figure 4.5(b) shows that in case T13, there is a slight discrepancy between the simulated and measured surface velocity fields, particularly along the centreline and side walls of the reservoir. The reasons can be the concentrated simulated flow pattern with a lower diffusion of the main jet and the reverse jet flow compared to the measurements. Furthermore, Figure 4.6 shows similar outputs in both the upstream and the downstream areas of case T7. The overall trend of surface velocity variations was reproduced using the numerical model. To provide a higher-resolution longitudinal distribution of the surface velocity, the simulated streamwise velocity along the reservoir length (i.e., beside the right wall, along the centreline and beside the left wall) was plotted against the measured one for case T8 and over the right half of case T13 in Figure 4.7. As shown in Figure 4.7(a), regarding case T8, except the area near the inlet and outlet, the numerical model results are reasonably consistent with the measurements. An identical condition was found for the other cases with A1 flow pattern (e.g., cases T7 and T9). The longitudinal velocity distribution for the left half of case T13, as depicted in Figure 4.7 (b), is notably close to that of the right half. Figures 4.5, 4.6 and 4.7 show that the numerical results are globally consistent with the measured surface velocity components in different geometries with different flow patterns.

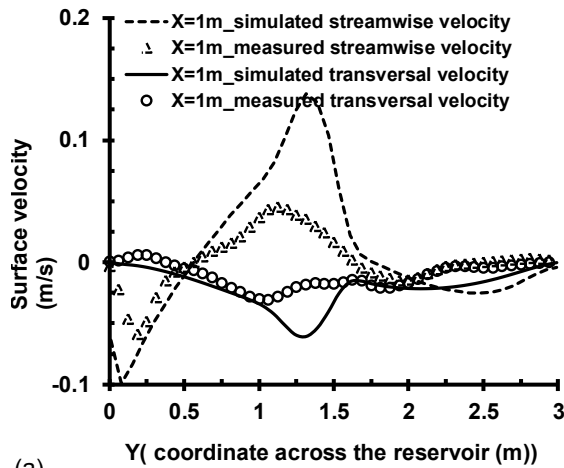


(a)

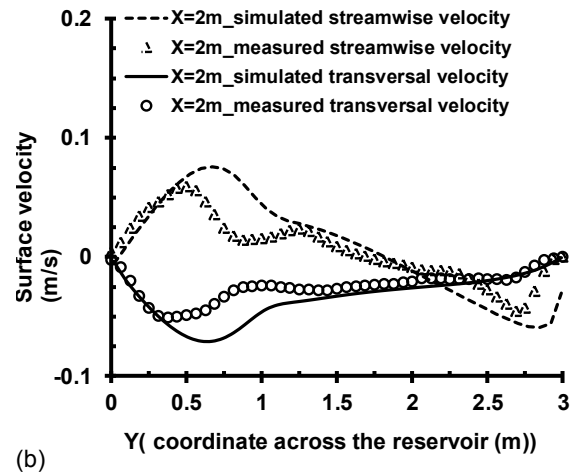


(b)

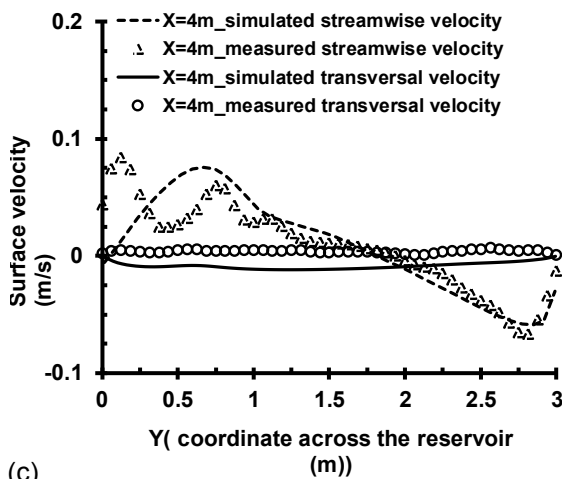
Figure 4.5 The measured streamwise and transversal surface velocity versus the simulated one at middle of the reservoirs length with flat bed condition for case: (a) T8; (b) T13.



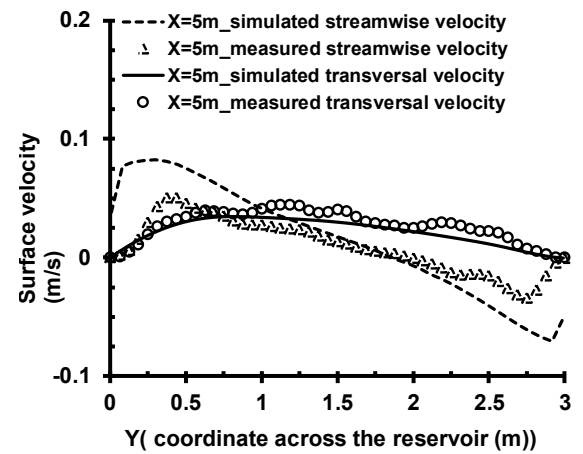
(a)



(b)



(c)



(d)

Figure 4.6 The measured streamwise and transversal surface velocity versus the simulated one for case T7 at: (a) $x=1$ m; (b) $x=2$ m; (c) $x=4$ m; (d) $x=5$ m.

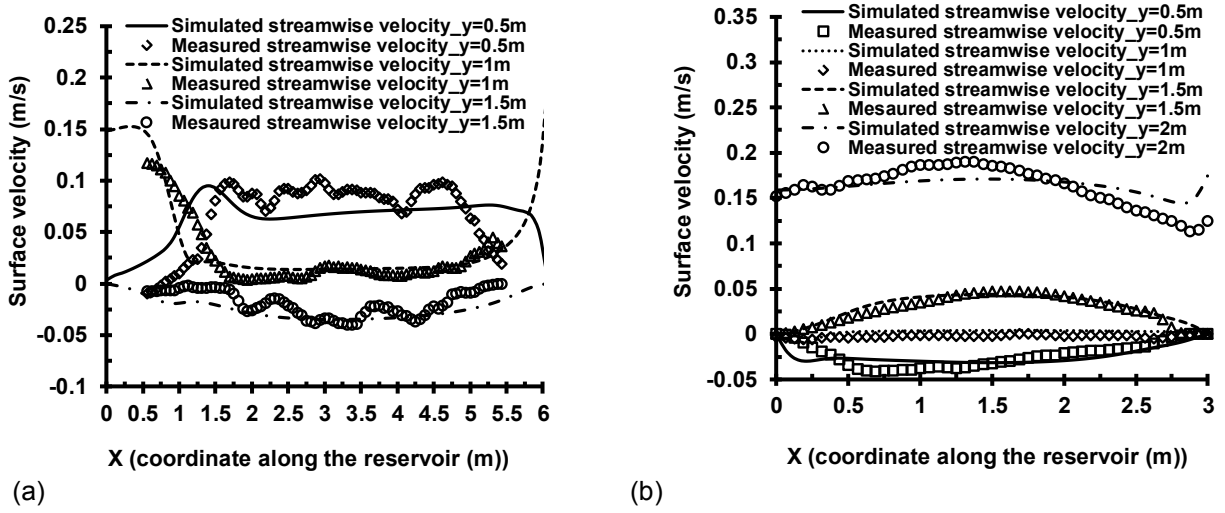
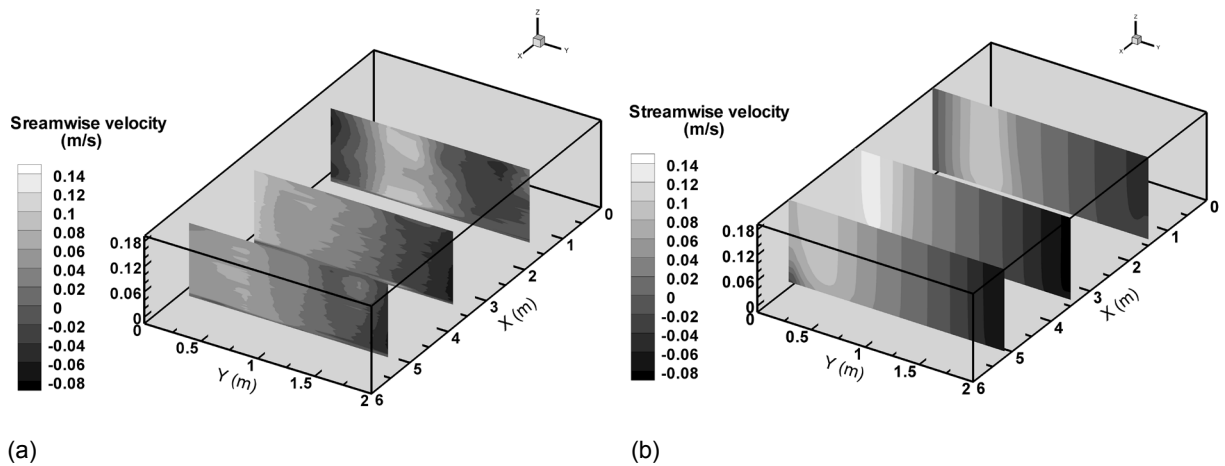


Figure 4.7 Longitudinal distribution of surface streamwise velocity over the flat bed reservoirs for: (a) case T8; (b) right half of case T13.

It should be noted that there may be more than one solution for initiating the A1 flow pattern. Thus, other types of slight disturbances of the inlet boundary condition can produce the A1 flow pattern in relevant cases (e.g., slight deflection of the inlet channel or combination of the inlet channel deflection and non-uniform cross-sectional velocity).

4.4.3 Simulation of 3-D velocity field

The velocity field distribution over the flow depth is important for analyzing the sediment transportation in reservoirs. Thus, the numerically simulated 3-D flow velocity field in the reservoirs with flat beds was compared with the measured 3-D velocity components that were provided using the UVP measurements. Figures 4.8(a) and 4.8(c) show the measured streamwise velocity distribution over the flow depth in upstream, middle, and downstream areas of cases T8 and T9, respectively. Figures 4.8(b) and 4.8(d) correspond to the simulated streamwise velocity for the mentioned case, respectively.



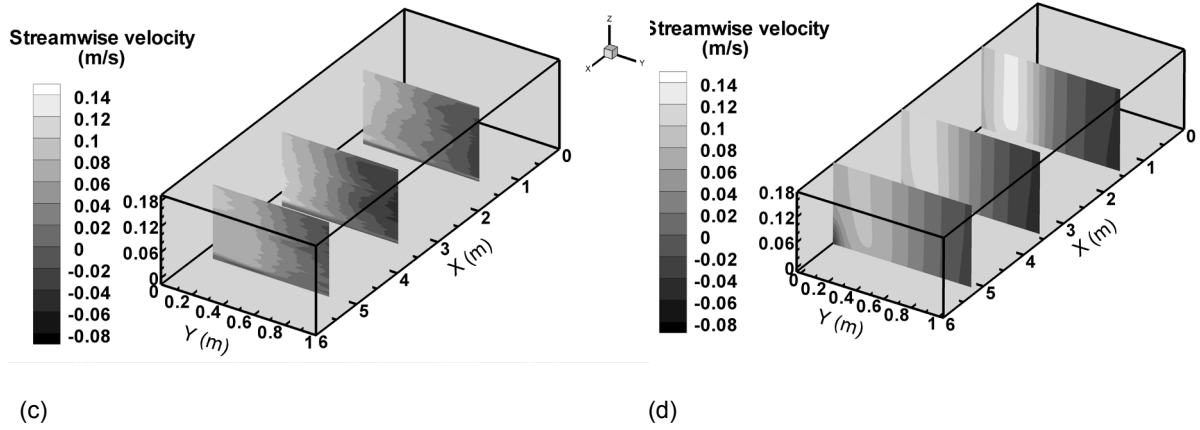
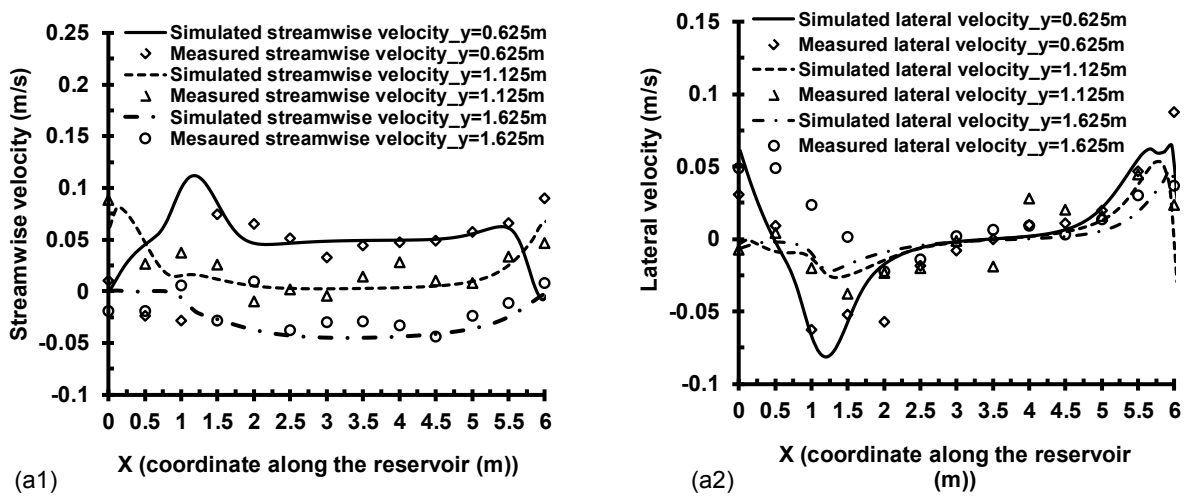


Figure 4.8 Left: The measured streamwise velocity distribution over the flow depth in shallow reservoirs with flat bed using UVP devices at $x=1.5$ m, 3.5 m and 5.2 m for case: (a) T8; (c) T9 and right: corresponding simulated 3-D velocity field for case: (b) T8; (d) T9.

Numerical outputs in Figure 4.8(b) and 4.8(d) reveal that higher longitudinal velocity is deflected towards the right bank side, and the reverse flow is reproduced near the left bank side. This change in flow direction across the reservoir is also qualitatively consistent with the experimental observations in Figure 4.8(a) and 4.8(c). The measurements also show that the vertical velocity magnitudes over the flow depth are notably small compared to the other velocity components (i.e., streamwise and lateral) when the reservoir bed is flat and horizontal. The discrepancy between the calculated and measured outputs can be attributed to the existing roughness of the side walls, which affects the flow field and was neglected in the computations.

To more comprehensively assess the numerical model results, the velocity field in the streamwise and lateral directions (i.e., U , V , respectively) at two different vertical levels from the bed (i.e., $Z=0.045$ m, 0.155 m), in three longitudinal sections for cases T8, is shown in Figure 4.9.



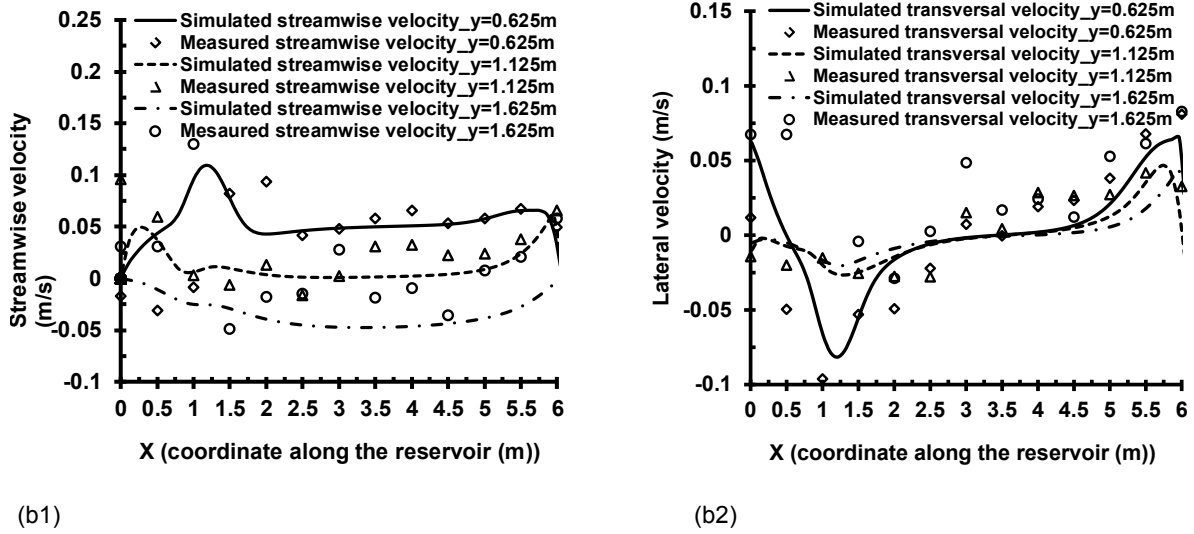


Figure 4.9 Left: the measured streamwise velocity over the flow depth using UVP versus the simulated one for case T8 with flat bed at (a1) $Z=0.045\text{ m}$; (b1) $Z=0.155\text{ m}$; right: measured lateral velocity over the flow depth using UVP versus the simulated one at (a2) $Z=0.045\text{ m}$; (b2) $Z=0.155\text{ m}$

This Figure shows that streamwise velocity magnitudes are different from the measured ones near the inlet, particularly at the level near the surface (i.e., $Z=0.155\text{ m}$). This difference is considerable for the longitudinal section adjacent to the left side wall (i.e., $y=1.625\text{ m}$) because a complex combined vortex is formed at the upstream left zone of the reservoir and consists of two smaller sub-vortices with different rotation directions. It should be noted that the numerical model mainly reproduces one vortex system in this area. The simulated streamwise velocities are generally consistent with the UVP measurements in other parts of the reservoir, although some fluctuations are found in the measurements. Regarding the transversal velocities, the numerical model outputs are reasonably consistent with the measurements except in the mentioned upstream zone. The numerical model outputs represent the negative transversal velocities, whereas the measured velocities show positive values because of the existing complex combined vortex.

4.5 Flow fields domain in shallow reservoirs with misshaped bed

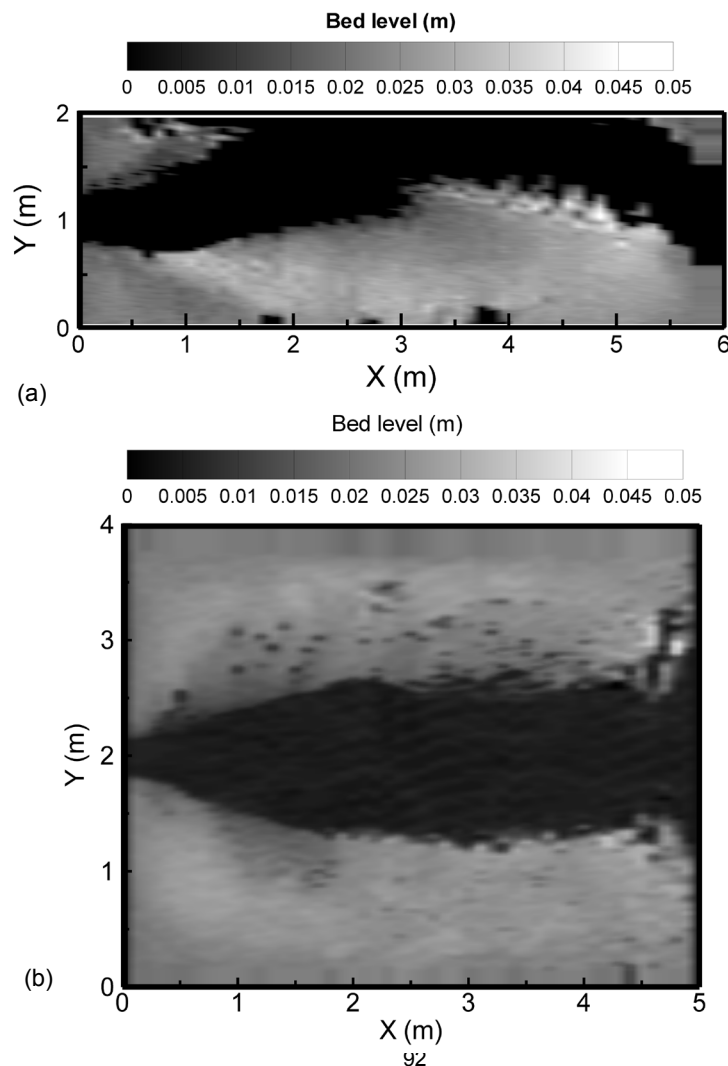
One type of practical application of 3-D numerical models can be the sustainable flood risk management in reservoirs, particularly those near urban areas, by effectively predicting the water levels and the consequential fluvial processes. In other word, the numerical model can be used for reproducing the complex flow field during the anticipated floods over the misshaped bed of existing reservoirs (e.g., after the flushing operation). In this part, discussion and analysis about simulated and observed flow pattern over shallow reservoirs with misshaped bed is supplied.

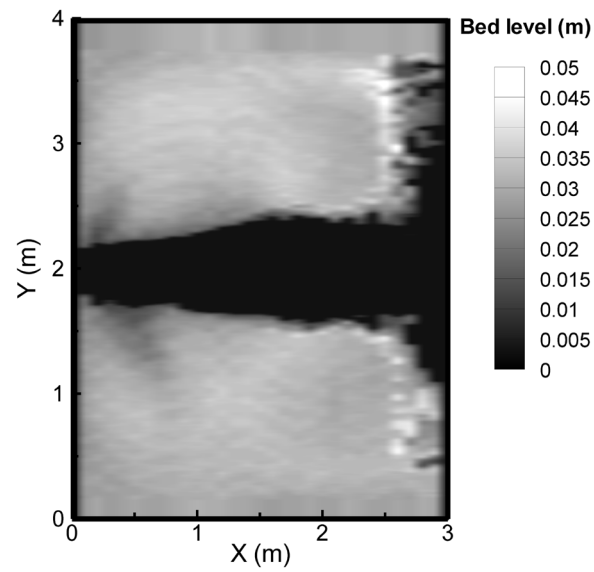
4.5.1 Simulation of 2-D surface velocity

The measured bed morphology for cases T8, T11 and T13 after sediment flushing and lowering the water head is shown in Figure 4.10. The bed morphology was introduced to the model as the initial boundary condition for each case to simulate the three-dimensional flow field. Moreover, a slight disturbance of the inflow boundary condition was not considered. Because of the shallower flow

condition with higher velocity components on the existing friction of various bed forms, the flow field simulation on misshaped beds is more complex than that on flat beds. For all three cases, the bed roughness was considered as 0.00015 m, which is 3 times the median grain size.

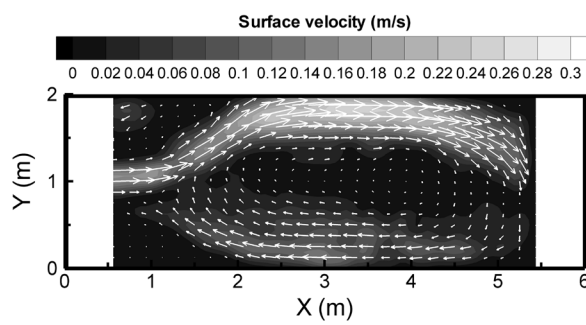
The measured surface velocity after flushing using the LSPIV technique and the simulated surface velocity using the 3-D model are shown in Figure 4.11. For case T8, Figures 4.11 (a1) and 4.11 (a2) show that the simulated hydraulic and geometric features of the main jet flow and reverse flow trajectory are slightly different from the measured ones. Here, the developed flushing channel attracts the jet flow and stabilizes the flow pattern. Similarly to the surface velocity pattern on flat beds, the reverse flow trajectory is longer and straighter, and the upstream vortices have smaller longitudinal size than the observed ones. The differences between the measured and simulated surface velocity patterns and sizes of the upstream vortices are more prominent in cases T11 and T13. Figures 4.11 (b1) and 4.11 (b2) show that the upstream corner vortices cannot be reproduced by the numerical model for case T11, whereas their sizes are underestimated in case T13, as shown in Figures 4.11 (c1) and 4.11 (c2). Compared to the flow patterns in shallow reservoirs with flat beds, there is more discrepancy between the simulation results and the measurements. One possible reason for this discrepancy is the presence of various types of bed forms with different roughness values at different places on the bed floor. Another reason is the k- ϵ turbulence model, which represents a lower diffusion in simulation of the flow field in shallow reservoirs (Dewals et al., 2008).



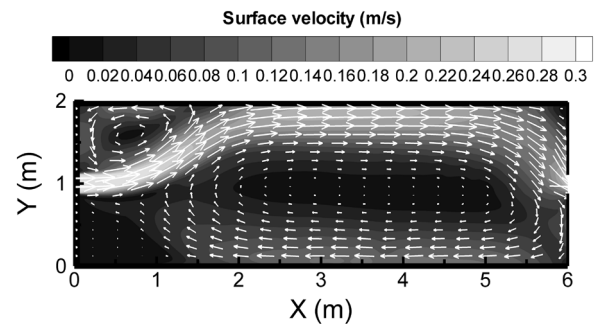


(c)

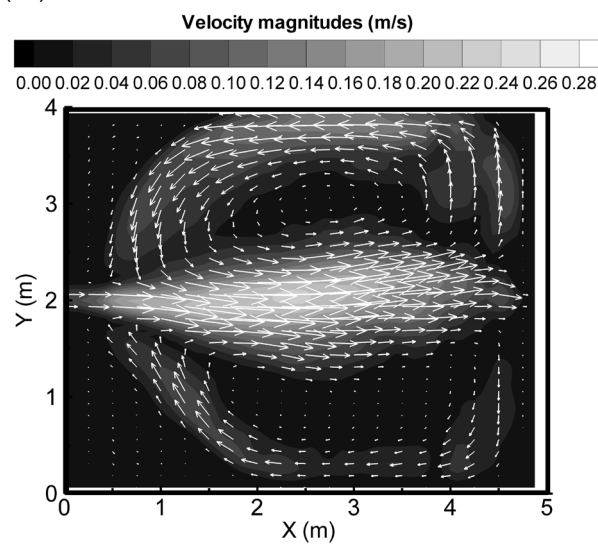
Figure 4.10 The misshaped bed obtained after sediment flushing with lowering the water level for case: (a) T8; (b) T11; (c) T13.



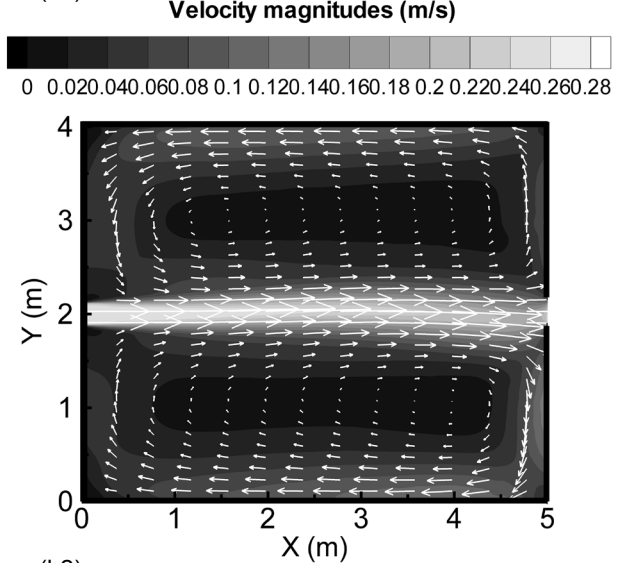
(a1)



(a2)



(b1)



(b2)

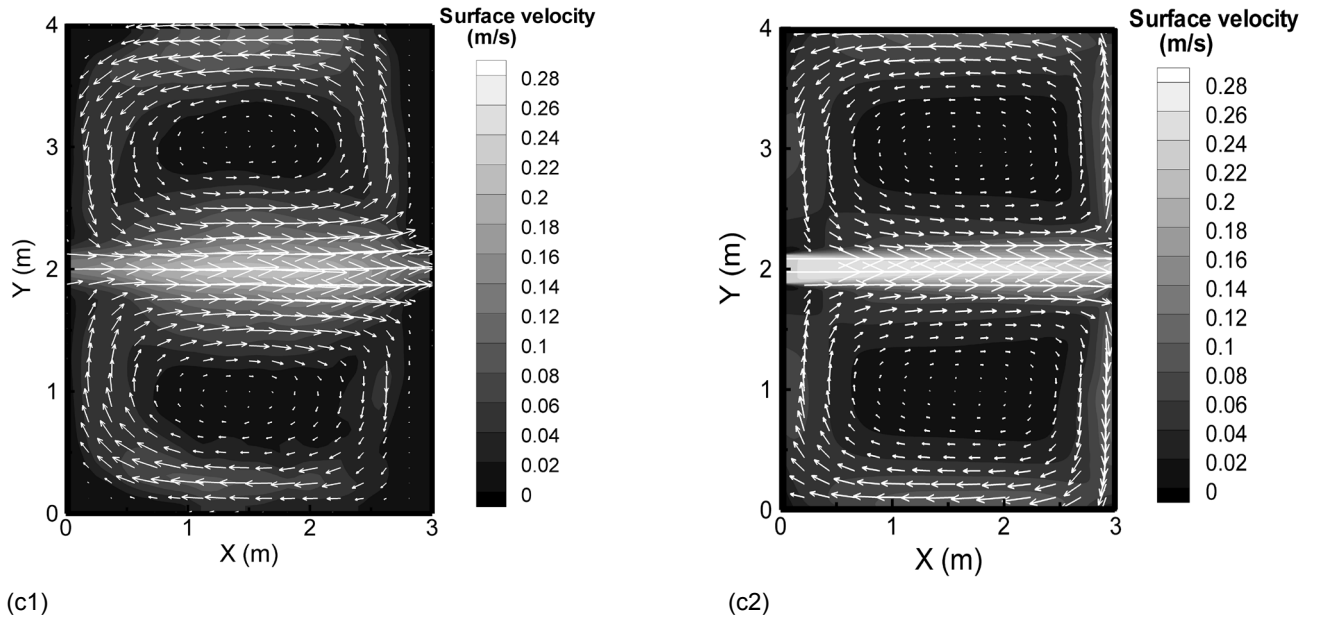


Figure 4.11 Left: The measured surface velocity field with vectors over misshaped bed after flushing for case: (a1) T8; (b1) T11; (c1) T13, respectively and right: corresponding simulated velocity field for each case.

The simulated streamwise and transversal surface velocities at the middle of the channel length were plotted against the measured ones in Figure 4.12 for two cases (i.e., T8, T13). Similar to the surface velocity field on flat beds, the numerical outputs are quantitatively in reasonable agreement with the measurements. Figure 4.13 also shows the simulated surface velocity versus the measured one in the upstream and downstream zones of case T11. This figure shows a small deviation of the main jet flow from the centreline of the reservoir, which first goes to the right side (Figure 4.13 (a)) and subsequently to the left-hand side (Figure 4.13 (b)). This deviation implies a non-straight forward flow motion along the centreline when the reservoir is wide.

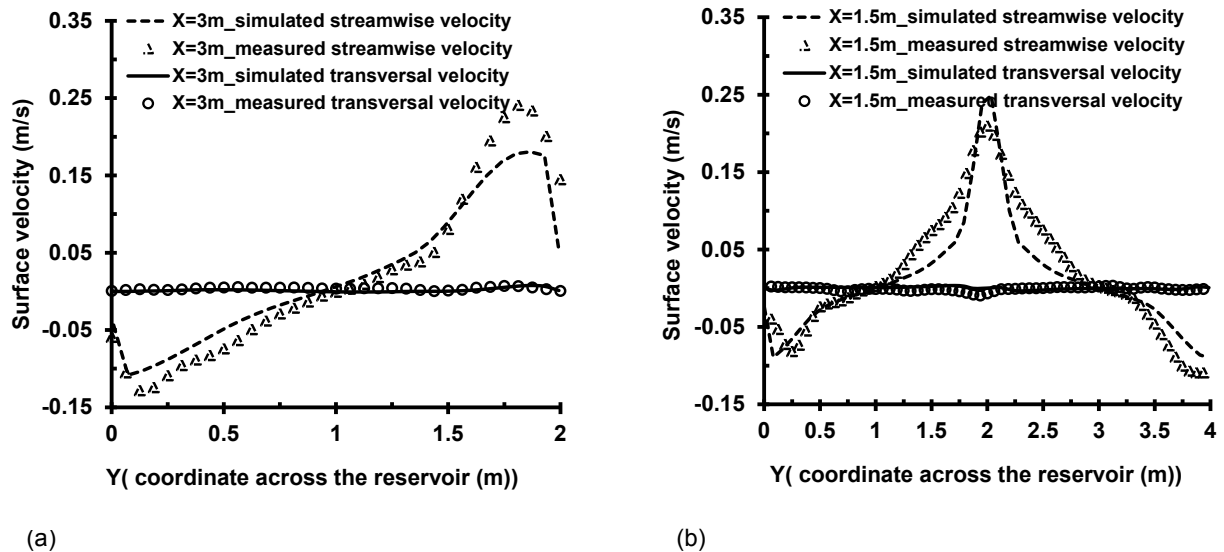


Figure 4.12 The measured streamwise and transversal surface velocity versus the simulated one at middle of the reservoirs length with misshaped bed condition for case: (a) T8; (b) T13.

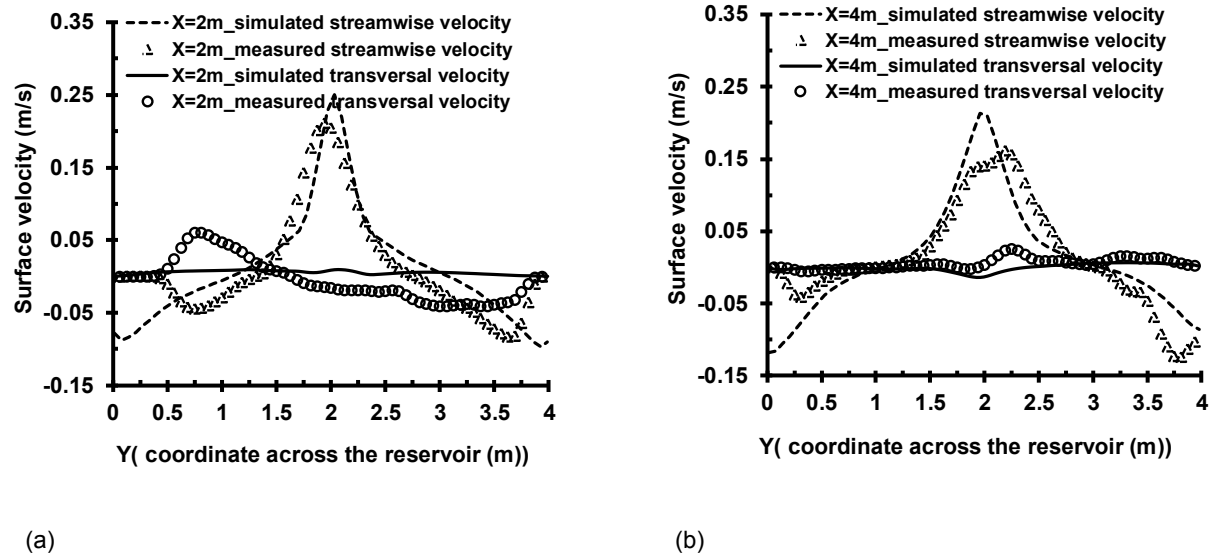


Figure 4.13 The measured and simulated surface velocity over the misshaped bed of case T11 at (a) $x=2$ m; (b) $x=4$ m.

4.5.2 Simulation of 3-D velocity variation over the flow depth

In case of shallow flows over flat beds, vertical velocities are small compared to the other components but it is not the case for shallow flows over misshaped beds where flow can be curved over the bed friction especially when bed forms (e.g., dunes and ripples) exist and 3-D features of flow appears (e.g., changing the velocity magnitude and direction over the flow depth in secondary flows domain, circulation zones in the vertical direction, and upwelling or downwelling fluid movements). As for the secondary flows, their governing zones may be small and local but their contribution on morphological process is significant and can further propagate especially in prototype reservoirs (e.g., sediment deposition in the inner part and erosion in the outer part of channel bending, and lateral development of the flushing channel in a reservoir during a free-flow flushing operation).

To gain more clarity regarding 3-D features of shallow flows over misshaped beds, the simulated lateral flow velocity contours over the depth beside the secondary flow velocity vectors at middle of the reservoir length are illustrated in Figure 4.14 (c) and 4.14 (d) for case T8 and the right half of case T13, respectively. These figures show the complexity of the 3-D flow field development on existing bed friction in shallow reservoirs with misshaped beds where the variation of velocity magnitudes and direction over the flow depth, and circulation zones in the vertical direction can be clearly seen. This type of outputs is beneficial for a precise analysis of flow characteristics when the bed has been disturbed and a complex bed geometry has been developed after the sediment-flushing operation (e.g., estimating the water levels in critical areas, and potential erosion and deposition zones during the anticipated floods).

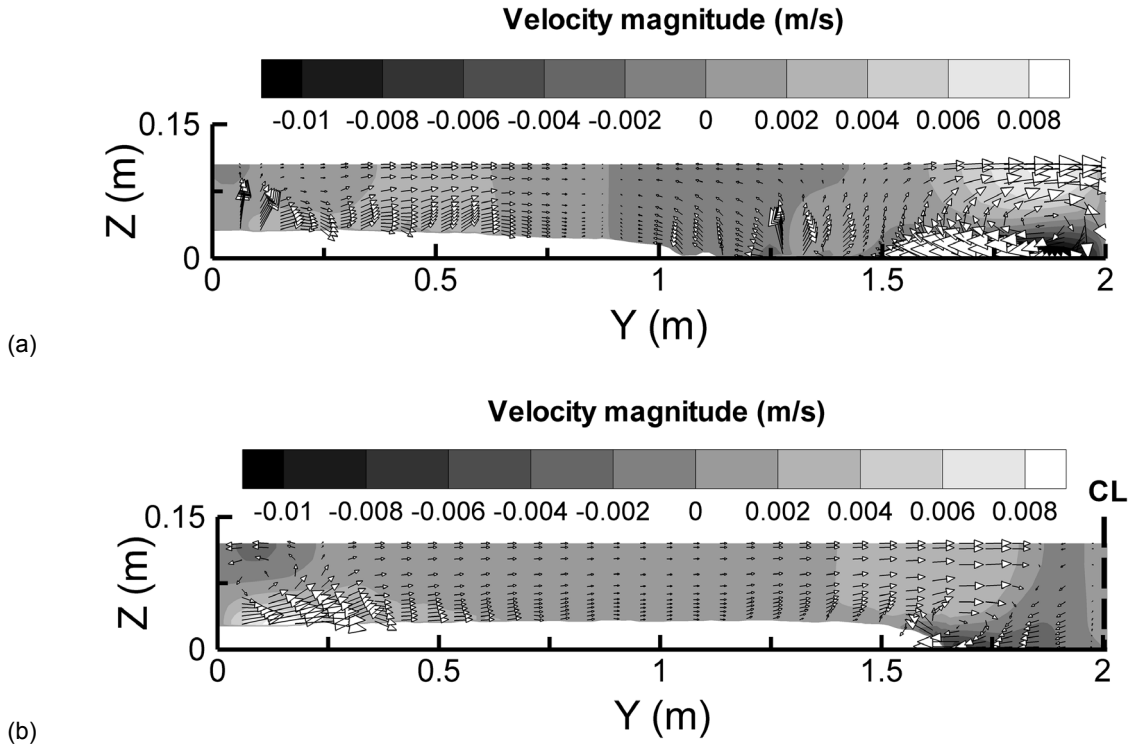


Figure 4.14 Simulated lateral velocity distribution over the flow depth at $x=3$ m on misshaped bed of (a) T8 and (b) right half of T13.

4.6 Sediment flushing simulation with water level drawdown

In order to control the reservoir sedimentation, different approaches such as bypassing, dredging, flushing, sluicing and upstream sediment trapping have been developed. Among several techniques, the flushing and sluicing plays an important role in the sediment removal and reduction, as they are efficient hydraulic sediment removal technique to restore the reservoir storage capacity (Morris & Fan, 1998 ; Liu et al., 2004). Sediment flushing in reservoirs involves several complex processes. During the flushing with drawdown, bottom outlets are opened to generate and accelerate unsteady flow towards the outlet. This process will initiate the progressive and retrogressive erosion pattern in tail and delta reaches of the reservoirs respectively (Batuca, 2000). However, detailed theoretical explanation of flushing channel formation in reservoir delta is scarce. For practical purposes, the pre-assessment of the flushing channel development characteristic using a 3-D numerical model would be beneficial to optimize the sediment flushing operation as well as the flood risk management in reservoirs.

Due to the importance of understanding the physical processes during the flushing channel formation and evolution, SSIIM 1 model was employed to simulate the flushing channel formation and evolution trend in a series of rectangular shallow reservoirs. Subsequently, this part focuses on figuring out the main physical processes of bed morphology changes during a flushing with lowering the water level. The outcomes of this part can help us to have a broader insight towards the sediment flushing simulation in real prototype scale reservoirs. This insight also would be useful for implementing some measures in reservoirs to enhance the flushing efficiency. Moreover, sediment concentration variation and its relation with the bed morphology changes were discussed since the concentration magnitude

can be an important environmental constraint for implementing an eco-friendly flushing in prototype reservoirs.

4.6.1 Longitudinal and lateral evolution pattern of flushing channel

Numerical outcomes as well as the observations in experimental runs revealed that if the water level is drawdown significantly, the flow starts to erode the bed progressively propagating to the downstream and retrogressively from the outlet towards upstream within the initial flushing channel formation. These main erosion trends have been illustrated schematically in Figure 4.15. The progressive trend was faster than retrogressive one. In the meantime, the initial flushing channel deepened and widened rapidly due to the strong jet flow and subsequent erosion.

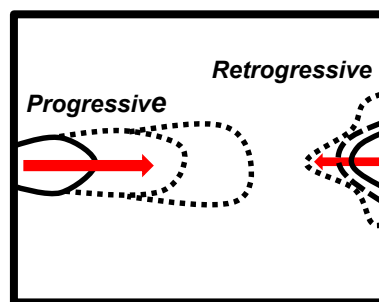


Figure 4.15 dominant erosion patterns within the initial flushing channel formation.

Then, after formation of initial flushing channel along the reservoir length, the rate of channel widening reduced noticeably until reaching a dynamic stable condition over the whole channel length. The mentioned process is very quick up to the slow widening stage so that the measurement of the bed evolution was difficult. Thus, just the final bed topography developed after the equilibrium stage was measured during the experiments. Briefly, flushing channel development characteristics can be described in three stages as :

- 1- Rapid widening and deepening stage within the flushing channel formation along the whole reservoir length.
- 2- Very slow widening stage after the flushing channel formation.
- 3- Dynamic stable condition over the flushing channel length.

In order to further quantitative assessment of the aforementioned processes, numerical results obtained from SSIIM 1 were compared with the experimental measurements in the equilibrium stage. Figure 4.16 demonstrates the longitudinal progressive and retrogressive pattern, reproduced numerically, along the centerline of case T8, T11 and T13 reservoirs. The side bank erosion and lateral development of flushing channel in the middle length of the mentioned reservoirs have also been shown in Figure 4.17.

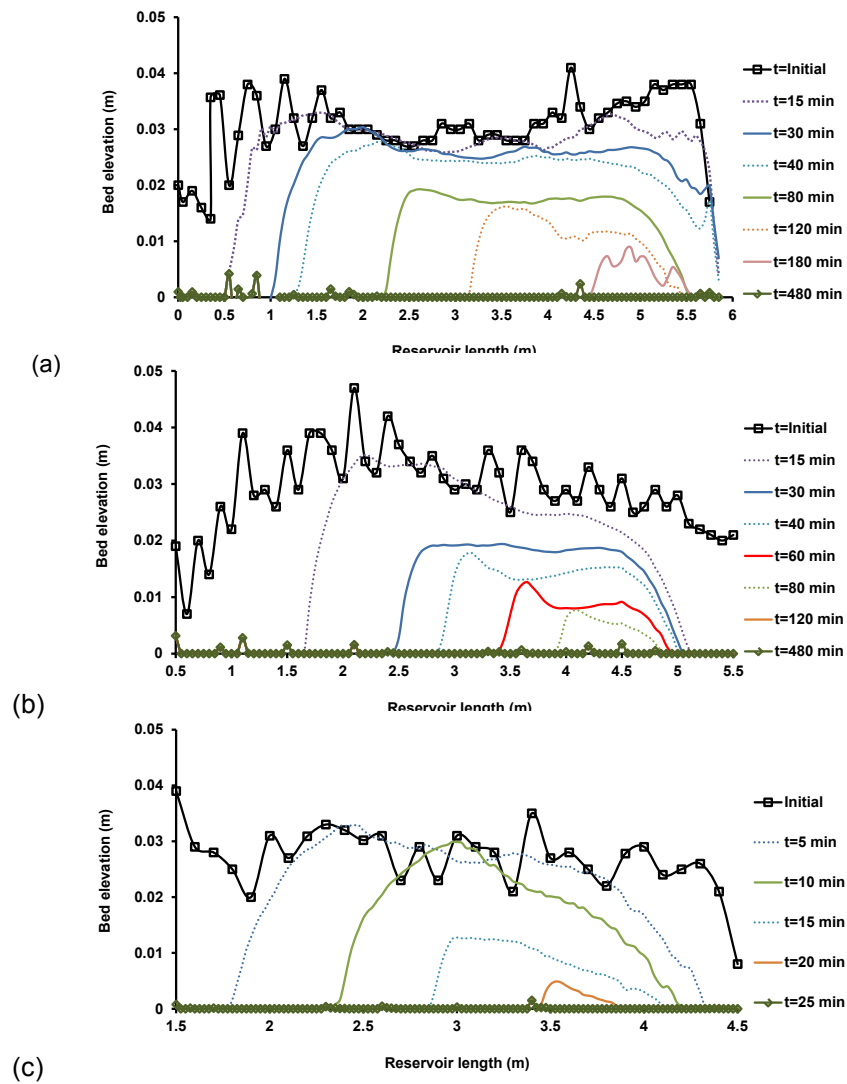
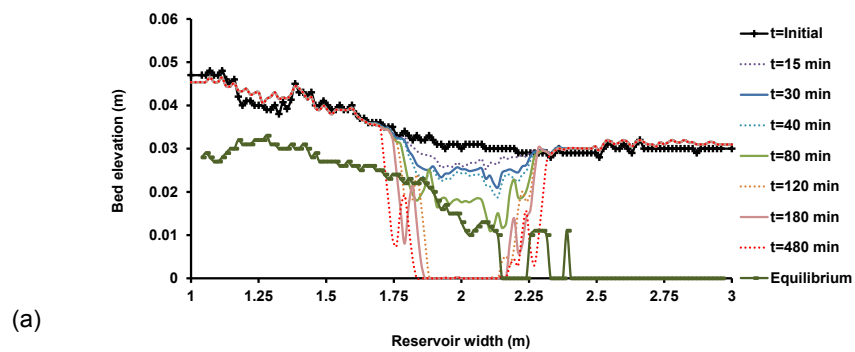


Figure 4.16 Longitudinal development of flushing channel along the centerline of case (a) T8, (b) T11 and (c) T13 simulated numerically.



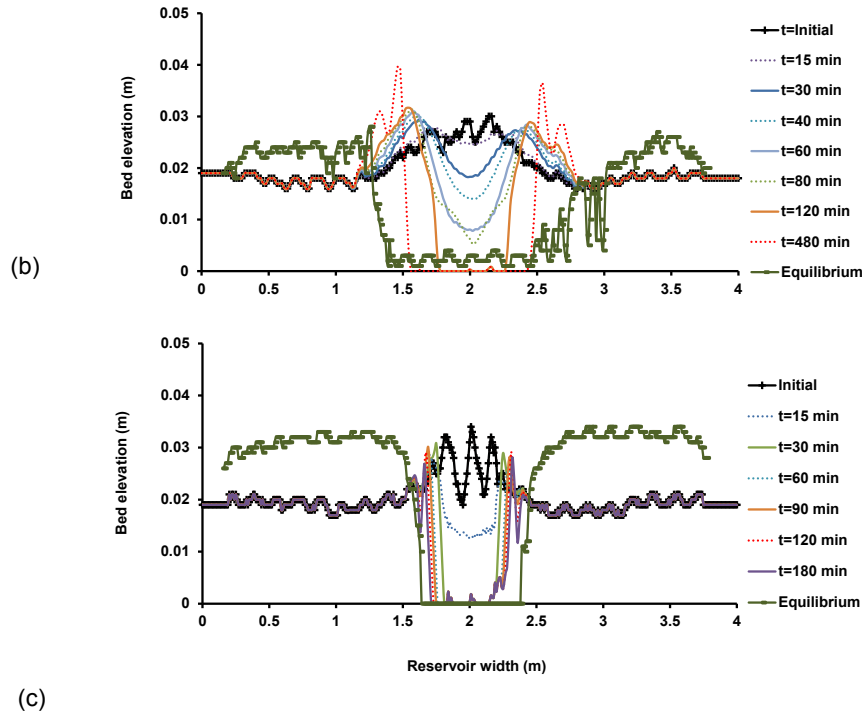
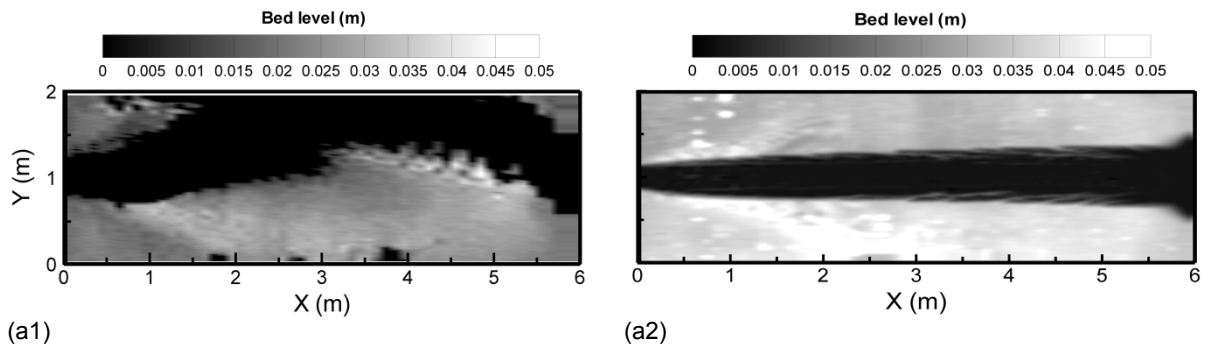


Figure 4.17 Bank erosion and lateral development of flushing channel for (a) T8, (b) T11 and (c) T13.

From the longitudinal point of view, Figure 4.16 apparently reveals that in reservoirs with shorter length (i.e., T13), the flushing channel forms faster than longer reservoirs (i.e., T11 and T13). Also, it can be observed that retrogressive erosion pattern is localized nearby the outlet and its development speed is much more lower than the progressive pattern. From the cross-sectional point of view, Figure 4.17 discloses that simulated flushing channel width is smaller than the measured one mainly because of a more concentrated jet flow as already mentioned in part 4.4.2. This matter may be attributed to application of standard $k-\epsilon$ turbulence model. Some modifications in empirical constants of standard $k-\epsilon$ turbulence model may increase the diffusion of the jet flow, which can contribute in improving the results of both flow and sediment transportation simulation. In addition, rapid widening and deepening process of channel in the early stage of flushing channel formation and also slow widening stage of the channel after formation of the flushing channel (i.e., stage 1 and 2 of flushing channel development characteristics) can be clearly seen in Figure 4.17.

Figure 4.18 illustrates the plan view of final measured bed level contours (Z) after 48 hours and simulated one after initiating the slow channel widening stage.



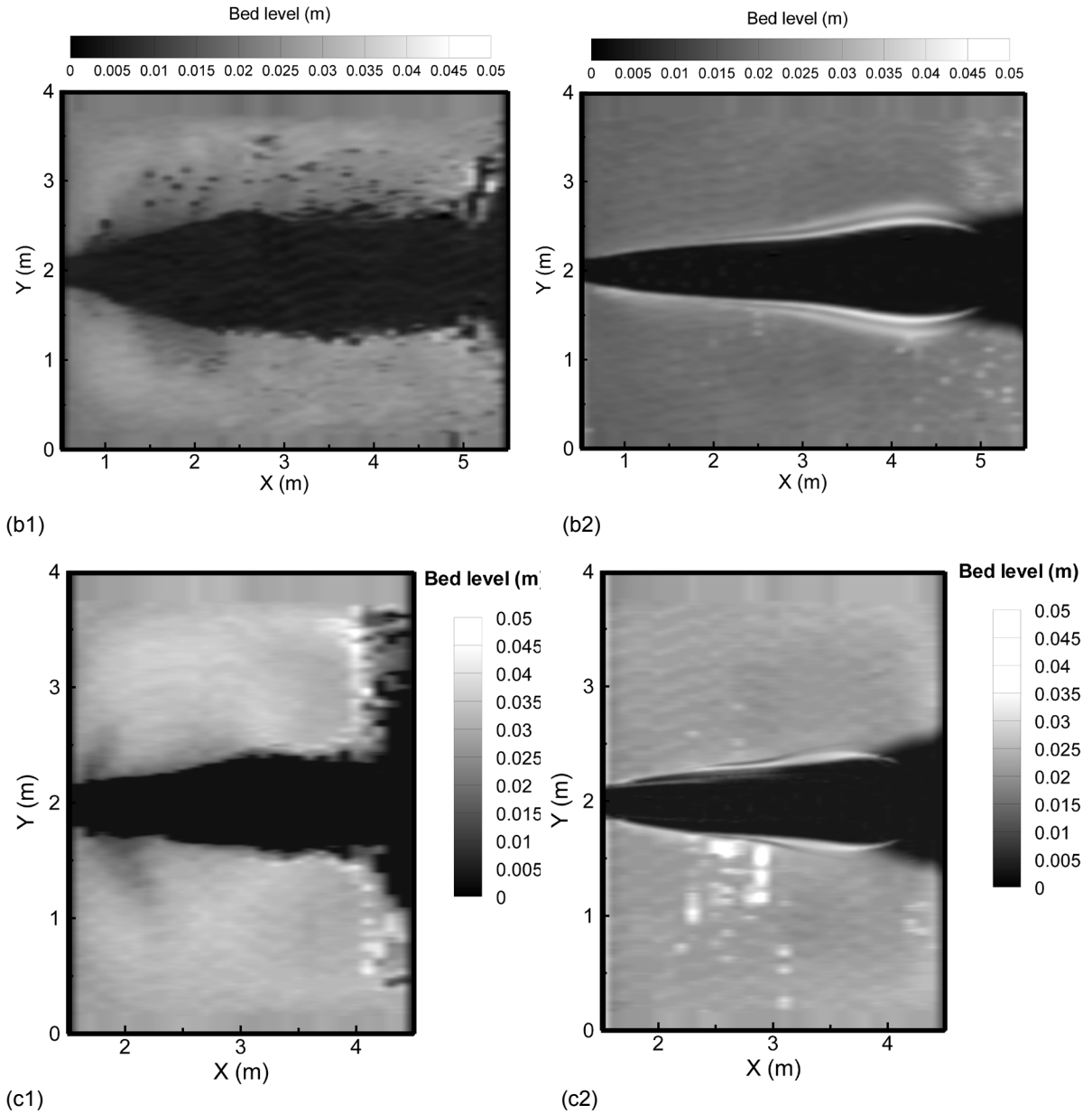


Figure 4.18 Left: The measured flushing channel shape and location for (a1) T8, (b1) T11 and (c1) T13 respectively and right: the corresponding simulated results.

The size, shape, location and evolution pattern of the flushing channel was simulated up to the early stage of slow widening phase. It takes long time to simulate whole 48 hours by 3-D numerical model. Thus, numerical computation was performed until starting the slow channel widening stage. Consequently, as soon as cumulative sediment pass variation within 30 minutes interval became smaller than 1%, simulations were stopped. As it can be seen from Figure 4.18, the characteristics of flushing channel components have been reproduced adequately using the numerical model except for case T8 in which the location of the channel was different than measured one. Although the geometry was symmetric, the channel did not develop along the shortest path from inlet to outlet during the experiment. The reason can be attributed to the accidental small disturbance in the inflow discharge distribution along the inlet channel width as discussed in part 4.4.1. In such a condition, location of the

flushing channel is very likely to change from the centerline of the reservoir. Since the numerical model assumes the uniform inflow discharge distribution and symmetric initial condition for the current cases, the flow direction would be straight and subsequently model is not able to break the symmetry of input data. As it was shown for case T11 in Figure 4.18 (b1) and 4.18 (b2), and also for case T13 in Figure 4.18 (c1) and 4.18 (c2), both experimental measurements and numerical outputs show that the channel width increased in the downstream direction similar to a T shape head. The longer simulation period, can represent a more similar channel width and also a T- head shaped channel close to the outlet.

4.6.2 Evaluation of geometry effect on sediment concentration variation and Flushing Efficiency (FE) :

Figure 4.19 illustrates the temporal variation of flushed out sediment discharge rates in milliliter per second along with the cumulative amount of sediment passed through the outlet for case T8, T11 and T13 using small time intervals (i.e., 5 minutes). This figure shows the sediment concentration variations with respect to the described flushing channel formation process in part 4.6.1. Besides, the total volume of flushed out sediment from the reservoir can be estimated using this figure. The Flushing Efficiency (FE) was also defined here as the volume ratio of flushed out sediment to cumulative deposited sediment after the deposition phase (i.e., second phase) :

$$FE = \frac{V_{out}}{V_{dep}} \quad (2)$$

where V_{out} is the volume of flushed out sediment during the flushing with lowering the water level and V_{dep} is the volume of already deposited sediment in the reservoirs in the second phase of experiments.

As it can be clearly seen from figure 4.19, the flushed out sediment discharge rate is very high and it is upward at early stages due to the initiation of rapid erosion process in the form of rapid widening and deepening of the flushing channel. Rapid erosion also occurs initially since the clear water has a high sediment transport capacity. The upward trend continues until reaching a peak value. Afterwards, the flushing channel starts to develop along the whole length of the reservoir and consequently water flow capacity to erode and transport additional sediment will decrease. In other words, the erosive forces will decrease when the flushing channel dimension develops longitudinally, vertically and laterally. Thus,

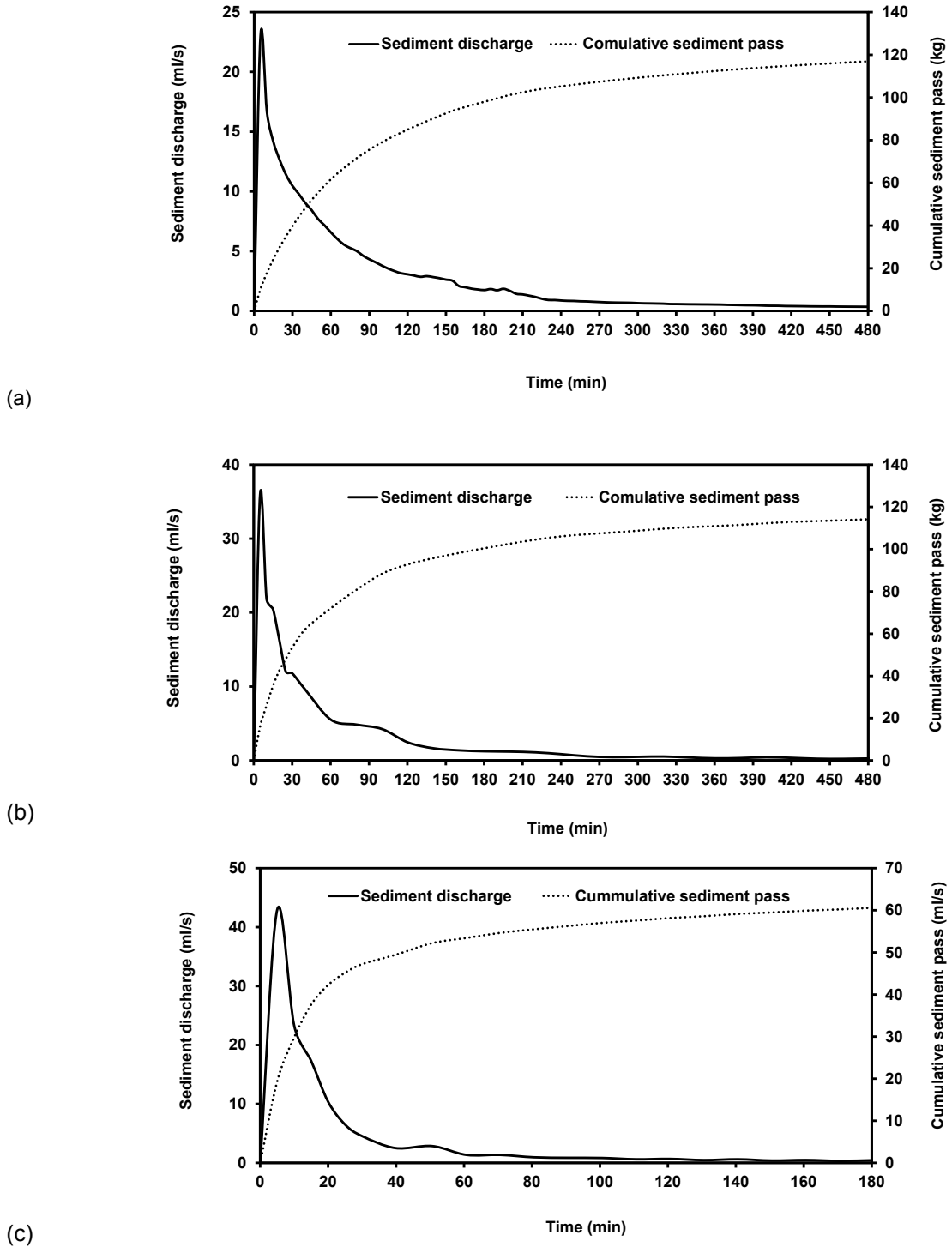


Figure 4.19 Temporal variation of flushed out sediment discharge rates and corresponding amount of commulative sediment passed through the outlet for case (a) T8, (b) T11 and (c) T13.

the sediment discharge rate decreases gradually. Finally, during the slow channel widening stage the sediment discharge rate remains almost unchanged with a low magnitude. Besides, it can be concluded that in s reservoirs with shorter length (i.e., T13), the flushing channel forms faster than other reservoirs and the dynamic stable condition is also acheived earlier. Of course the amount of flushed out sediment volume would be smaller in shorter reservoirs. For calculating the FE with respect to its definition, the final value of cummulated sediment passed through the outlet that was shown in Figure

4.19 was used. Due to the application of time intervals for calculating the sediment discharge variation while a significant sediment discharge fluctuations may appear within small time intervals, it is suggested to use the bed topography changes before and after the flushing operation to calculate a more accurate value of flushed out sediment amount in prototype cases.

The measured and calculated FE has been plotted versus the reservoir shape factor in Figure 4.20. In drawdown flushing, FE increases significantly in experimental runs when the reservoir shape factor increases. In other word, the flushing efficiency with drawdown will be high for narrow reservoir geometries. The numerical model outputs showed a similar trend, although calculated ones by numerical model represents slightly lower values than measured ones for Case T11 and T13. The main reason is due to the termination of computation as soon as slow widening stage starts. Nevertheless, in case T8, measured and numerically calculated FE has a large difference. This is because of the wide meandering flushing channel formation in the experiments (i.e., Figure 4.18 (a1)) whereas the width and length of the simulated straight flushing channel is smaller than the measured one (i.e., Figure 4.18 (a2)). Subsequently, the volume of the flushed out sediment as well as the flushing efficiency were higher in the experiments for case T8.

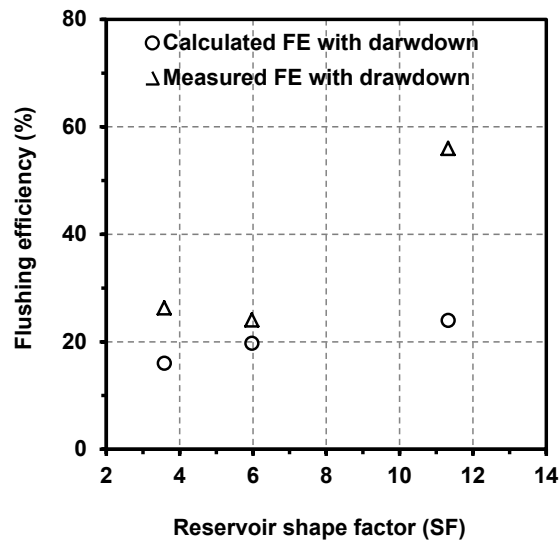


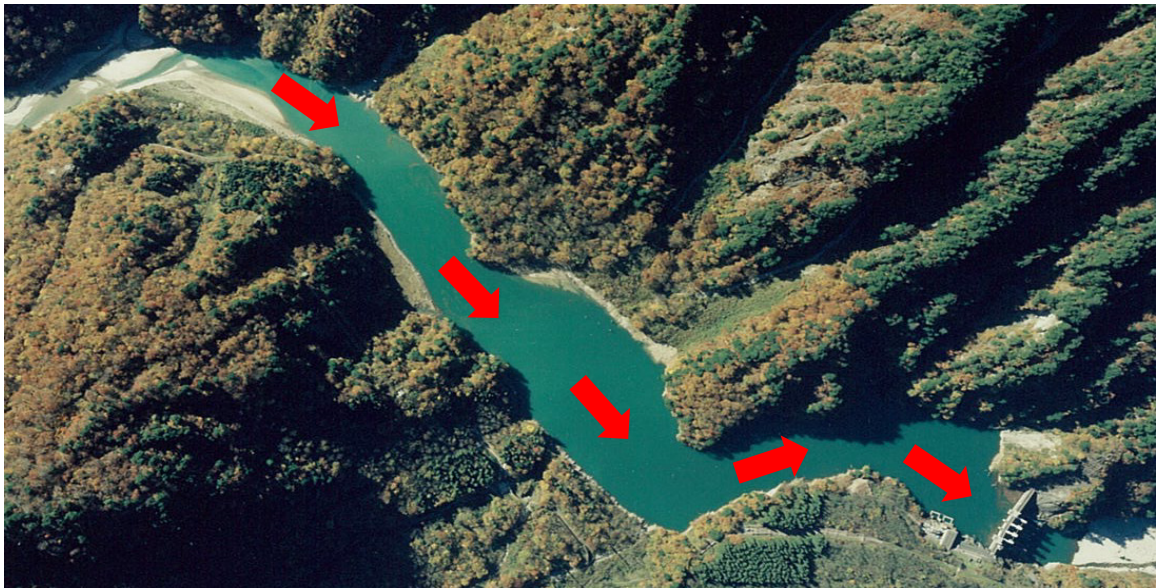
Figure 4.20 Measured and calculated FE for various reservoir geometries achieved after flushing with water level drawdown.

Chapter 5 3-D numerical modeling of sediment flushing from reservoirs in prototype scales: case study of Dashidaira and Unazuki

In Japan, generally the sediment yield by rivers is high due to the geologically young mountains, steep slope, and flashy flow regimes as well as wide spread landslides. Consequently, incoming sediment loads to the reservoirs fed by these rivers are high (e.g. Kurobe River located in Toyama prefecture). The loss of reservoir storage volume due to the sediment deposition reduces the dam's effective life time and also diminishes the reservoir function for flood control purpose, hydropower generation, irrigation and water supply which represents a substantial economic loss (Morris & Fan, 1998; Shen, 1999). Of all the existing measures, flushing plays a major role in reservoir storage capacity restoration since it is an efficient hydraulic technique for sediment removal (Lieu et al., 2004). During a flushing operation with full drawdown, the incoming flood erodes a flushing channel into the deposited sediment and flushes out both fine and coarse materials from the reservoir. The flushing channel evolution procedure is a complex phenomenon especially in steep reservoirs owing to the dynamic interaction between the unsteady flow field and bed variations. However, the sediment flushing in reservoirs located across the Kurobe River is a more challenging work because of the significant difference of grain size distribution in upstream and downstream areas of the reservoirs. While the upstream areas are covered with gravel and cobble sized bed materials, fine type sediments (e.g. small sand, silt and clay) have been widely distributed in the downstream areas. Loss of the stored water beside the lack of knowledge about the amount of the flushed out sediment, and location of the erosion-deposition areas in the reservoir are drawbacks for choosing the free-flow sediment flushing. Therefore, it is important to have an initial assessment of the impacts of an upcoming drawdown flushing operation in the reservoir.

The aerial view of the meandering reservoir of Dashidaira and Unazuki dam has been depicted in Figure 5.1. In this chapter, 3-D model SSIM 2 that applies the Finite Volume Method (FVM) in combination with a wetting/drying algorithm was employed and the bed evolution trend along with the surface velocity field was simulated in Dashidaira and Unazuki reservoirs for 2012 flushing operation. To do this, first, a sensitivity analysis of the Total Volume of Flushed out Sediment (i.e., TVFS) to main empirical parameters was performed for Dashidaira reservoir to figure out the appropriate ranges for the empirical parameters. Measured bed levels before and after the flushing operation were used for the model verification purpose. In addition, variation of flushed out sediment discharge in different

stages of flushing operation was assessed quantitatively for both mentioned reservoirs. More specific and detailed discussions about each category were provided in the relevant parts.



(a)



(b)

Figure 5.1 Aerial photo of (a) Dashidaira reservoir at almost maximum operational water level, and (b) Unazuki reservoir during the free-flow condition (photos from Kurobe River Work Office).

5.1 Model set-up procedure in SSIIM 2 for simulating the reservoir flushing

This part describes the necessary steps for preparing the SSIIM 2 program to perform the flow and sediment field computations. For more detailed technical descriptions and recommendations about

each step, it is suggested to refer to the User's manual prepared by Olsen (2014). This manual is available online and become updated periodically.

1- Input files preparation (Pre-processing)

There are different input files in SSIIM 2 program for different purposes. Here, the necessary ones, for model setup purpose, are highlighted and described. These input files are :

“geodata”, “timei”, “control” “koordina” and “fracres”.

geodata file contains the geometrical data in the form of x, y, and z coordinate of each measured point. The purpose of this file is to use the geometrical data obtained from the field measurements, digitized maps or GIS system.

timei file is used for introducing the time series of water level, water discharge and inflow sediment concentration. Using this file enables the user to introduce the flow hydrograph that enters into the computational grid. However, the format of input data arrangement and also application instruction for this file is different in SSIIM 1 and SSIIM 2.

control file also contains the main parameters used for computation such as downstream water level, water discharge, roughness value and etc. If the control file is not defined when the program starts, the program makes a default control file accordingly. Then this file can be stored and modified suitably. It is also possible to change some parameters in control file when the program is running. These changes may affect the accuracy and convergence of the numerical solution.

As for the koordina file, similar to the timei file, its application is different in SSIIM 1 and SSIIM 2. In SSIIM 2, this file is utilized for describing the corresponding water level in each point of the bed geometry using the structured format. Thus, it is possible to make an initial computational grid, first, with a simple assumption of a high and horizontal water level that covers the whole possible computational domain during the computation. In other words, a merely large water level should be employed that covers both wet and dry cells. This is a sensible method since some already dry areas are changed into wet areas when river/reservoir channel develops laterally or when the reservoir is refilled. Afterwards, koordina file can be read and used to start the simulation with a sloped water surface or with an actual downstream water level that is usually lower than the initial assumption. Consequently, after reading the koordina file the computational mesh is regenerated and includes a number of dry bed cells that can be wetted later (Olsen, 2014). The explained procedure for application of koordina file is strongly recommended for simulation of flushing channel formation and evolution in reservoirs and also is applied in this study.

fracres file is used for defining the grain size distribution in the river/reservoir bed cells in two layers (i.e, active and inactive). Both layers are assumed to exist below the computational domain. The upper active layer supplies the real computational domain with sediments when erosion occurs and it takes away the sediment when deposition happens. The height of the inactive layer is usually set as a high value to feed the active layer with sufficient amount of sediment. In Figure

5.2 the real computational domain where the water motion and sediment transport takes place, and also virtual active and inactive layers that located below the river bed has been depicted (Rüther & Olsen, 2007). In *fracres* file, similar to the *koordina* file with structured format, each grid intersection is described with an *i* and *j* number. Then, the amount of each sediment size fraction should be introduced in each bed cell corresponding to each intersection number. Usually, an accurate measurement of bed sediment size distribution in longitudinal and transversal directions, and especially below the river/reservoir bed is not available in the study zone (i.e., river/reservoir) while it can play a significant role in sediment transportation process. As a simplified assumption, sediment size distribution with spatial varying fractions can be achieved using the linear interpolation between the fractions size and amount of the available bed material samples.

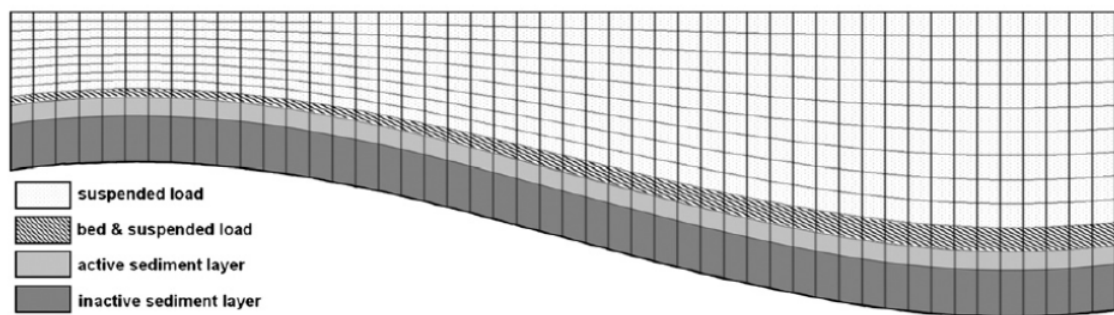


Figure 5.2 Schematic longitudinal section of computational domain used for simulating the water motion and sediment transport process in SSIIM 2 (Rüther & Olsen, 2007).

2- Grid generation

Grid generation is the first step in preparing input files for SSIIM 2 program. Thus, bed topography data is required for making the computational grid. The topographic data can be extracted through the bathymetry data of the river/reservoir bed. In SSIIM 2, the bathymetry data can be read directly by the input file called "geodata". Then, a mesh that covers the whole computational domain, with the appropriate number of grid lines, in X-Y plane is defined by the user in the graphical user interface of the program. The computational grid, then, can be constructed in the vertical direction between bed and water surface according to the *z* coordinates of bathymetry points by invoking the corresponding algorithm. Number of cells also may be different in different places of the computational grid due to the employed unstructured grid type in SSIIM 2. This is a robust approach since program can read many scattered points directly and the 3-D computational mesh is made automatically. Other alternative is transferring the bed topography grid from SSIIM 1 to SSIIM 2 and then generating the vertical grid lines, as before, if the computational mesh is already available in the structured format in X-Y plane. All the described procedure is done through the *GridEditor* option in *View* menu of the Graphical User Interface (GUI).

After constructing the grid, it should be stored in a file called "unstruc" file. This file contains all the geometry data such as coordinate of all grid line intersections or which cells are connected to other cells.

The information of inflow/outflow water (e.g., location of inflow and outflow and corresponding discharge magnitudes) should also be defined and stored in this file. For storing the developed computational grid, option *Write unstruc file* from *File* menu should be used.

3- Specifying the initial water discharges

After making the grid and storing it in *unstruc* file, the location of inflow and outflow along with the corresponding discharges should be specified graphically over the computational grid in SSIIM 2. To do so, the option *DischargeEditor* from *View* menu should be selected immediately after making the grid and storing it in *unstruc* file as described before. Otherwise, the *unstruc* file should be invoked from *File* menu using *Read unstruc* file. Then, the type of discharge in each place (e.g., inflow or outflow) and the corresponding discharge magnitude should be defined in the appeared user interface. It is also possible to have different places for inflow and outflow discharges in the computational grid. However, sum of the inflowing discharges should be equal to the sum of outflowing discharges (Olsen, 2014).

4-Utilizing the GUI (Post-processing)

Monitoring and evaluating the results is fall into the post-processing stage. Both versions of SSIIM program has a graphical module. User can select different options from a list of variables directly from *Sediment variable* or *Variable menu*. This is the easiest way to see the results within the calculation or at the end of calculation. In both versions of SSIIM, the grid can be seen from the plan view but the longitudinal and cross-sectional profiles just can be seen directly in SSIIM 1 since the structured grid is utilized.

In SSIIM 2, during a time dependent sediment calculation, a time interval can be set in the program to produce a type of output file called “bedres”. This file contains the outputs regrding the bed level variation in each determined time interval. In many cases, it is necessary to present the results for analyzing purpose in a more advanced level with a higher quality of graphics. This can be done in SSIIM program with other postprocessing programs such as *Tecplot* or *Paraview*. Input files for both mentioned programs can be written manually from the GUI or automatically by setting the appropriate parameters in control file (Olsen, 2014).

5.2 Required data for model set-up and availability

In order to make the input files for Dashidaira and Unazuki reservoirs model (i.e., model building) and perform the simulation for calibration and verification purpose, some specific data should be used. In other words, data will be used for both model building and verification purpose.

The necessary data, can be divided into three main cathegories : Bed geometry data within the reservoir, hydrodynamic boundary conditions, and sediment boundary conditions. Geometrical data consists of bed topography data before the flushing operation for model building and after the flushing operation for model calibration and verification. The bed topography data can be obtained through the measured cross-sectional bed profiles. The hydrodynamic boundary condition includes the the entering discharge (i.e., inflow) into the reservoir from the upstream boundary and water level changes during

the flushing operation which are introduced to the model for model building and used during the sediment transport simulation process for model calibration purpose. The sediment boundary condition includes the data about spatial variation of bed materials size distribution for model building, and also includes data about the concentration of different sediment size fractions enters into the reservoirs. The latter one is employed for simulating the sediment transportation process during a flushing operation.

In the consequent parts, the availability of the mentioned data regarding Dashidaira and Unazuki reservoirs will be discussed.

5.2.1 Bed geometry data

In both Dashidaira and Unazuki reservoirs, a regular bed profile measurement is conducted before and after the flushing operation by Dam owners. Figure 5.3 shows the bed topography that has been extracted from the measured bed profiles before and after the flushing operation in June 2012 at Dashidaira and Unazuki reservoirs. To have a more accurate insight on the bed topography variations due to

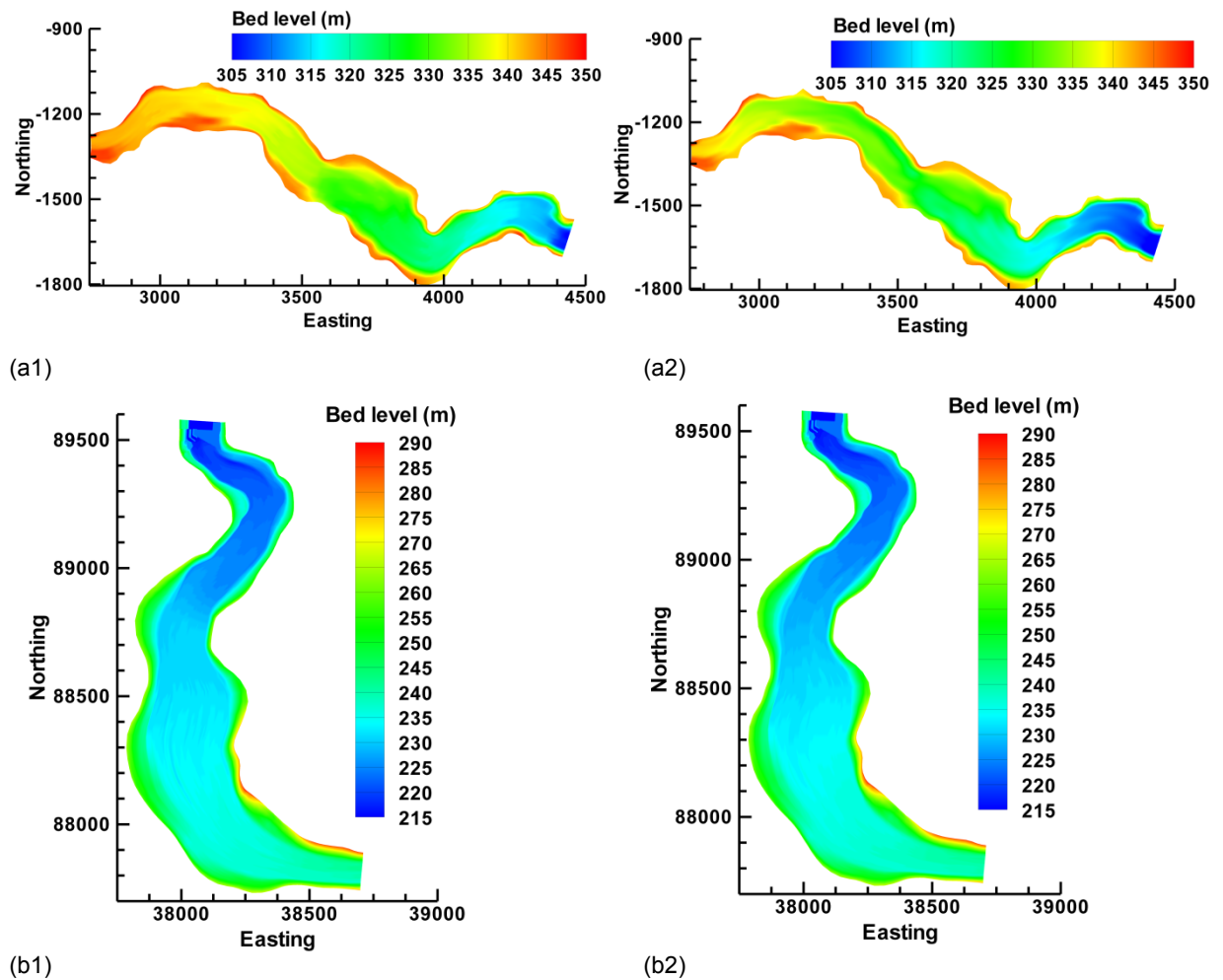


Figure 5.3 Measured bed topography at Dashidaira reservoir (a1) before and (a2) after the flushing operation; Measured bed topography at Unazuki reservoir (b1) before and (b2) after the flushing operation. Illustrated measurements correspond to the free-flow flushing event conducted in June 2012.

the flushing operation, bed changes which was extracted by comparing the measured bed levels before and after the flushing are depicted in Figure 5.4. As can be observed, there is no remarkable erosion along the left bank of wide middle area in Dashidaira reservoir (i.e., dead zone) and also from the upstream area of Unazuki reservoir. Increasing the bed material erosion from these areas would be beneficial to enhance the flushing efficiency and thereby to keep a bigger reservoir capacity.

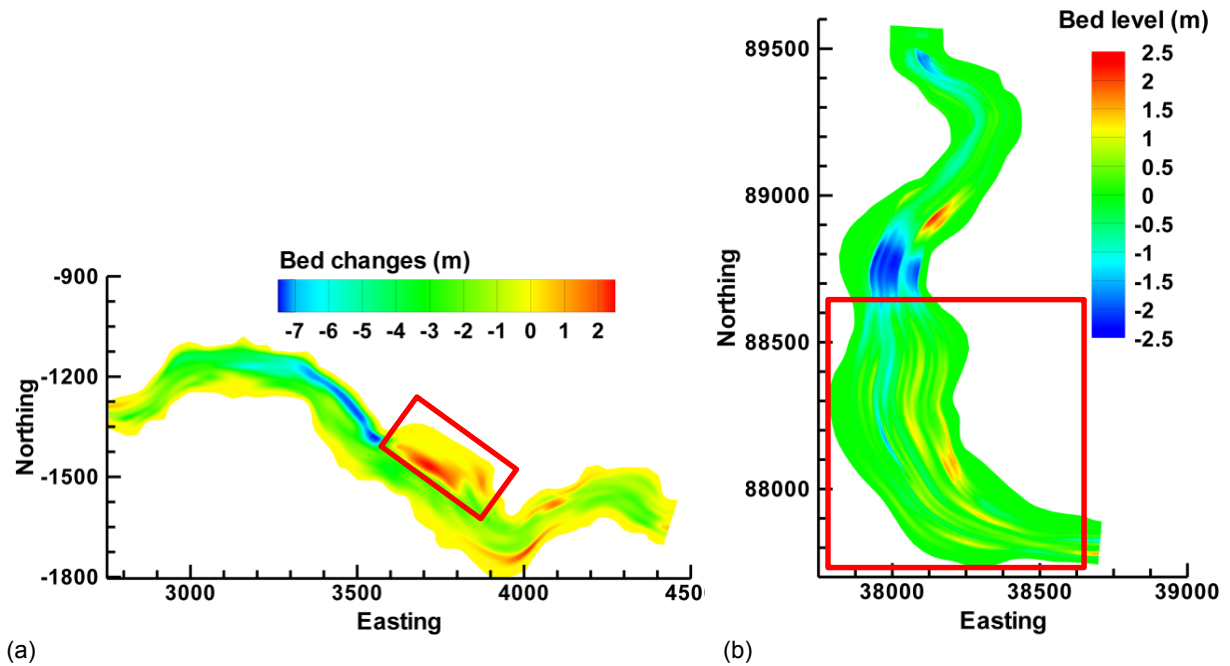


Figure 5.4 Bed changes over the whole reservoir after the flushing operation in (a) Dashidaira reservoir ; (b) Unazuki reservoir. Negative contours represent the erosion areas and positive contours represents the deposition areas ; Red lined rectangles shows segments of the reservoirs without remarkable erosion during the flushing.

Comparing Figure 5.3(a1) and Figure 5.3(a2) reveals that bed erosion occurs over the whole reservoir length when the flushing channel develops. However, the flushing channel can not be propagated considerably in far upstream areas. Reason can be attributed to the type of the existing bed material that are mainly coarse and are difficult to be eroded. The bed material distribution in Dashidaira and Unazuki reservoirs are discussed in part 5.2.3. As can be also seen from Figure 5.3(b1) and Figure 5.3(b2), the bed erosion caused by the flushing operation is not noticeable in Unazuki reservoir. This is more highlighted in the upstream half of the reservoir where is covered with coarser materials. Using the bed topography data before and after the flushing operation enables us to estimate the Total Volume of Flushed out Sediment from a reservoir that is called TVFS hereafter. The measured TVFS in Dashidaira and Unazuki reservoirs were 409000 m³ and 56000 m³, respectively. The measured TVFS in Unazuki reservoir is small compared to TVFS in Dashidaira reservoir since Unazuki reservoir has been built recently and bed condition is just shifting to the equilibrium condition. This means that during a coordinated sediment flushing operation, when the sediment load enters into Unazuki reservoir from Dashidaira reservoir and natural run-off, sediment deposition may occur instead of sediment erosion and sediment flush out from the reservoir. This may result in decreasing the storage capacity in Unazuki reservoir instead of increasing the capacity after a coordinated sediment flushing operation. In case of Dashidaira reservoir, bed is almost reached to the equilibrium condition. Consequently, a

significant amount of deposited sediment would be flushed out from Dashidaira reservoir after each flushing operation.

As can be observed from Figure 5.4(a), deposited materials along the left bank of the wide middle area shown with a red line rectangle, cannot be removed effectively in Dashidaira reservoir. Information about the flushing channel shape in this area can be useful for implementing the proper measures to enhance the sediment erosion in the mentioned dead zone and increase the flushing efficiency. In case of Unazuki reservoir, deposited materials in upstream area of Unazuki reservoir shown with the red rectangle in Figure 5.4(b) also can not be flushed out effectively. Using appropriate measures may contribute in increasing the bed material erosion from the highlighted area of the Unazuki reservoir. It should be taken into account that bed material type in the dead zone of Dashidaira reservoir is mainly fine whereas bed material type in the highlighted area of the Unazuki reservoir is coarse.

5.2.2 Hydrodynamic boundary conditions

The water levels at dam sites as well as the Inflow and outflow discharge are recorded during the free-flow sediment flushing operation with 60 minutes intervals. This information, then, utilized as the hydrodynamic boundary condition to introduce the water level drawdown procedure and discharge variations to numerical model during the simulation of a free-flow flushing operation. Figure 5.5 shows the water level and discharge rates, in both Dashidaira and Unazuki reservoirs, during the free-flow sediment flushing operation event in June 2012.

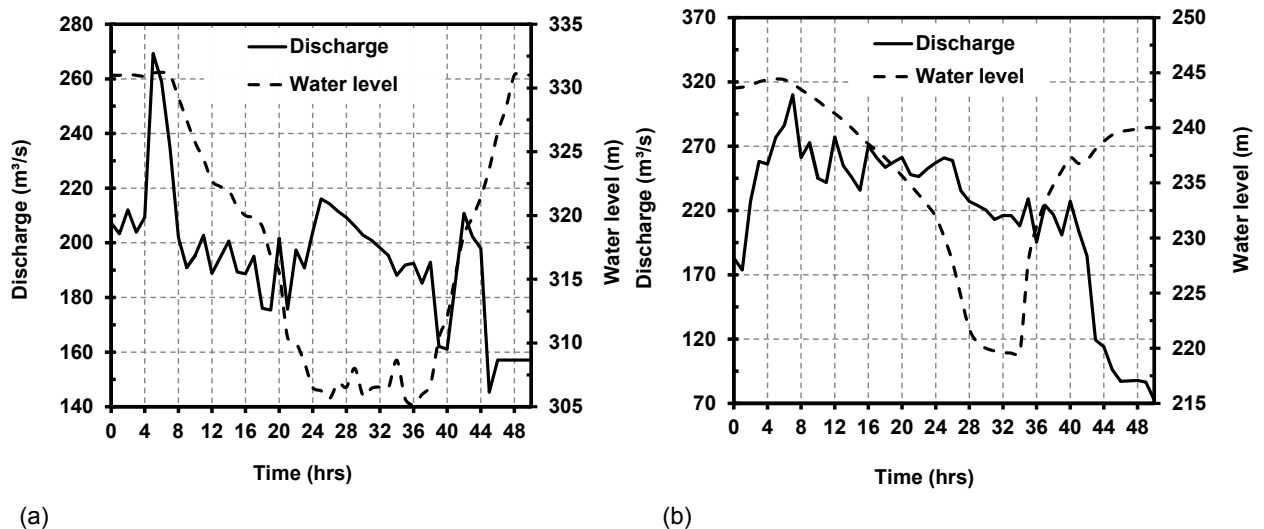


Figure 5.5 Water level and discharge rates during the flushing operation performed in June 2012 at (a) Dashidaira reservoir; (b) Unazuki reservoir.

In Dashidaira reservoir, preliminary drawdown was from 8 to 21 hours and free-flow state was from 21 to 38 hours after starting the flushing operation. Preliminary drawdown and free-flow state was from 8 to 27 and 27 to 34 hours after starting the flushing operation in Unazuki reservoir.

5.2.3 Sediment boundary conditions

Seven sediment sizes, ranged between 316 μm and 0.37 mm were considered as the representative bed grain sizes for both Dashidaira and Unazuki reservoirs. A non-uniform bed material size distribution with spatial varying fractions was introduced to the model using the seven representative sediment sizes. More specifically, the computational domain was divided into a number of small segments and each segment has its own non-uniform grain size distribution over the corresponding computational grid of the segment. In other words, in almost all grid nodes of each segment, a similar non-uniform sediment size distribution has been introduced. Since it is almost impossible to provide many onsite bed samples, with a high resolution in lateral and longitudinal directions, the employed approach here can be a reasonable assumption to represent a non-uniform sediment size distribution with spatial varying fractions over a whole reservoir area.

The spatial variation of the sediment fractions within the reservoir area was obtained based on the size interpolation of the available bed material samples. The number of bed material samples was bigger in Dashidaira reservoir compared to Unazuki reservoir. Thus, the number of segments with same non-uniform sediment size distribution was 27 in Dashidaira reservoir model while it was 12 in case of Unazuki reservoir model. Subsequently, a more accurate sediment size distribution can be made in Dashidaira reservoir. Figure 5.6 illustrates the aforementioned segments in each reservoir. Table 5.1 and 5.2 shows the non-uniform sediment size distribution before the flushing operation in different segments of Dashidaira and Unazuki reservoirs, respectively.

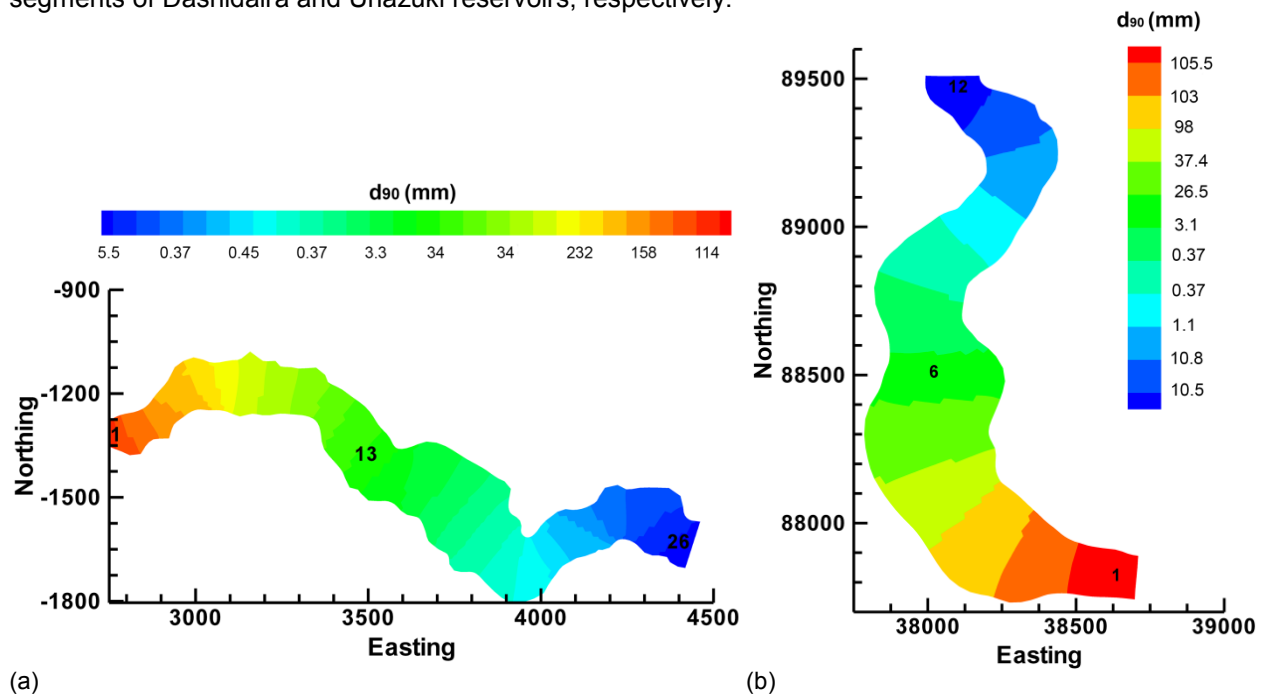


Figure 5.6 Schematic illustration of segments with identical non-uniform sediment size distribution at (a) Dashidaira reservoir; (b) Unazuki reservoir; Tags on the figures show the corresponding segment numbers.

Figure 5.5 reveals the expected non-uniform sediment size distribution in both reservoirs that is consistent with the deltaic deposition process. Beside the information about non-uniform bed material distribution over the 2-D reservoir plane, information regarding the vertical distribution of bed material sizes can represent a more realistic condition. Picking more onsite bed sediment samples including the vertical profile of sediment size distribution, before the flushing operation, can decrease the existing uncertainty about the sediment boundary condition and of course implies more efforts and costs.

Sediment size (mm)	1	3	5	7	9	11	13	15	17	19	21	23	25	26
(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)
316	8	12.54	2	0	0	0	0	0	0	0	0	0	0	0
118.3	36	60.3	74	2.52	0	0	0	0	0	0	0	0	0	0
37.4	29	9.05	6	76.26	75.66	71.25	29.87	1.47	0	0	0	0	0	1
11.8	23	5.03	4	6.57	5.82	7.5	24.68	5.15	0	0	0	0	0	11.5
3.7	1	5.03	3	7.58	10.58	13.75	24.68	17.65	0	0	0	0	0	16
1.2	1	3.02	5	2.02	1.06	2.5	15.58	20.59	0	13.33	11.11	0	0	20
0.37	2	5.03	6	5.05	6.88	5	5.19	55.15	100	86.67	88.89	100	100	51.5

Table 5.1 Average sediment size distribution in selected segments of Dashidaira reservoir shown in Figure 5.6 (a).

Sediment size (mm)	1	2	3	4	5	6	7	8	9	10	11	12
(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)
316	0	0	0	0	0	0	0	0	0	0	0	0
118.3	73	63	53	40	10	1	0	0	0	0	0	0
37.4	16	21	26	36	38	21	1	0	0	4	9	9
11.8	8	10	9	10	20	23	5	0	0	3	8	7
3.7	1	1	3	8	12	22	16	1	0	3	6	7
1.2	0	0	0	0	4	10	29	5	0	8	6	6
0.37	2	5	9	6	16	23	49	94	100	82	71	71

Table 5.2 Average sediment size distribution in existing segments of Unazuki reservoir shown in Figure 5.6 (b).

Table 5.1 and 5.2 both reveal the variation of bed material distribution pattern in Dashidaira and Unazuki reservoirs. While the bed material sizes are coarser in upstream segments of the reservoirs, their sizes become finer in downstream segments as their locations get closer to the dam site. However, some coarser bed materials can be found in areas nearby both dams. These coarser materials could not be released from the reservoir within last flushing operations.

The mentioned remarkable change in sediment sizes is a challenging condition for model to accurately simulate the sediment transportation process during the flushing operation and an intensive work would be required for model calibration when such kind of spatial variation of bed material size exists.

As for the concentration of different sediment size fractions enters into the reservoir, it should be noted that major sediment inflow for Dashidaira reservoir was wash load. Since the wash load basically assumed to be transported without deposition in the reservoir, its effect on simulation of the flushing process was neglected. For Unazuki reservoir, major part of released bed materials from Dashidaira reservoir enters the Unazuki reservoir and then a portion of these materials would be passed through

the Unazuki reservoir and flushed out from the reservoir. Subsequently, concentration of different sediment size fractions that enters the Unazuki reservoir should be introduced. Since direct measurement for concentration of different sediment sizes entering the Unazuki reservoir is not available, other alternatives for estimating the entering sediment concentration should be considered. To do this, the available time series estimation of the sediment influx discharge at the reservoir entrance that has been extracted using the equilibrium sand feeding equation was employed. This data then was converted to the volumetric sediment concentration for each sediment size fraction to use as the sediment influx boundary condition for simulations.

5.3 Model set-up for simulation of flushing in Dashidaira reservoir

The computational grid was made based on the measured bed levels before the flushing presented in Figure 5.3 (a1) and it has been depicted in Figure 5.7. The mesh cell size in streamwise and transversal direction was 10-20 (m) and 5-10 (m) respectively. The bed material density was assumed to be 2650 (kg/m³). Fig 5.7 shows the initial computational grid adjustment before starting the drawdown stage. The water levels and inflow discharge fluctuations shown in Figure 5.5 (a) were employed as the hydrodynamic boundary conditions in the simulations. In addition, a non-uniform bed material size distribution with spatial varying fractions, as described in part 5.2.3, and Table 5.1 was used to make the initial bed material distribution in the reservoir area.

Figure 5.8 illustrates the measured bed levels after the flushing operation in June 2012 along with the location of cross-sections (A-L) for a further assessment of bed variations during the flushing simulation. Almost 2 km length of the study case has been divided into three areas, namely, area I, II and III to analyze the study outcomes based on the type of the bed material exists in each area. In area I, bed material type is coarse while in area III and the latter half of area II, it changes into fine type of sediments as it was shown in Table 5.1.

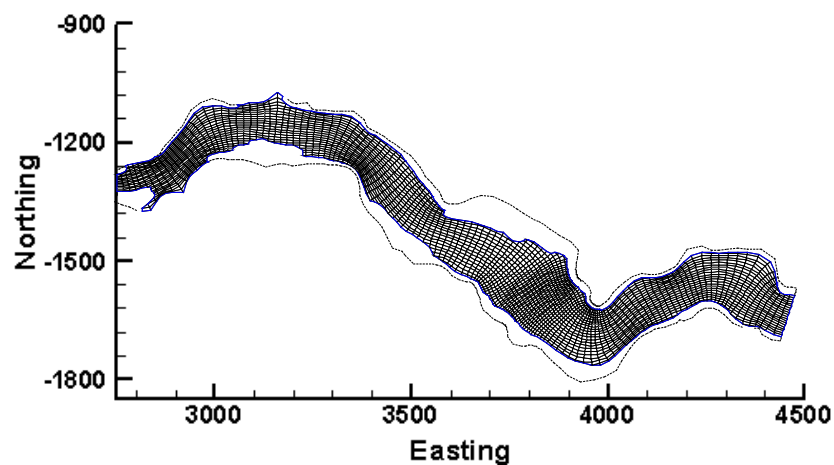


Figure 5.7 Computational grid for Dashidaira reservoir at the beginning of simulation ($t=0$).

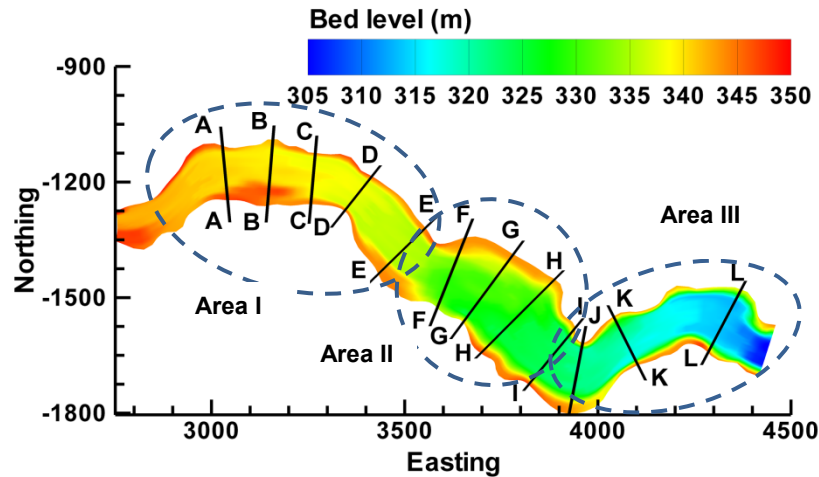


Figure 5.8 Cross-sections A-L for bed evolution assessment. Area I, II and III are zones with different bed material type.

5.3.1 Sensitivity analysis and model calibration

For calibration purpose, first, a reference case was established with assuming general values for the empirical parameters (e.g. roughness, active layer thickness, water content of the bed material and critical angle of repose). Then, sensitivity of the computed Total Volume of Flushed out Sediment from the reservoir (that is called TVFS hereafter) to the mentioned empirical parameters was investigated. The computed TVFS was compared with the measured TVFS (i.e. 409000 m³) and if it is greater than 70% of the measured one, then, it was considered as a model case for the further assessment. Afterwards, the final simulated bed topography pattern in the candidate case was compared both qualitatively and quantitatively with the measurements (Esmaeili et al., 2015a). Qualitative assessment is performed to check whether the erosion in area I and specially the lower part of this area (i.e. section E-E) could be simulated or not and quantitative assessment is conducted to measure the deviation of simulated bed levels from the measured one.

Table 5.3 shows the sensitivity analysis of TVFS to the major selected empirical parameters compared to the reference case. In addition, Root Mean Squared Error (RMSE) and Mean Absolute Error (MAE) of simulated bed levels after the flushing operation was calculated to quantitatively assess the deviation of simulated bed levels from the measured ones. As can be seen, TVFS was enhanced with increasing the active layer thickness and the water content whereas it was reduced with increasing the critical angle of repose. When a larger amount of bed material can be eroded in one time step (i.e. thicker active layer), a higher volume of erosion from deposits is expected. Assuming a higher water content in the sediment depositions (e.g. 50%), which decreases the submerged density of the bed material, leads to a higher sediment entrainment. A higher critical angle of repose keeps a steeper side bank of the flushing channel after each time step that results in a further deepening of the channel which is not an efficient approach for increasing the TVFS. In contrast, a lateral development of the flushing channel (i.e., channel widening) that is favorable for increasing the TVFS, can be achieved by a lower critical angle of repose. The effect of angle of repose has been depicted on lateral development pattern of flushing channel in Fig 5.9 (Esmaeili et al., 2015a).

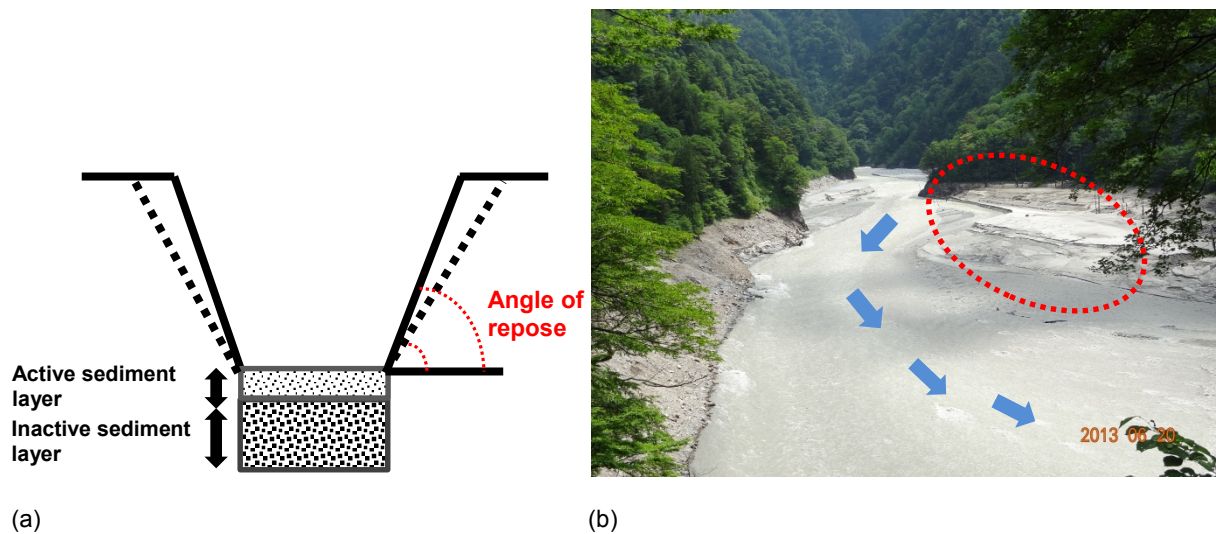


Figure 5.9 (a) Schematic illustration of the angle of repose effect on bank erosion and lateral development of flushing channel ; (b) bank failure and consequent channel widening during the free-flow flushing operation in Dashidaira reservoir (Photo taken by author).

Due to a better performance in qualitative assesment in prediction of TVFS, Meyer-Peter-Müller sediment transport formula (i.e., called MPM formula hereafter) was selected for the reference case and subsequently for calibration purpose. The roughness, active layer thickness, water content of the bed material, and critical angle of repose was finally set to 0.5 m, 1 m, 40% and 33 degrees respectively through the calibration process in the refernce case because these values can result in a reasonable TVFS and also a better accuracy based on the RMSE and MAE assessment.

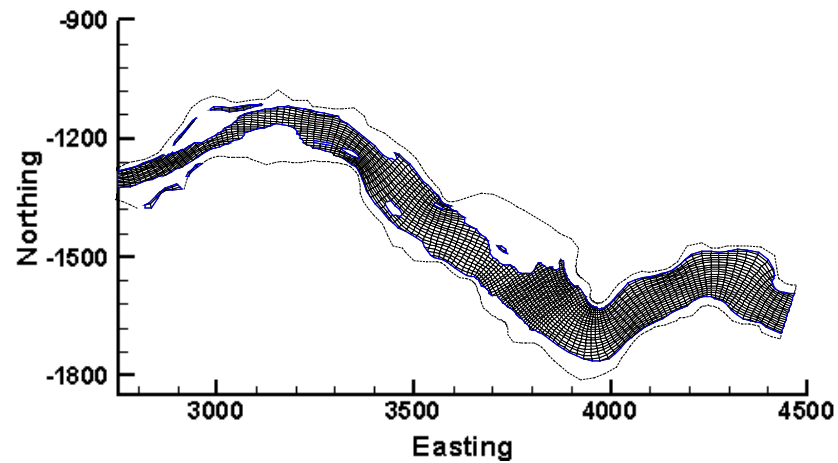
Parameter	Active layer thickness (m)			Water content of the bed material			Critical angle of repose (degree)		
	0.3	0.45	0.85	0.50	0.43	0.40	33	34	35
TVFS (KCM)	261	290	300	369	306	313	311	302	284
RMSE (m)	3.03	2.5	2.76	3.26	3.06	2.5	2.78	2.17	2.93
MAE (m)	2.17	1.73	1.95	2.25	2.10	1.80	1.98	1.54	1.98

Table 5.3 Sensitivity analysis of reference case to the selected empirical parameters (1KCM= 1×10^3 m³).

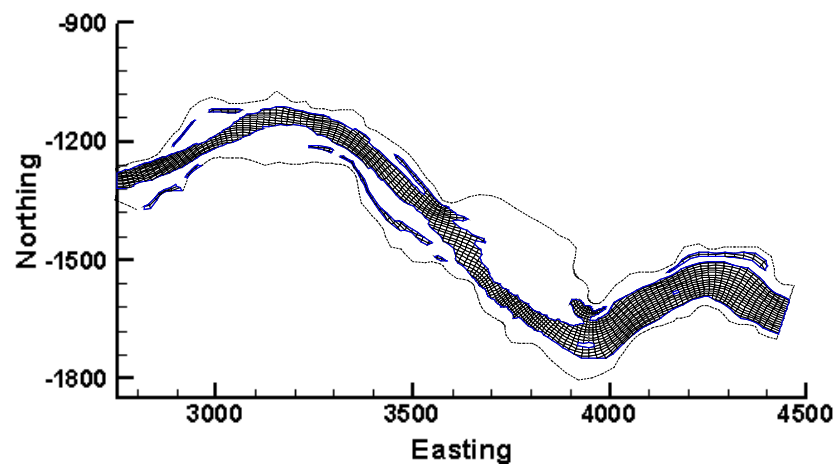
5.3.2 Bed changes simulation using Meyer-Peter-Müller and Van Rijn's bed load formula in Dashidaira reservoir

The measured bed levels before and after the flushing operation in June 2012 was used to set up and verify the calculated morphological bed changes. Similar to the observations in the prototype, a distinctive flushing channel appeared in numerical simulations during the free-flow condition. Figure 5.10 shows the computational grid adjustment at the beginning (i.e., $t=10$ hrs) and middle of drawdown stage (i.e., $t=16$ hrs), during the free-flow condition with a low water head in the reservoir (i.e., $t=32$ hrs), and also within the recovery stage (i.e., $t=44$ hrs). Cells with a lower water head than a specified

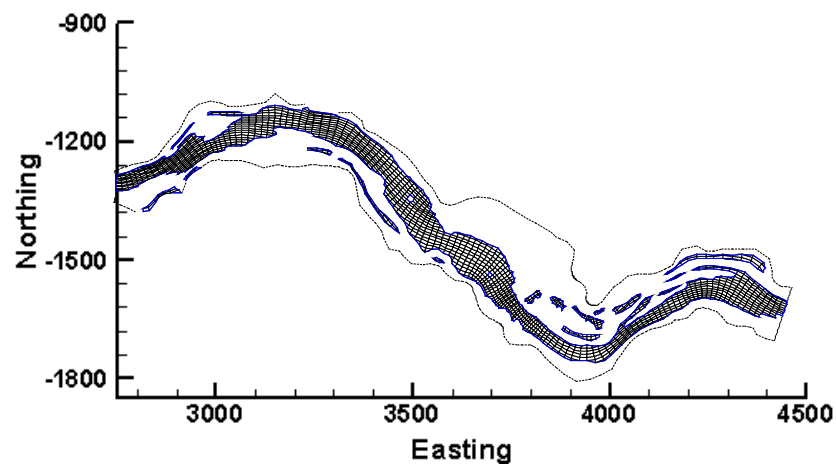
value will be removed from the computational domain due to the employed wetting/drying algorithm. Outputs reveal that flow is deviated to the right hand side of area II during the free-flow condition and therefore, flushing channel location is close to the right bank. This flow deviation can be attributed to the attraction of main flushing flow by the thalweg of existing channel, which is close to the right bank.



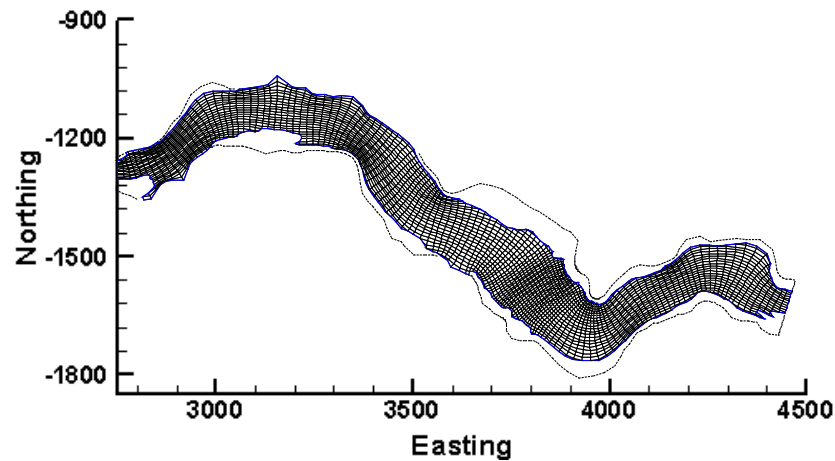
(a)



(b)



(c)



(d)

Figure 5.10 Computational grid (a) at the beginning of drawdown stage ($t=10$ hrs); (b) at the middle of drawdown stage ($t=16$ hrs); (c) during the free-flow condition ($t=32$ hrs); (d) within the refilling stage.

Figure 5.11 shows the surface water velocity at the beginning and middle of drawdown stage, and also during the free-flow condition, respectively. As can be seen clearly from Figure 5.11(a), a complex flow field with strong reverse flow pattern and water stagnation zone develops over the lower half of the reservoir. This can be attributed to the complex geometry of computational domain and variation of flow depth from shallow conditions in upstream areas to deep conditions in downstream areas, at the same time, on the existing bed roughness in different areas of the computational domain. In addition, Figure 5.11(b) reveals the onset of water flow concentration in the flushing channel while the water level is still decreasing during the drawdown stage. This process contributes in widening and deepening of the flushing channel. During the free-flow condition, velocities rise up to 4.5 m/s and super critical flows emerges in several zones of the flushing channel as can be observed in Figure 5.11(c). The complexity of flow field further magnifies in channel bends where the secondary currents exist. As for the secondary currents, their governing zones may be small and local but their contribution on morphological processes is significant. 3-D numerical model also captured the characteristic of bed development in the channel bend (i.e. erosion at the outside of the bend and deposition at the inside) and reproduced the nonsymmetrical velocity profile over the width as well as tilting the lateral water surface in the apex of the channel bend (i.e., section X-X of Figure 5.11(c)). These outputs have been illustrated in Figure 5.12 (Esmaeili et al., 2015a).

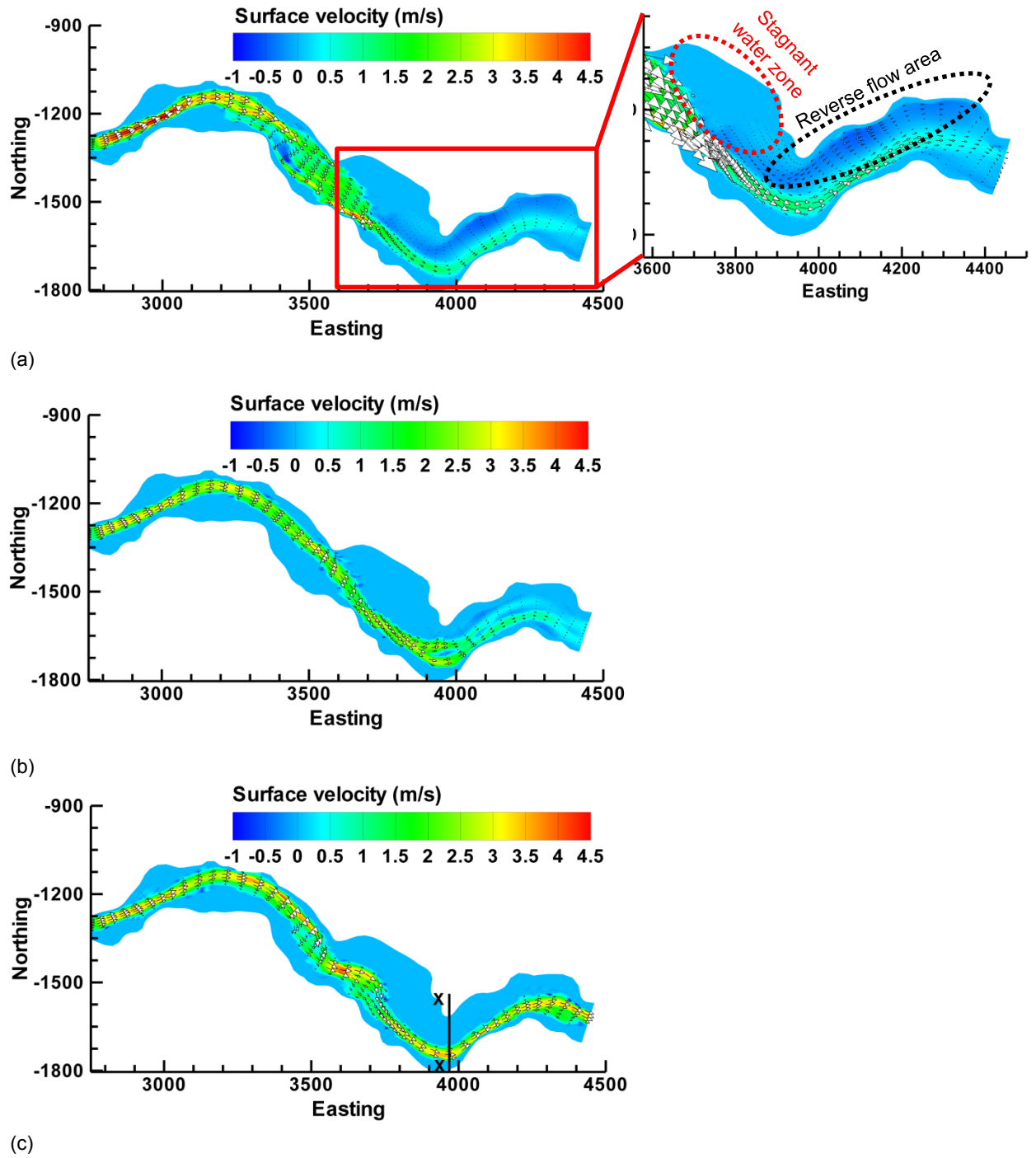


Figure 5.11 Surface velocity fields at different stages of 2012 free-flow flushing operation in Dashidaira reservoir: (a) at the beginning of drawdown stage ($t=10$ hrs); (b) at the middle of drawdown stage ($t=16$ hrs); (c) during the free-flow condition ($t=32$ hrs). Right illustration in Figure 5.11(a) shows the stagnant water zone and reverse flow domain.

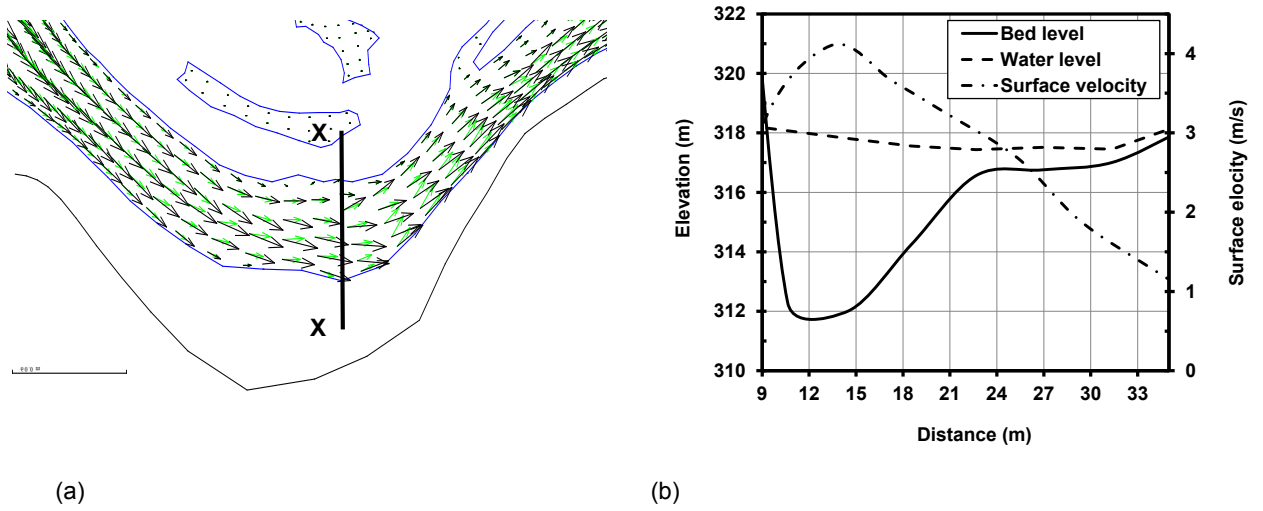


Figure 5.12 (a) Simulated flow velocity field in the flushing channel during the free-flow state ($t=32\text{hrs}$), around a channel bending area ; surface velocity vector (black) and bottom velocity vector (green) show the secondary currents development in the channel bend; (b) bed morphology and surface velocity distribution pattern in cross-section x-x at the apex zone of the bend (Esmaili et al., 2015a).

Figure 5.13 illustrates the simulated final bed topography after flushing using MPM and Van Rijn formula versus the measured one. The computed TVFS was $313.14 \times 10^3 \text{ m}^3$ when MPM formula was used whereas it was $333.91 \times 10^3 \text{ m}^3$ when Van Rijn's formula was utilized. Figure 5.14 depicts the measured and simulated bed changes in different cross sections after the flushing operation in the reservoir. The location of cross-sections can be found in Figure 5.8. In order to have a more quantitative insight into the outcomes, the Bed Changes Index (BCI) has been defined as:

$$BCI = \frac{\sum_{i=1}^n (z_{i_ms} - z_{i_reference})}{n} \quad (1)$$

where z_{i_ms} is the measured or simulated bed level after flushing operation at each grid node and $z_{i_reference}$ is measured or simulated reference bed level (e.g., before flushing) at the corresponding node, which is used for comparison purpose to provide information about erosion or deposition. n is also the number of nodes considered for comparison purpose. The positive and negative values of BCI represent the deposition and erosion condition, respectively. In other words, BCI reveals the average change of bed levels in each target zone and one can simply notice the dominant morphological process (i.e., erosion or deposition) in the zone compared to the reference case. Table 5.4 clearly shows the average bed level changes using BCI parameter in upstream, middle and downstream area utilizing the measurements, and also using MPM and Van Rijn bed load sediment transport formula.

Parameter	Measured bed level			Simulated bed level using MPM formula			Simulated bed level using Van Rijn formula		
	I	II	III	I	II	III	I	II	III
BCI (m)	-2.03	-0.67	-0.59	-1.61	-1.59	0.96	-0.15	-3.11	-0.18

Table 5.4 Average bed changes in different areas of Dashidaira reservoir using measurements and computations.

As can be seen from Figure 5.14, in the upstream area I, in cross-sections A-A to E-E simulated bed levels using MPM formula had a better agreement with the measurements compared with simulated ones using Van Rijn formulas. This can be found quantitatively from table 5.4 where the measurements disclose 2.03 m average erosion while using MPM formula and Van Rijn's formula show 1.61 m and 0.15 m average erosion, respectively. However, the severe erosion in the lower part of area I, along the left bank, could not be simulated properly. The reason can be attributed to the elimination of cells with shallow water head from the computational domain in this area because of the employed wetting/drying algorithm. In the wide middle segment (i.e. area II), calculated bed levels by all the sediment transport formulas are lower than the measured ones, which represents a higher erosion rate. In addition, predicted morphological bed changes in this area using MPM formula had a smaller overestimation of erosion compared to bed changes obtained from Van Rijn formula. In the area close to the dam (i.e. area III), results of the simulations show a narrower flushing channel compared to the measurements regardless to the type of used sediment transport formulae. This can be attributed to the complex flow field formed by secondary currents and reverse flow pattern in the wide bending area close to the dam. In addition, bed forms (i.e. dunes and ripples) which are developed in this area will increase the complexity of the flow pattern and subsequently the sediment transportation process. The empirical formula used in the model to take the bed forms effect into account, has been developed for small Froude numbers. Therefore, it is very likely to simulate different bed forms than existing ones. Outcome would be a different calculated morphological bed pattern. Moreover, in this area the sediment transport capacity is reached to its highest level that would be smaller than the original value in the prototype because of the simplified downstream boundary condition. Nevertheless, application of Van Rijn formula results in a wider flushing channel in area III which is in a better agreement with the measurements. When MPM formula is employed, the eroded coarse materials from area I may be deposited again in area III where the flow velocity is reduced due to a larger water depth (Esmaeili et al., 2015b). The better performance of Van Rijn's bed load formula in area III can also be found from Table 5.4. While application of MPM formula results in deposition and rising bed levels (i.e., 0.96 m), using Van Rijn formula represents a small magnitude of erosion in this area (i.e., -0.18 m). Since bed material type and water head varies in different segments of the reservoir, each bed-load transport formula (i.e. MPM and Van Rijn) may provide a more reasonable prediction of bed evolution pattern in a specific segment.

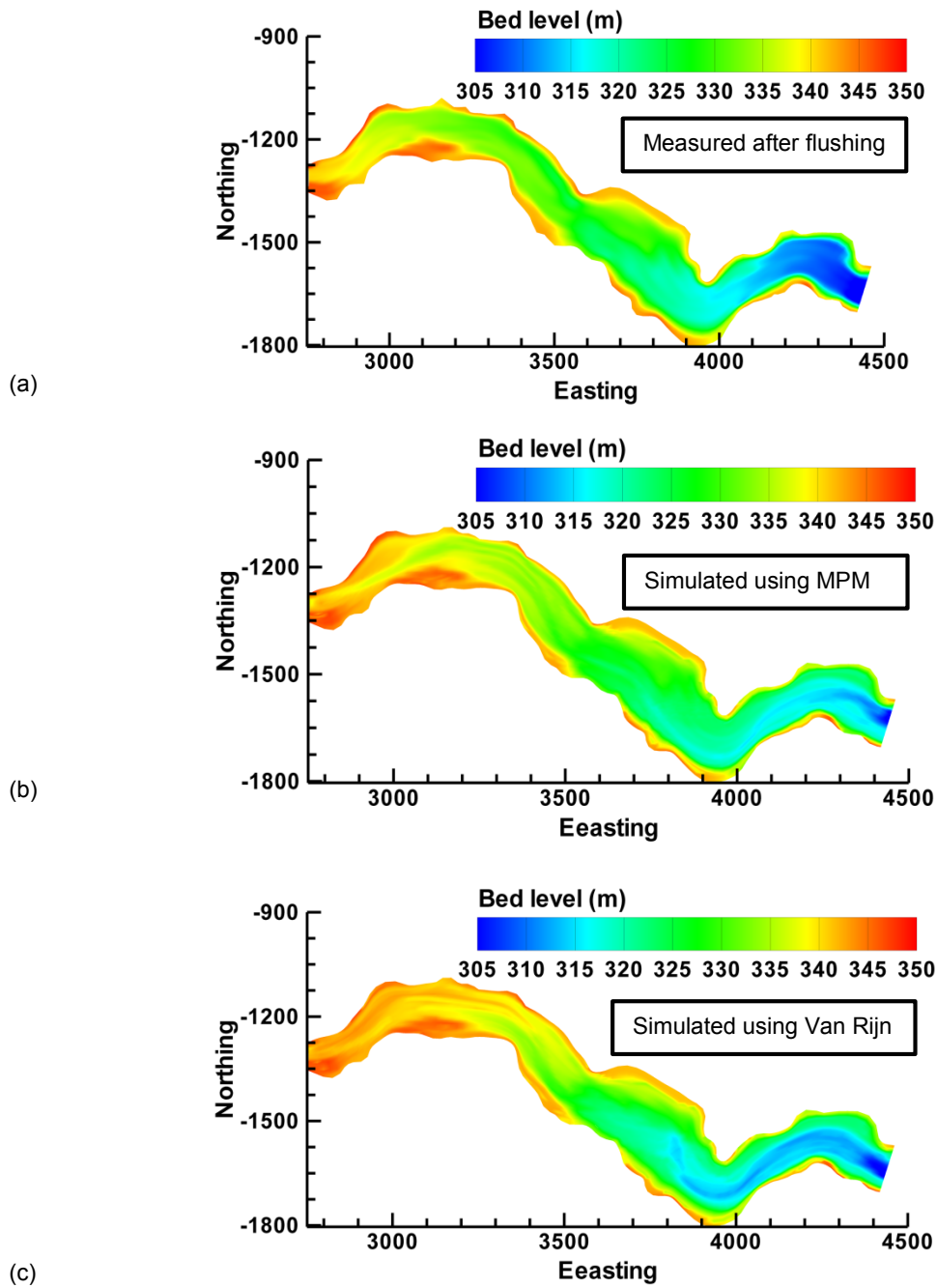
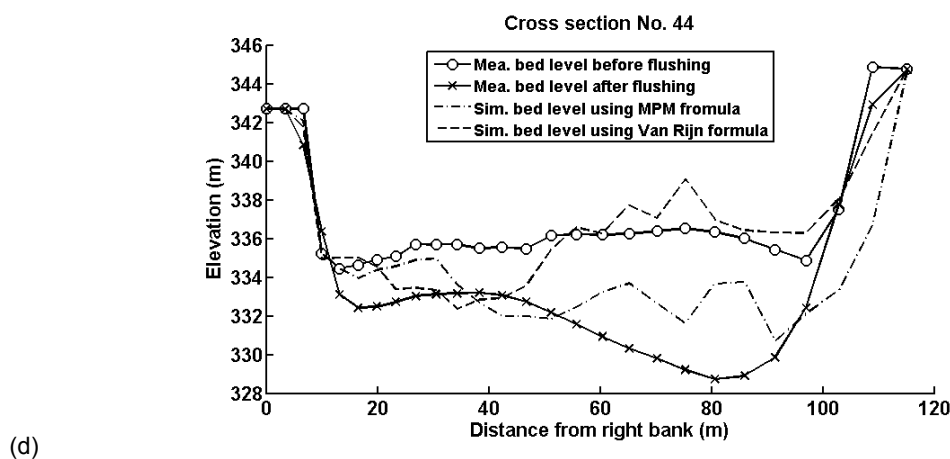
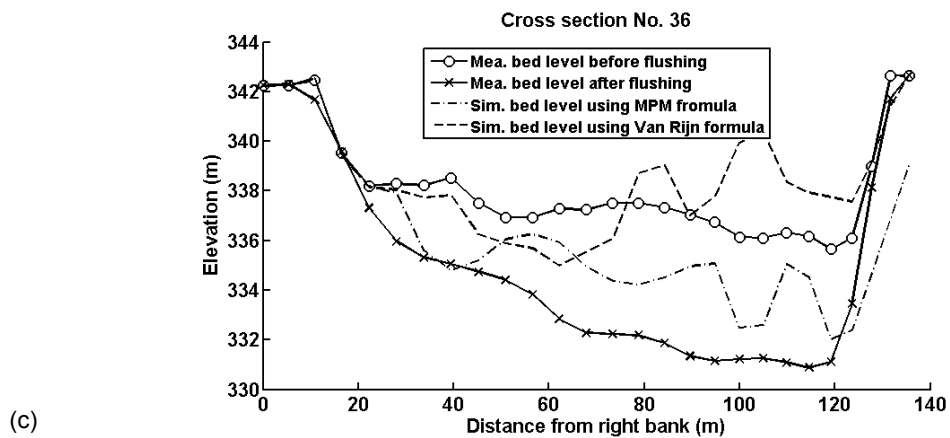
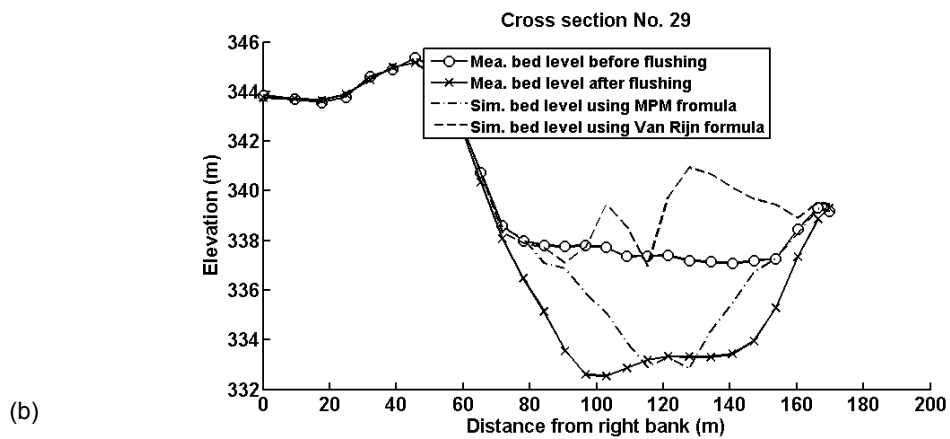
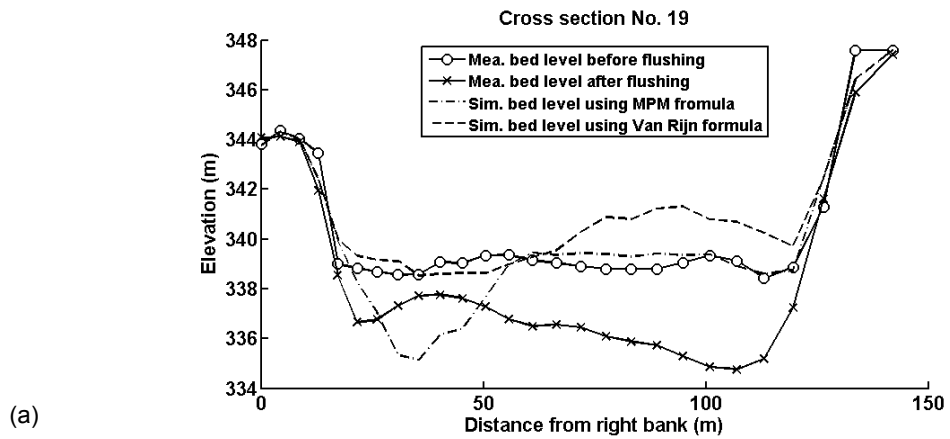
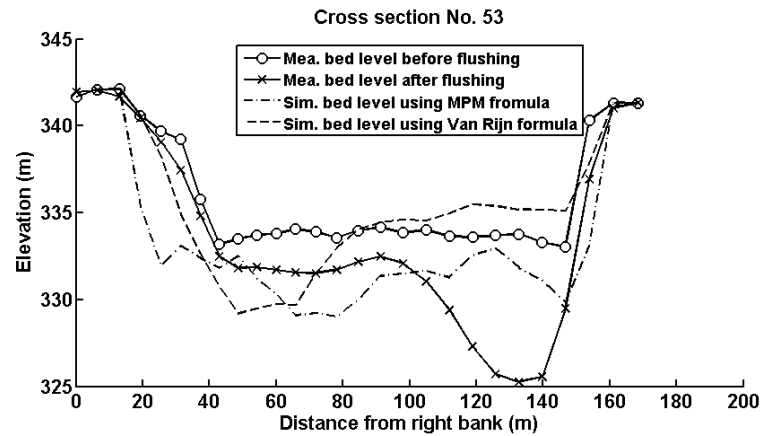
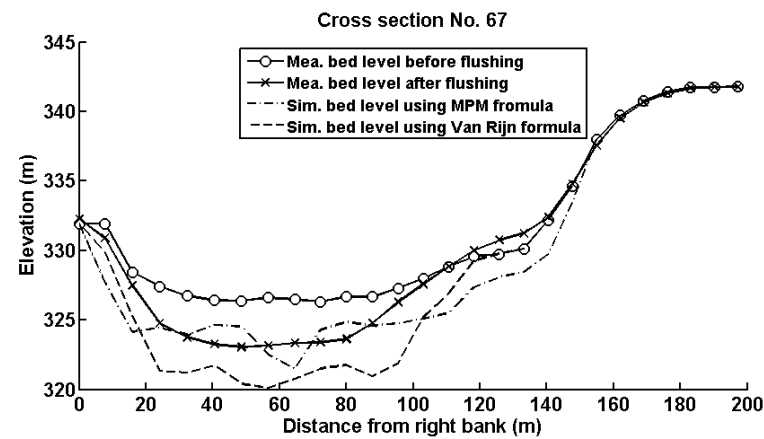


Figure 5.13 Plan view of bed levels after flushing operation: (a) measured one; (b) simulated one using MPM formula; (c) simulated one using Van Rijn formula.

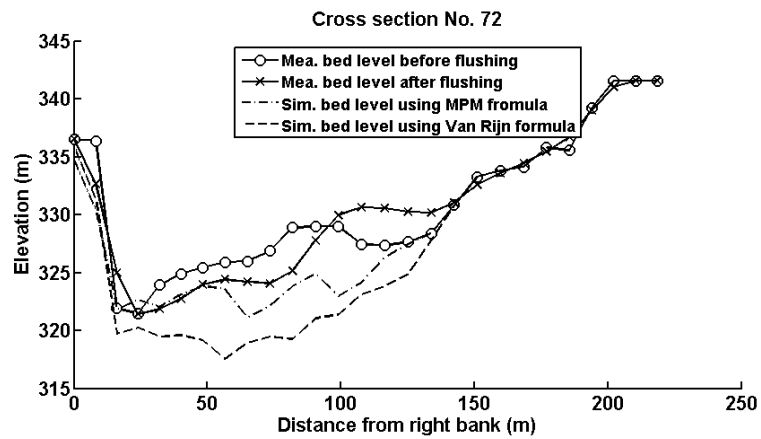




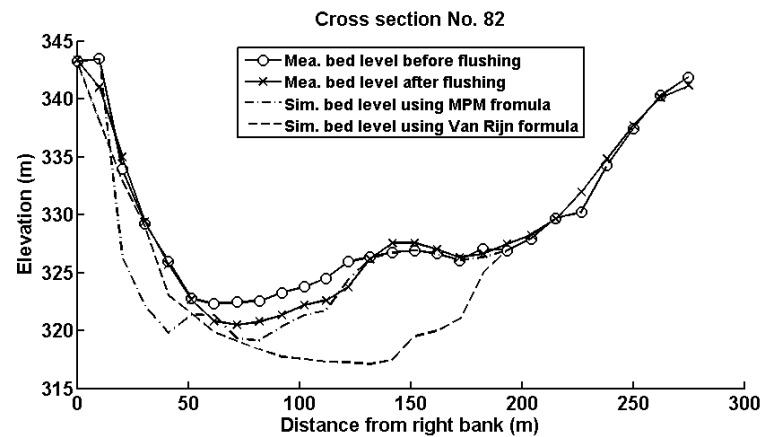
(e)



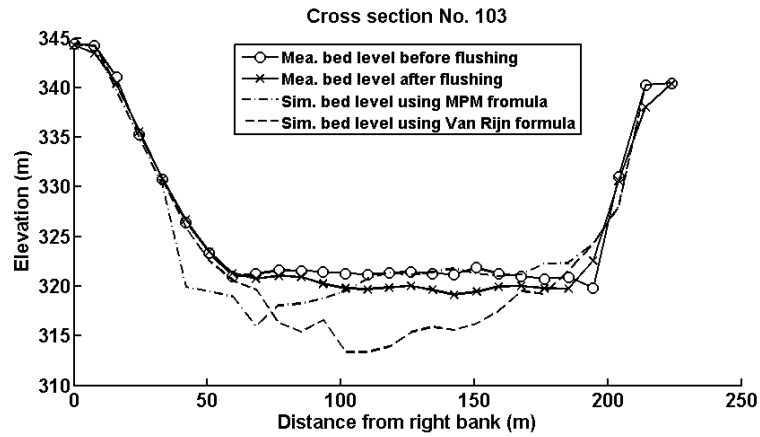
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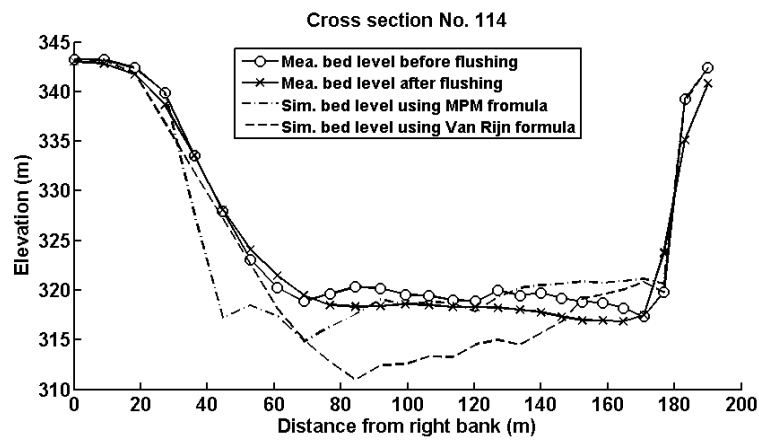
(g)



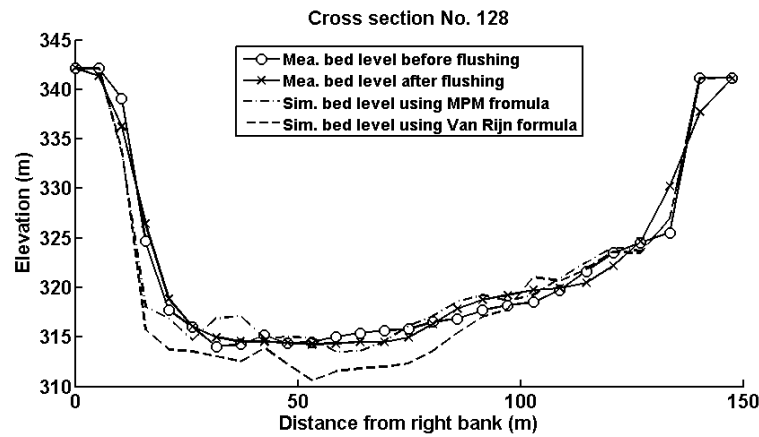
(h)



(i)



(j)



(k)

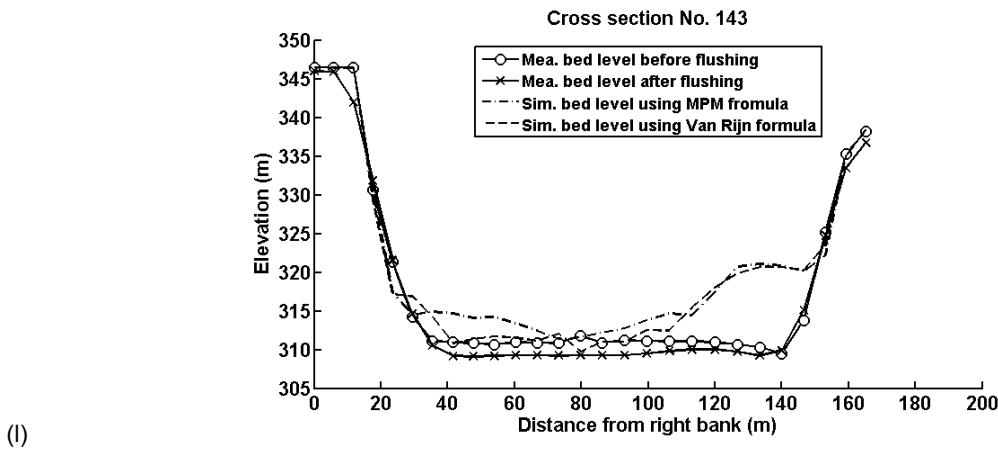


Figure 5.14 Bed evolution pattern in cross-sections (A-A to L-L) illustrated in Figure 5.8, respectively using MPM and Van Rijn formula (e.g., (a) corresponds to cross-section A-A and (l) corresponds to cross-section L-L). Mea. & Sim. are abbreviations for Measured and Simulated bed levels, respectively.

5.3.3 Computation of the variation of flushed out sediment concentration and discussions

Figure 5.15 demonstrates the computed fluxes of eroded bed material that is flushed out in one hour intervals during the flushing process mainly as the bed load. As can be clearly seen, an increase in the average flow velocity and in the fluctuation of turbulences, that is the case for the final portion of the preliminary and the free-flow stage, tends to enhance the sediment mobility and subsequently sediment entrainment. In other words, a correlation between the flushed out sediment discharge and water discharge as well as the water level variation can be observed (Esmaili et al., 2015a).

Smaller sediment size tends to be eroded and flushed out earlier than larger sediment sizes. For instance, sand sized sediment (i.e., with diameter of 3.7mm) experiences a high concentration immediately after the flood peak at $t=5$ hrs. The concentrations of smaller sediment sizes are sensitive to both water level and also discharge fluctuations (e.g., sediment size 3.7 mm and 11.8 mm). On the other hand, bigger sediment sizes are more sensitive to water level changes than to discharge variations (e.g., 118.3 mm and 37.4 mm). Thus, coarser bed materials are mainly flushed out at the end of the drawdown stage and during the free-flow state (Esmaili et al., 2015a).

As a result, introducing an additional water discharge with small fluctuations during the free flow condition may increase the possibility of sediment entrainment. In addition, increasing the water level drawdown speed may also increase the erosion of bed materials.

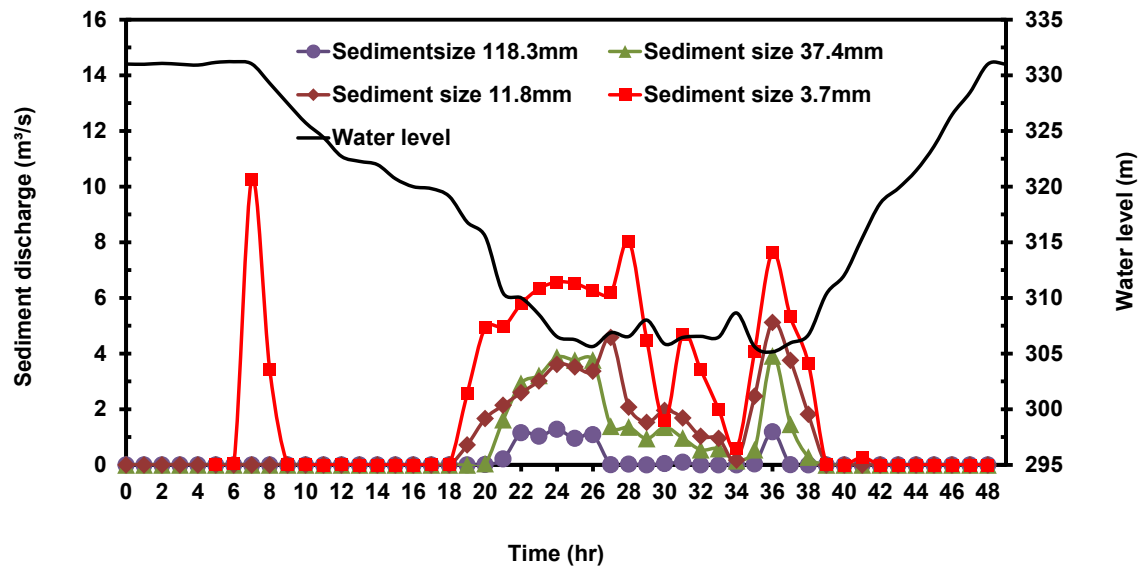


Figure 5.15 Temporal variation of flushed out sediment discharge from Dashidaira reservoir in one hour intervals.

5.4 Model set-up for simulation of flushing in Unazuki reservoir

The computational grid was made based on the measured bed levels before the flushing presented in Figure 5.3 (b1) and it has been depicted in Figure 5.16. The mesh cell size in streamwise and transversal direction was 10-20 (m) and 5-10 (m) respectively. The bed material density was assumed to be 2650 (kg/m³). Figure 5.16 shows the initial computational grid adjustment before starting the drawdown stage. The water levels and inflow discharge fluctuations were shown in Figure 5.5 (b) were employed as the hydrodynamic boundary conditions in the simulations. In addition, a non-uniform bed material size distribution with spatial varying fractions, as described in part 5.2.3, and Table 5.2 was used to make the initial bed material distribution in the reservoir area.

Figure 5.17 illustrates the measured bed levels after the flushing operation in June 2012 along with the location of cross-sections (A-H) for a further assessment of bed variations during the flushing simulation. As already mentioned in part 5.2, deposited sediments from the upstream areas of Unazuki reservoir could not be eroded effectively and instead sediment deposition occurs in this area. This happens because the reservoir has been newly built and still the equilibrium condition has not been reached over the whole reservoir, which causes the coming sediment load to deposit. The sediment deposition would be also continues during the coordinated sediment flushing when the flushed out sediment from Dashidaira reservoir comes into the Unazuki reservoir. Since upstream part of Unazuki reservoir is wider compared to the other parts of the reservoir and also has been covered by coarser materials, sediment transportation process would be complex in this area. Due to the importance of increasing the sediment erosion from this area for a more efficient flushing operation, this study focuses locally on this area. Therefore, upstream area of the study case has been divided into two sub areas, namely, area I

and II. Bed materials are coarse in both sub-areas but they are coarser in area I since it is located further upstream.

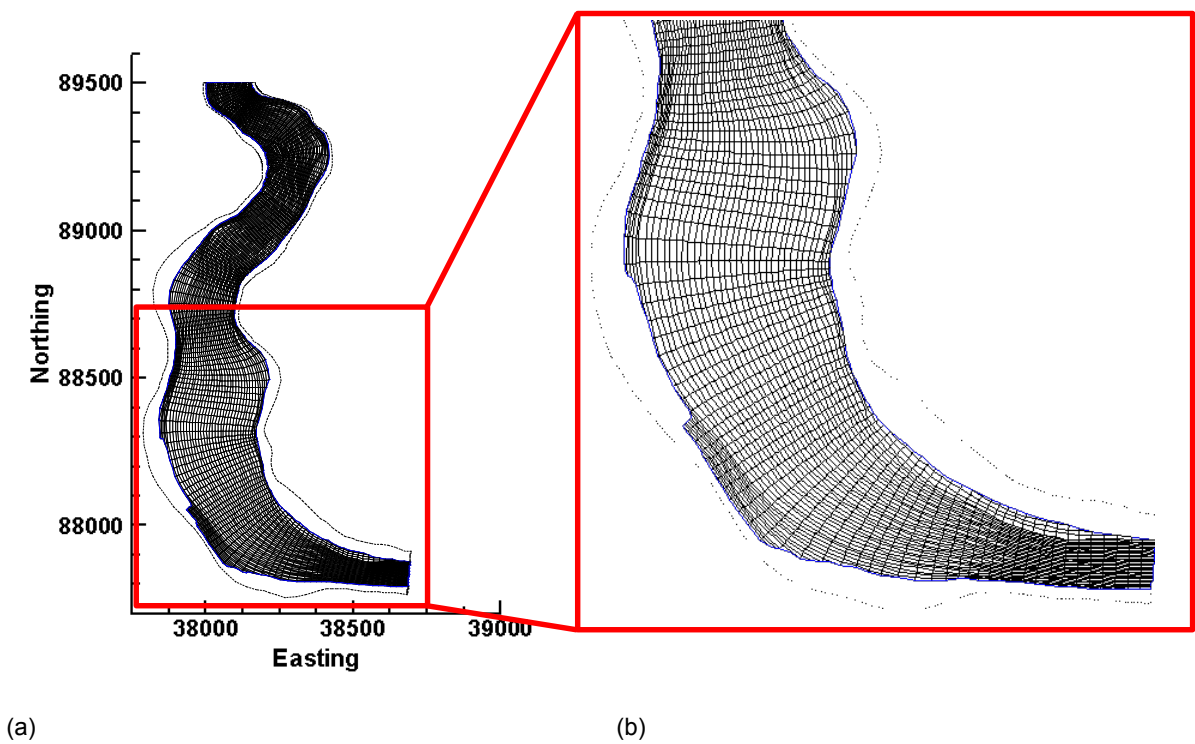


Figure 5.16 Computational grid for Unazuki reservoir at the beginning of simulation ($t=0$) ; (a) for the whole reservoir ; (b) for the study area.

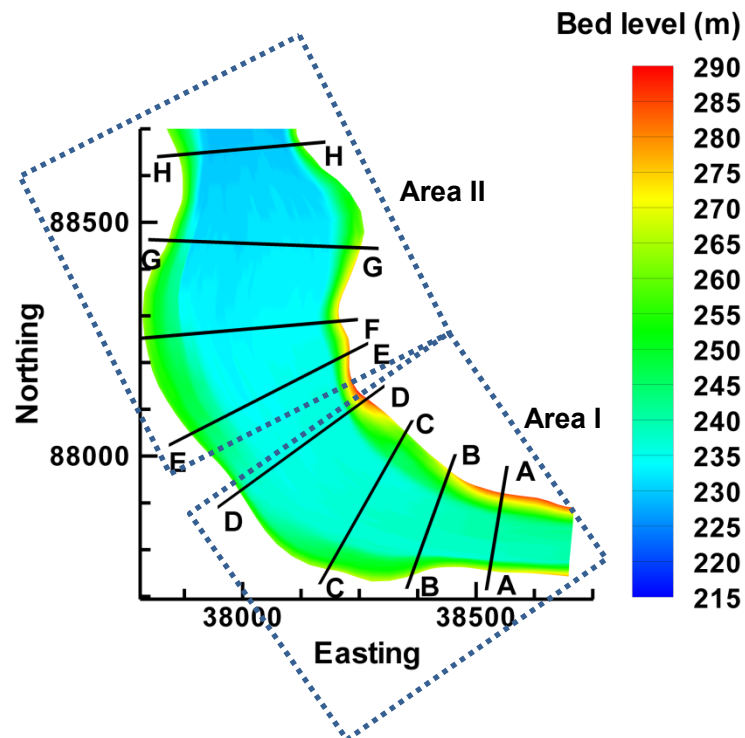


Figure 5.17 Cross-sections A-H for bed change assessment in Unazuki reservoir at the study area. Area I and II are both covered with coarse materials.

5.4.1 Model calibration

Since Unazuki reservoir has been located downstream of the Dashidaira reservoir and the flushing operation in the Unazuki reservoir is linked to the Dashidaira reservoir, the sediment transportation characteristics is affected by the flushing operation in the Dashidaira reservoir. Besides, in contrast to the Dashidaira reservoir which mainly sediment erosion in the whole reservoir area, both sediment deposition and erosion occurs in Unazuki reservoir as the result of flushing operation. Thus, for the calibration purpose in the study area of Unazuki reservoir, the already obtained values for main empirical parameters (e.g. roughness, active layer thickness, water content of the bed material and critical angle of repose) in Dashidaira reservoir was employed. In addition, the Meyer-Peter-Müller sediment transport formula was used because of its generally better performance in predicting the morphological bed changes caused by the flushing, especially in areas covered with coarser materials, as it was shown in case of Dashidaira reservoir. Afterwards, the final simulated bed topography pattern in the study area of the Unazuki reservoir was compared both qualitatively and quantitatively with the measurements. Qualitative assessment is performed to check whether the erosion in the lower part of area II could be simulated or not and quantitative assessment is conducted to figure out whether dominant morphological pattern (i.e., erosion or deposition) in each area can be simulated or not, and also to measure the deviation of simulated bed levels from the measured one.

Table 5.5 shows the BCI parameter of the calibrated case in the study areas and compared with the measured one. In addition, Root Mean Squared Error (RMSE) and Mean Absolute Error (MAE) of simulated bed levels after the flushing operation in the calibrated case were calculated. While BCI parameter shows that dominant morphological pattern similar to the measured one (i.e., deposition in Area I and erosion in Area II) represents numerically, RMSE and MAE show that deviation of simulated bed levels from the measured ones in area I is bigger than area II. This pronounces the complexity of the simulation of sediment transportation in areas covered with the coarse materials using the empirical sediment transport formulas (i.e., Meyer-Peter-Müller formula). It should be also taken into account that in the study area a very shallow flow condition appears, especially during the free-flow stage, which can be deflected easily due to the small disturbances since there is no a distinctive flushing channel to attract and stabilize the flow. This issue has been already discussed in Chapter 4, Part 4.4 and 4.5. In addition, as already mentioned in case of Dashidaira reservoir, bed levels are not measured immediately after and before flushing. Since Kurobe River is a steep river with dynamic bed changes and high sediment yield, small floods can disturb easily the morphology of river bed in the study area of the reservoir. Thus, it would be difficult to simulate the bed changes representing a high level of quantitative agreement with the measurements that are not conducted soon after the flushing operation in this area.

Parameter	Measured bed level		Simulated bed level using MPM formula	
	Area I	Area II	Area I	Area II
BCI (m)	0.11	-0.07	0.11	-0.10
RMSE (m)	-	-	1.23	0.96
MAE (m)	-	-	0.62	0.58

Table 5.5 Measured and simulated average bed changes in study areas of Unazuki reservoir along with the RMSE and MAE of the simulated bed levels.

5.4.2 Bed changes simulation using Meyer-Peter-Müller bed load formula in Unazuki reservoir

The measured bed levels before and after the flushing operation in June 2012 was used to set up and simulate the morphological bed changes. Figure 5.18 shows the computational grid adjustment at the middle of drawdown stage (i.e., $t=18$ hrs), and also during the free-flow condition with a low water head in the study area (i.e., $t=32$ hrs). Cells with a lower water head than a specified value will be removed from the computational domain due to the employed wetting/drying algorithm. Outputs reveal that flow deviates to the left hand side during the drawdown stage and, then, during the free-flow stage, it deviates towards the right half of the reservoir. Thus, the narrow flushing channel location would be shifted from the left hand side towards the right hand side.

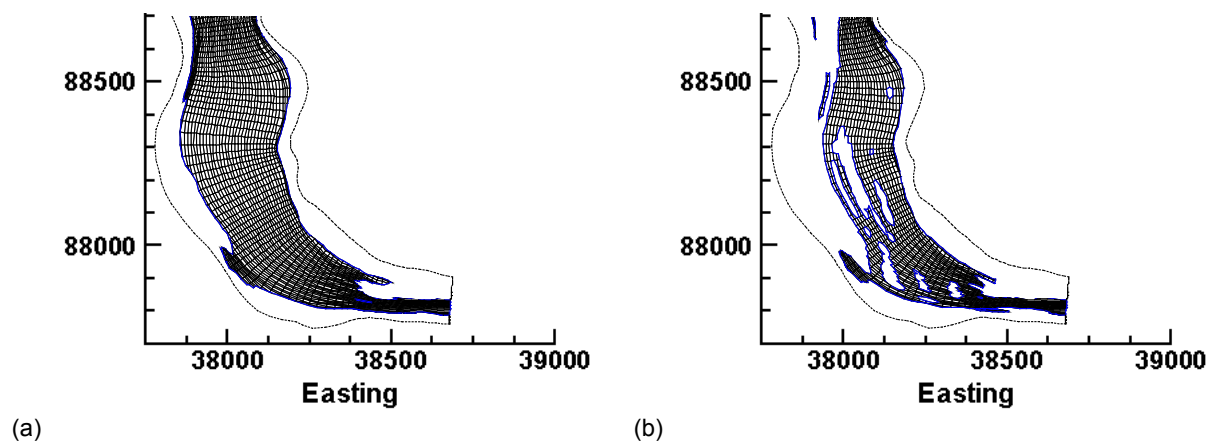
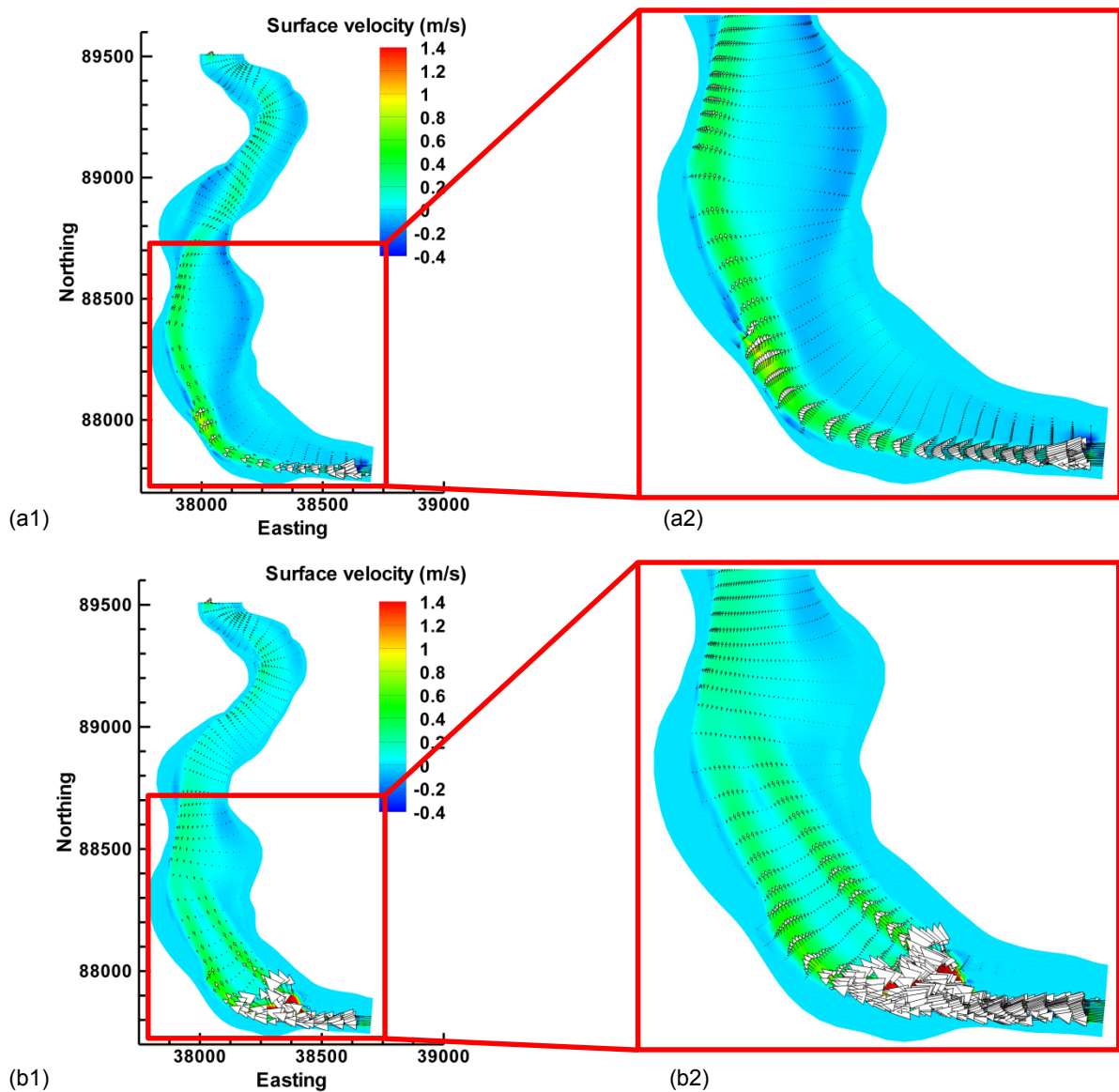


Figure 5.18 Computational grid (a) at the middle of drawdown stage ($t=18$ hrs); (b) during the free-flow condition ($t=32$ hrs).

Figure 5.19 shows the surface water velocity at the beginning, middle of drawdown stage, and also during the free-flow condition, respectively. As can be seen clearly from Figure 5.19 (a1) and 5.19 (a2), the flushing flow, at the beginning of drawdown (i.e., $t=10$ hrs), deviates to the left bank in the study zone (i.e., upstream half) while it changes its path towards the right bank in downstream half. This can be

attributed to the complex geometry of computational domain (i.e., meandering) and variation of flow depth from a wider and shallower condition in upstream half to a narrower and deeper condition in downstream half on the existing bed roughness. Figure 5.19 (b1) and 5.19 (b2) also reveal the gradual change of flushing flow location from left bank towards the center of the reservoir. The onset of flow bifurcation and water flow concentration in two distinctive narrow flushing channels (i.e., right and left) during the drawdown stage can be seen in these figures as well. This process (i.e., bifurcation and water level drawdown) contributes in developing of these two narrow channels. Afterwards, during the free-flow condition, velocities rise up to 2.5 m/s and super critical flows emerge in several zones of the formed flushing channels as can be observed in Figure 5.19(c1) and Fig 5.19 (c2). It should be also noted that when transition of the channel location happens from the left side towards the center of reservoir and later on towards the right side, a distributed and irregular longitudinal scouring in the form of small channel networks will be created over the transition area as it has been shown in Figure 5.20.



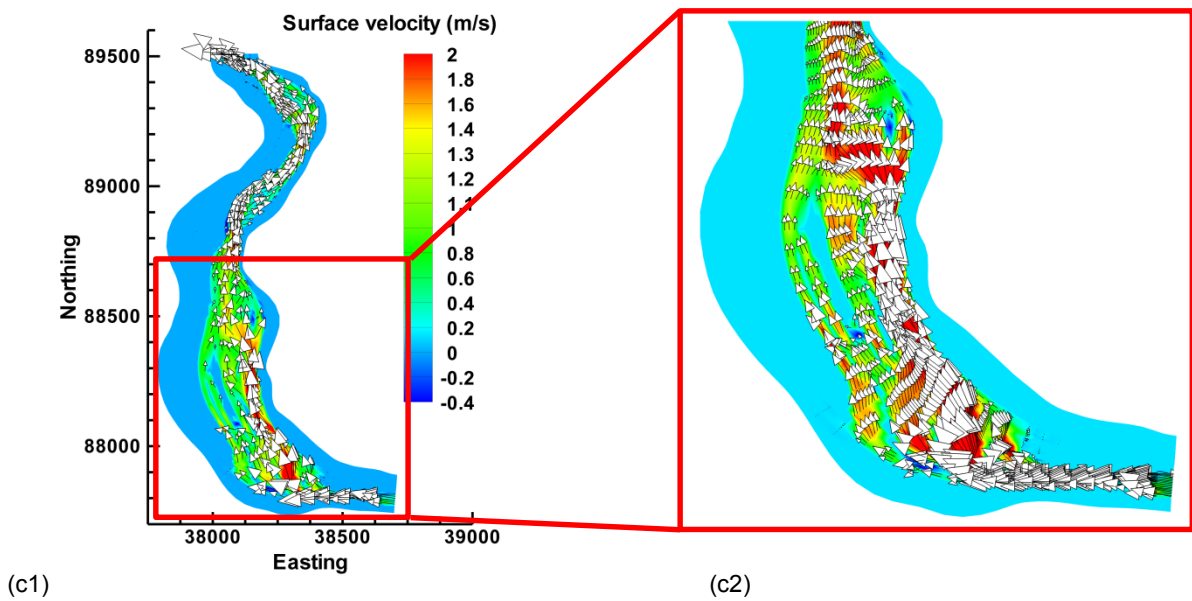


Figure 5.19 Surface velocity fields at different stages of 2012 free-flow flushing operation in Unazuki reservoir: (a1) at the beginning of drawdown stage ($t=10$ hrs); (b1) at the middle of drawdown stage ($t=18$ hrs); (c1) during the free-flow condition ($t=32$ hrs). Figures 5.19 (a2), (b2), and (c2) show the surface velocities over the study zone at corresponding times, respectively.

Figure 5.20 illustrates the measured bed changes after flushing versus the simulated one using MPM formula in the study area. As can be seen from Figure 5.20, both numerical model outcomes and measurements illustrate that deposition is the dominant result of flushing in area I, although some erosion occurs. In this area, there are some narrow and distributed channels where the erosion is overestimated by numerical model. The size and location of these channels are also different compared to the measurements. Regarding the area II, simulation results show that erosion magnitude in this area is bigger than area I, which is consistent with the measurements. However, numerical model overestimated the erosion in this area and represents a different erosion pattern especially in the lower half of area II. While measurements show a wide uniform channel erosion, simulations showed a wide erosion channel with a deeper part close to the right bank. It should be taken into account that in both area I and II, both deposition and erosion takes place, and their magnitudes are small (e.g., less than 2 m) compared to the reservoir width that makes it difficult to be simulated properly. In addition, bed forms (i.e. dunes and ripples) which are developed in this area will increase the complexity of the flow pattern and subsequently the sediment transportation process. The empirical formula used in the model to take the bed forms effect into account, has been developed for small Froude numbers. The problem can be further magnified in this study area with a shallow flow condition. Therefore, it is very likely to simulate different bed forms than existing ones. Nonetheless, the outcomes of the numerical model shows an overall better agreement with an aerial photo taken at an early stage after the flushing operation in terms of morphological bed changes in both areas as shown in Figure 5.21. Figure 5.22 also depicts the measured and simulated bed changes in different cross sections after the flushing operation in the reservoir. The location of cross-sections can be found in Figure 5.17. These cross sections disclose the aforementioned morphological bed changes quantitatively.

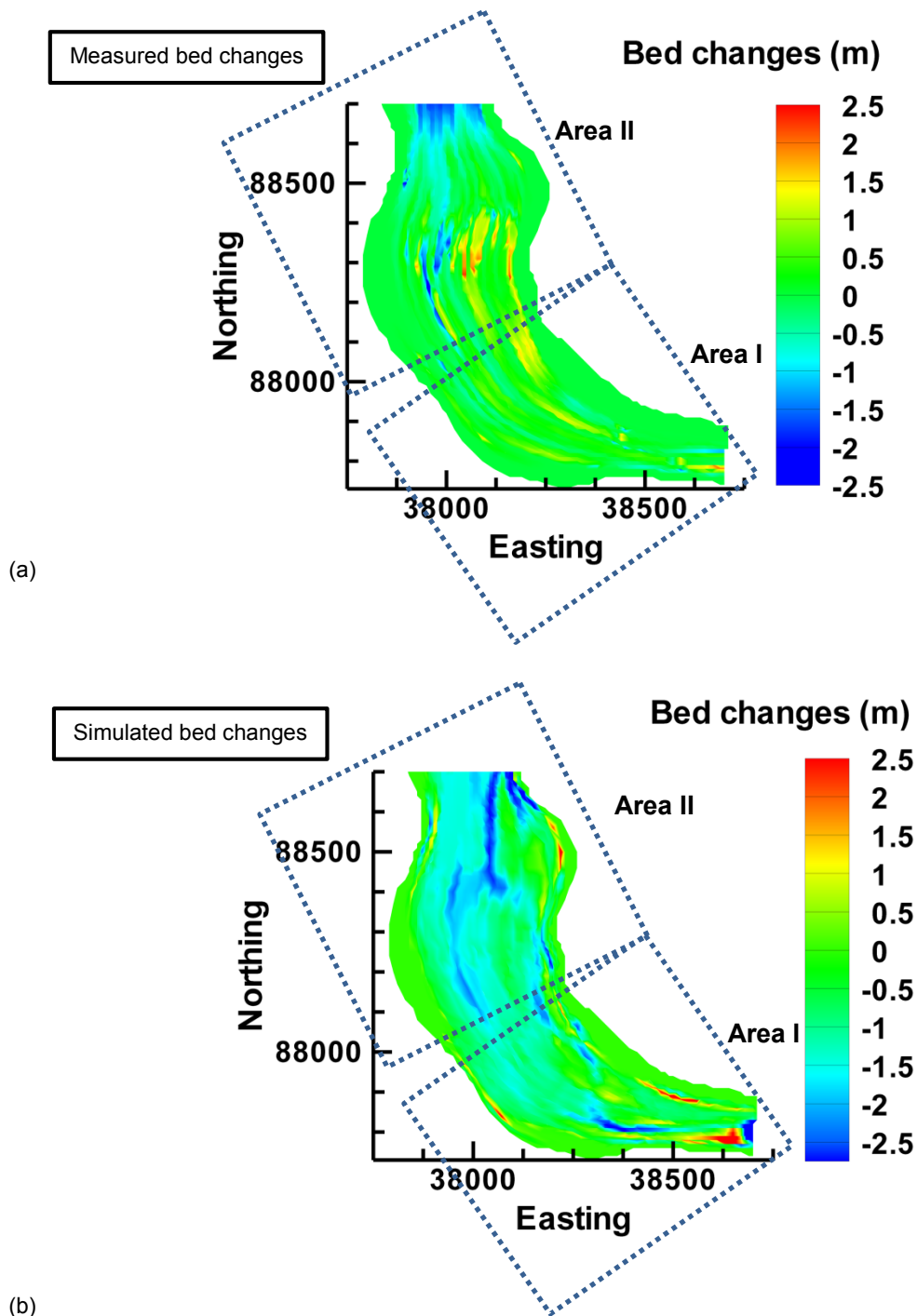
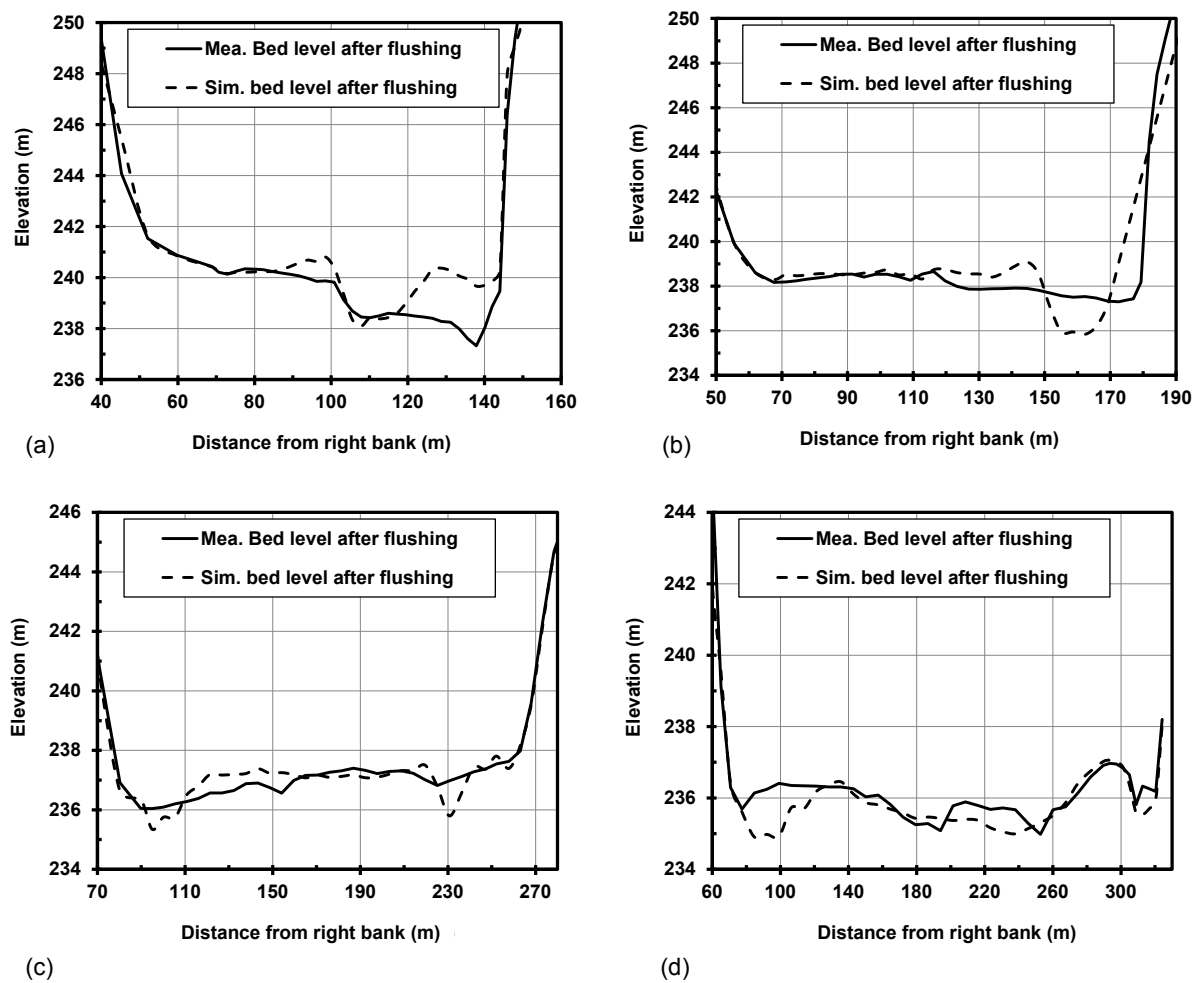


Figure 5.20 Plan view of bed variations after the flushing operation using (a) measured bed levels; (b) simulated bed levels.



Figure 5.21 Aerial photo showing the bed variations after a flushing operation in the target study area of Unazuki reservoir.



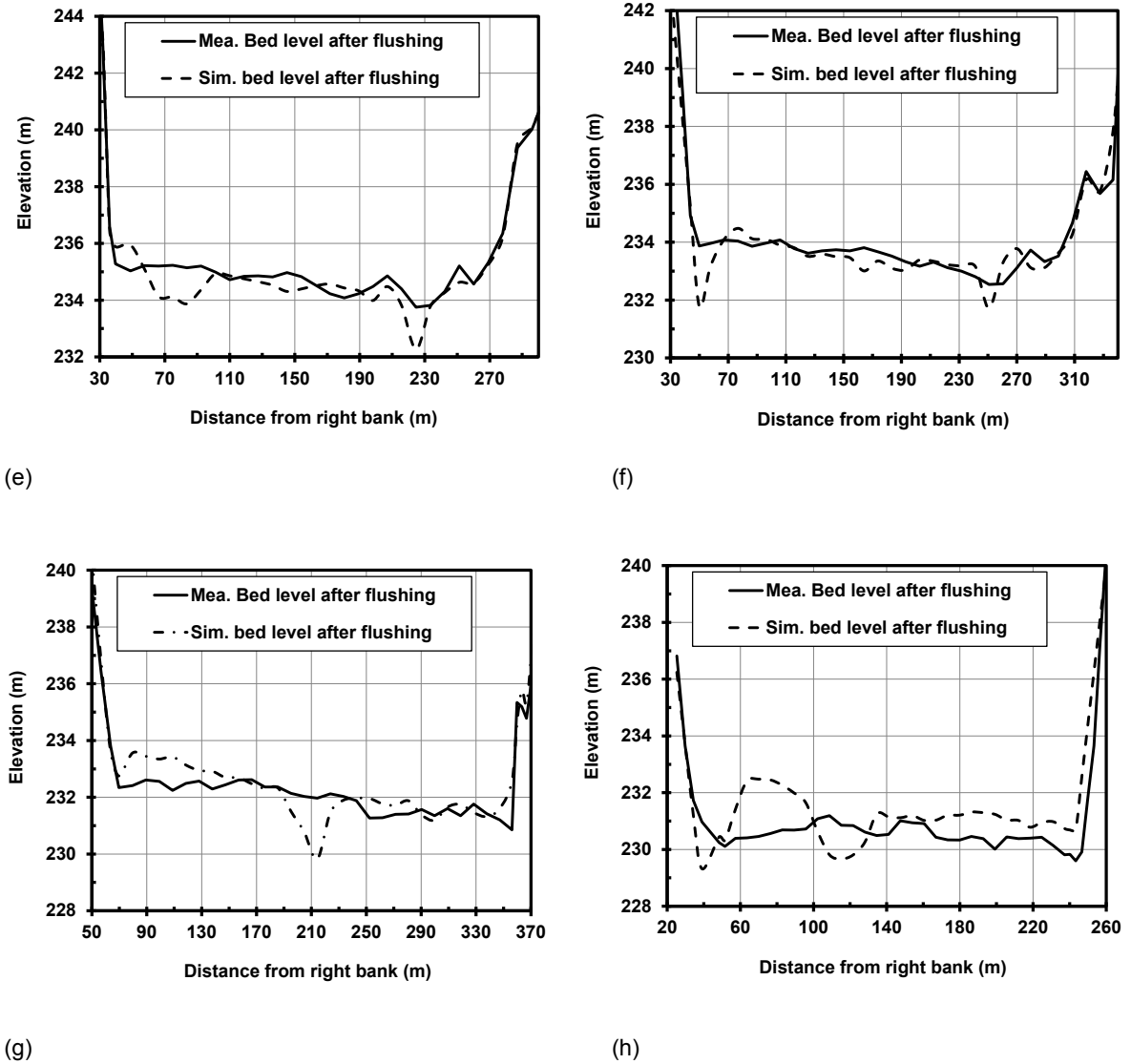


Figure 5.22 Bed changes in cross-sections (A-H) located across the study area illustrated in Figure 5.17. (a) corresponds to cross section A-A; (b) corresponds to cross section B-B; (c) corresponds to cross section C-C; (d) corresponds to cross section D-D; (e) corresponds to cross section E-E; (f) corresponds to cross section F-F; (g) corresponds to cross section G-G; (h) corresponds to cross section H-H.

Chapter 6 Potential approaches for enhancing the efficiency of sediment flushing in case study reservoirs

One of the interests of dam reservoir owners is increasing the ratio of the total volume of flushed out sediment from a reservoir to the volume of used water (i.e., flushing efficiency). Increasing the flushing efficiency can result in a more robust sediment management strategy in a dam reservoir through escalating the recovered reservoir volume from deposited materials. For instance the additional available reservoir volume can be beneficial for flood control, irrigation and drinking water supply, and more hydropower electricity generation. It should be taken into account that, increasing the flushing efficiency has some environmental implications. The increased flushed out sediment volume may be deposited in the stream channel, a reservoir, a water intake, irrigation channels etc (Morris & Fan, 1998). The deposited sediment clogs the river bed and can potentially damage the habitat structure of aquatic animals (i.e., fish and benthos). Besides, the sediment concentration may be drastically goes up so that it would be much bigger than what occurs in a natural fluvial system, even when a natural flood occurs. However, assessment of the environmental impacts through increasing the flushing efficiency is beyond the scope of this study and this chapter mainly focuses on different measures to increase the free-flow flushing efficiency itself.

Two general types of measures (i.e., soft and hard) were suggested and discussed in this chapter. Soft-measure scenarios correspond to manipulation of water level and discharge rates during a free-flow flushing operation. Introducing the additional discharge (i.e., discharge scenarios) or increasing the waterlevel drawdown speed (i.e., water level scenarios), are considered for soft-measure scenarios. Hard-measure scenarios implies the construction of flow-guide structures, and auxiliary longitudinal channels in the reservoir to enhance the sediment erosion in specific segments of the reservoir. While soft-measures affect the hydrodynamic characteristics over the whole reservoir during a time slot, hard measures influence the hydrodynamic characteristics in a specific zone of the reservoir during the whole time of simulation. In other words, soft-measures influence the whole scale but hard measures influence a local scale to affect the morphological processes in the reservoir. To figure out the effect of each scenario in each reservoir (i.e., Dashidaira and Unazuki), numerical outcomes were compared with the accepted calibrated model (i.e., reference case) for each reservoir that were presented in Chapter 5.

6.1 Discharge scenarios

The coordinated sediemnt flushig operation in Dashidaira and Unazuki reservoir with free-flow condition is performed when the first major flood happens in rainy season. Therefore, the necessary discharge for free-fow operation is suuplied naturally by the flood event in Kurobe River. As it was shown in Figure 2.17, there are other dams located in the upstreaam of Dashidaira reservoir that can supply artificial additional discharge during the free-flow flushing operation. In this section, the effect of introducing an additional artificial discharge during the drawdown and free-flow stage was evaluated on the bed changes and flushed out sediment volume in Dashidaira reservoir to investigate whether this type of so-called soft measure can increase the flushing efficiency or not. Then, the effective approcah was also employed to the Unazuki reservoir.

6.1.1 Introducing an Additional constant Discharge during the Free-flow stage in Dashidaira reservoir (ADF scenario)

Introducing an additional inflow discharge to Dashidaira reservoir form another reservoir located upstream is practically feasible during the free-flow flushing operation. This additional inflow discharge can enhance the sediment entrainment by escalating the average flow velocity and turbulence fluctuations when the gate operation is included (Esmaeili et al., 2015b). In ADF scenario, a constant additional discharge is added to the original discharge values during the free-flow condition in reference case while the original water level of reference case remains unchanged. For example, ADF 75 scenario represents the application of modified discharge rates by introducing a constant 75 m³/s additional discharge during the free-flow condition. In this part, effects of different ADF scenarios on TVFS and bed level changes using different ADF scenarios are evaluated numerically in Dashidaira reservoir. This is performed through comparing the numerical outcomes under different ADF scenarios with the numerical results available from the reference cases (i.e., calibrated model) for Dashidaira resevoir. Figure 6.1 illustrates the original water level and Discharge rates together with discharge modifications for different ADF scnarios in Dashidaira reservoir.

In Figure 6.2, calculated bed levels of reference case using the original discharge rates of 2012 flushing event and calculated bed levels under ADF 75, ADF 110, and ADF 170 scenarios have been depicted. The TVFS increased from $313.14 \times 10^3 \text{ m}^3$ to 356.04×10^3 under ADF 75 scenario, to 396.10×10^3 under ADF 110 scenario, and to $425 \times 10^3 \text{ m}^3$ under ADF 170 scenario. As can be observed from Figure 6.2, introducing the additional discharge increased the erosion in all areas but the effect in area close to the dam (i.e., area III) is more highlighted. Before the flushing operation, area III is mainly covered with the fine materials. During the simulation of flushing operation, eroded coarser materials from area I would be deposited again in area III. Thus, introducing an additional discharge during the free-flow condition can contribute in flushing out the deposited materials in this area (Esmaeili et al., 2015b).

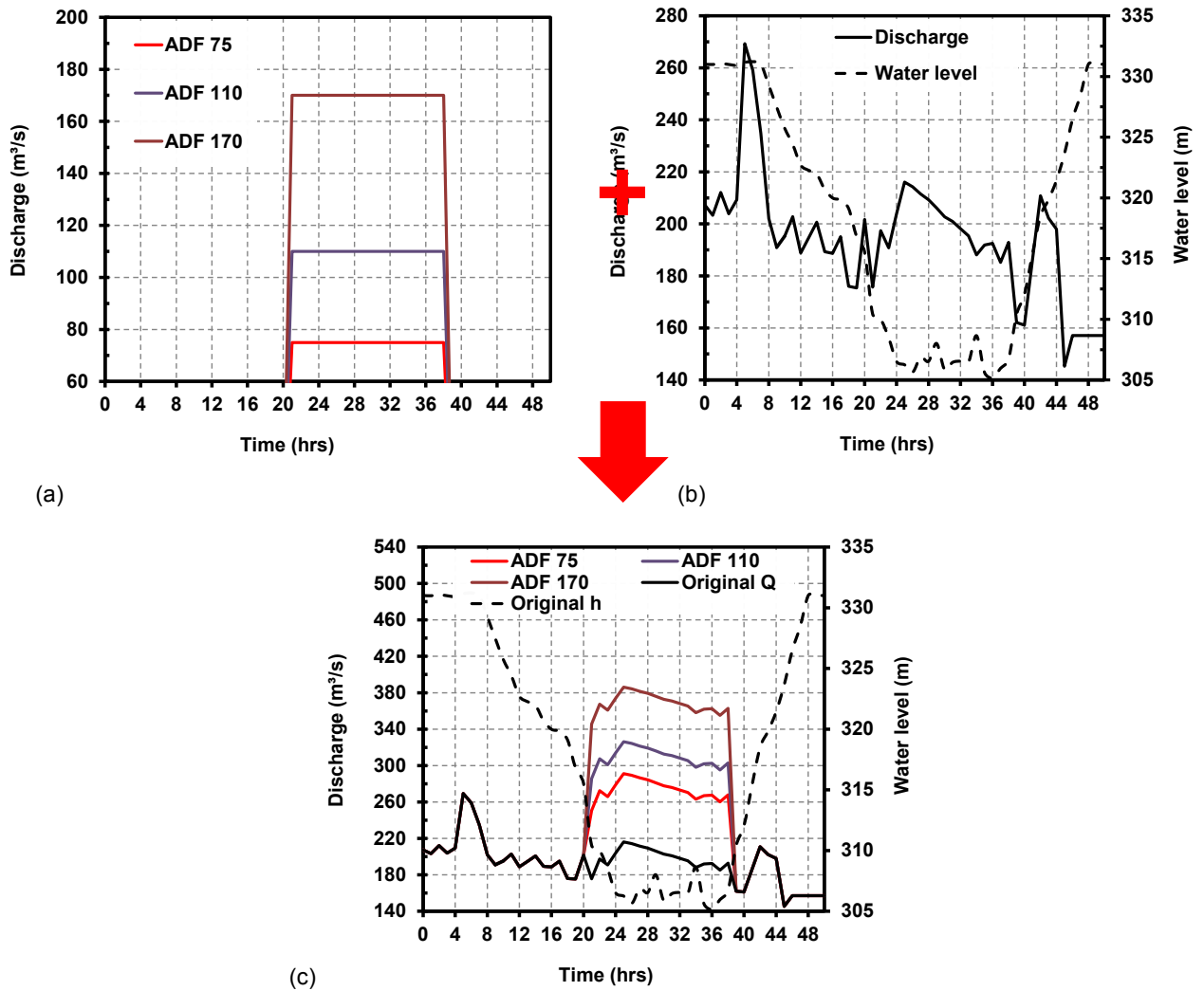


Figure 6.1 (a) constant additional discharges for different ADF scenarios in Dashidaira reservoir; (b) original water level and discharge rates during 2012 flushing in Dashidaira reservoir; (c) Water level and discharge rates for different ADF scenarios in Dashidaira reservoir.

In Figure 6.3, the simulated bed changes with and without introducing the additional discharge (i.e., under ADF 110 and ADF 170 scenario, and in reference case) have been plotted at cross-sections A-A, E-E, F-F, H-H, K-K, and L-L, respectively. The location of these cross-sections can be found in Figure 5.8. Table 6.1 shows the average bed level changes compared to the simulated reference case using *BCI* parameter in upstream, middle and downstream area for ADF 60, ADF 75, ADF 110, and ADF 170 scenarios.

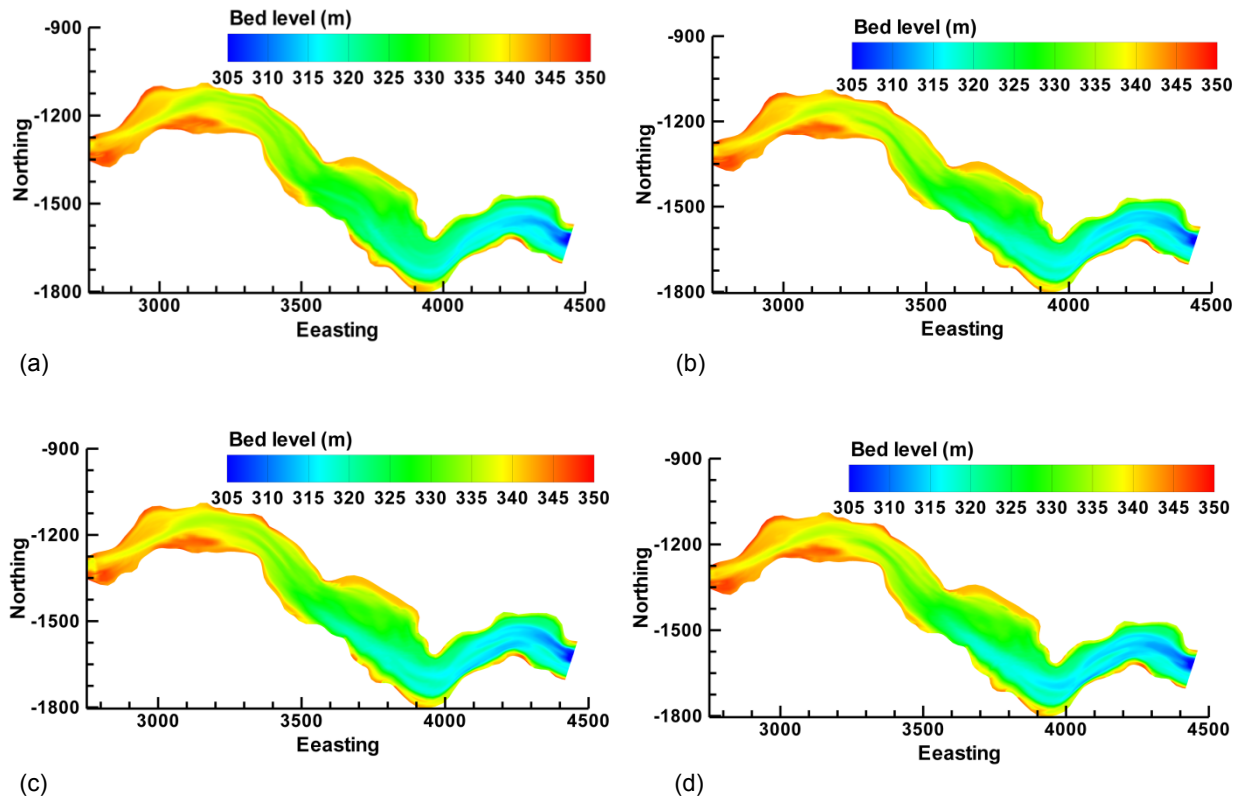


Figure 6.2 Simulated morphological bed levels under (a) the original discharge rates of 2012 flushing event; (b) ADF 75 scenario; (c) ADF 110 scenario; (d) ADF 170 scenario.

	ADF 60			ADF 75			ADF 110			ADF 170		
Parameter	I	II	III	I	II	III	I	II	III	I	II	III
BCI (m)	0	-0.03	-0.13	0.32	-0.47	-0.54	0.06	-0.49	-0.55	0.20	-0.76	-0.90
TVFS (KCM)	321.73			356.04			396.10			425		

Table 6.1 Average bed level changes in different areas of the reservoir under different ADF scenarios compared to the reference case; 1 KCM= $1 \times 10^3 \text{ m}^3$.

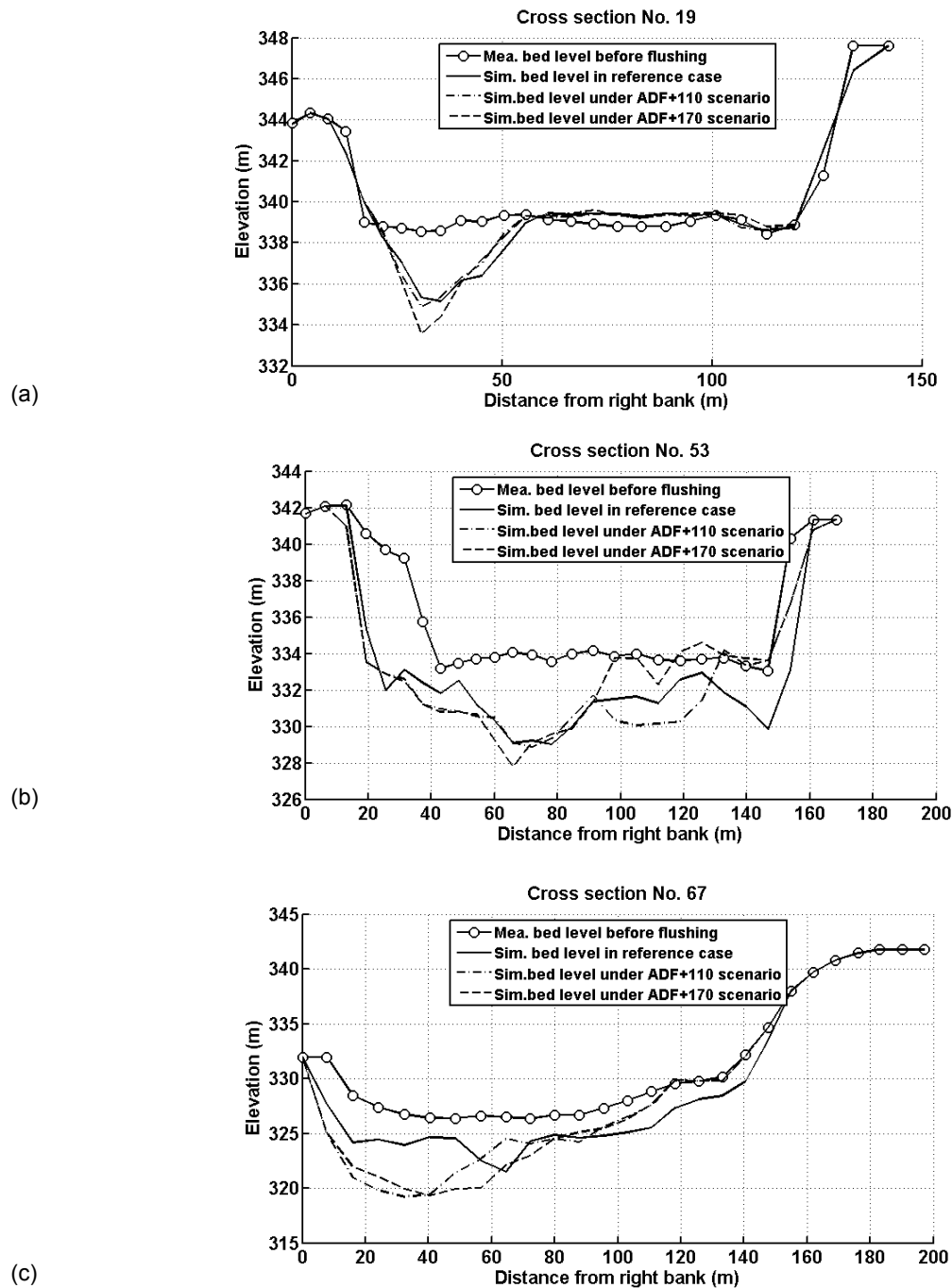
Figure 6.3 and Table 6.1 reveal that introducing an additional discharge within the free-flow condition has a marginal effect on the erosion of coarser sediments from area I while it has a further contribution in the development of already formed flushing channel in area II and III where the existing bed sediments are finer (Esmaeili et al., 2015b). This can be attributed to the major role of water level drawdown stage in locating the initial flushing channel place and the formation of the channel as revealed by experimental and numerical model studies by Kantoush and Schleiss (2009) and Esmaeili et al. (2014b). It should be noted that from the reservoir entrance up to cross section A-A position, BCI showed 0.10, 0.04 m, 0.06 m, and 0.10 cm average erosion compared to the reference case. Since BCI shows average deposition in area I, one can conclude that eroded coarser bed materials are deposited

after passing the cross section A-A on smaller fractions of bed materials due to the flow depth increase and the consequent velocity reduction. Subsequently, bed armoring process happens that prevents the smaller underneath bed materials from eroding. If this process happens in areas close to the side banks, the erosion chance of deposited coarser materials may be decreased since the flow velocity is low in these areas even when additional discharge is high (i.e., ADF 170 scenario). If deposition of coarser materials occurs in the main flushing channel, the erosion chance will increase even with a lower additional discharge (i.e., ADF 60 scenario). In addition, the inflowing discharge during the free-flow condition is concentrated in the concise flushing channel which mainly contributes in slow widening and deepening of the existing concise channel. This widening and deepening procedure is more effective in areas covered by finer sediments (e.g. area III). Thus, increasing the amount of constant additional discharge to 170 m³/s during the free-flow condition may result in a further increase in the flushing channel width and depth in the lower part of area II and in the whole area III (Esmaeili et al., 2015b).

The effect of introducing constant additional discharges under various ADF scenarios on the Flushing Efficiency (i.e., *FE*) and the Total Volume of Flushed out Sediment from the reservoir (i.e., *TVFS*), when *FE* is the ratio of the flushed out sediment volume to the used water volume, has been illustrated in Figure 6.4. Horizontal axis shows the ratio of average discharge during the free-flow stage using different ADF scenarios (i.e., Q_2) to the average discharge during the free-flow stage when no additional discharge is introduced (i.e., Q_1). This axis shows that how much average discharge should be increased to enhance the erosion (i.e., *TVFS*) in Dashidaira reservoir. FE_2 and FE_1 are the flushing efficiencies when an ADF scenario is employed and when no additional discharge is employed, respectively. Left and right vertical axes figure out the variation of *FE* and *TVFS*, respectively. It should be noted that variations of *FE* and *TVFS* in vertical axes and changes in the used water discharge during the free-flow condition in the horizontal axis were all calculated relative to the reference simulated case (i.e., 2012 flushing operation).

As can be observed from this figure, *TVFS* increases when the used discharge increases during the free-flow stage. In case of *FE* variation, it became about -6.5% when the ADF +60 scenario is used. In this case, the increase in the flushed out sediment volume was smaller than the increase in the used water volume. Then, under ADF 75, 90, and 110 scenarios both *FE* and *TVFS* variation became upward and positive according to the increase in the used average discharge during the free-flow stage. This trend continues up to reaching a maximum value. Then, *FE* variation became downward and following that it remains in the almost same level for ADF 150 and ADF 170 scenarios while *TVFS* variation kept the upward trend. This can be attributed to the non-linear relation between the water discharge and sediment entrainment rate. In such a condition increasing the discharge magnitude during the free-flow condition can increase the *TVFS*, but this increase would not be proportional to the discharge increase that causes the *FE* variation to be downward. According to the diagram shown in Figure 6.4, when the average discharge during the free-flow stage increased about 55% (i.e., $Q_2/Q_1=1.55$), the *FE* variation had the highest value for the study area of Unazuki reservoir. In such a condition, the total used water volume for the flushing operation increased about 20%. It should be

taken into account that when the average discharge during the free-flow stage increases artificially in the study case, the entering sediment load to the reservoir may increase that can lead into sediment deposition and consequently a lower TVFS and FE. Implementing the same scenario on different simulated reference cases (i.e., different years flushing operation) and making their average can be utilized to increase the accuracy of shown diagram in Figure 6.4.



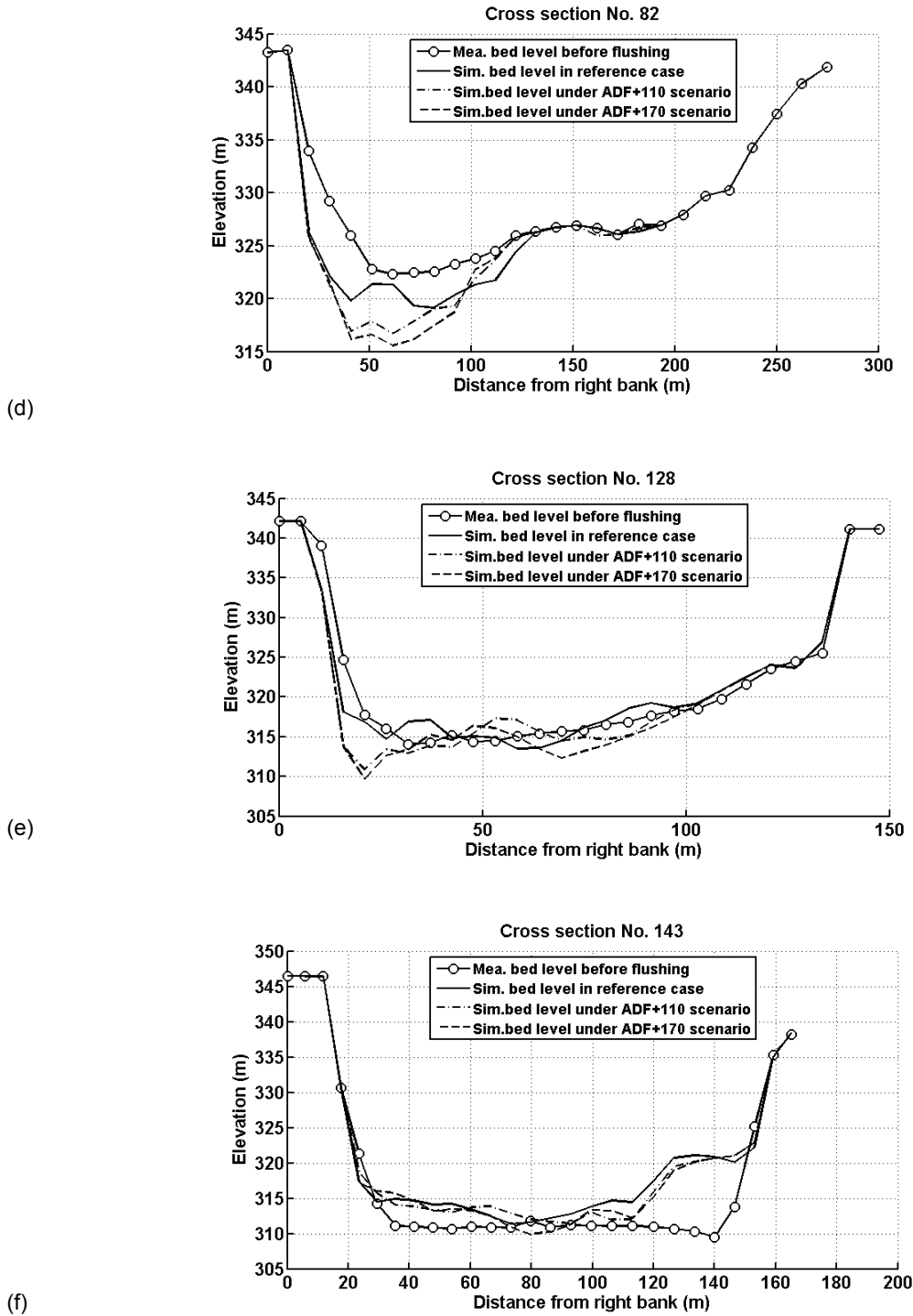


Figure 6.3 Measured bed level before flushing along with the simulated one after flushing in the reference case, under ADF 110, and under ADF 170 scenario at the location of a) cross section A-A; b) cross section E-E; c) cross section F-F; d) cross section H-H; e) cross section K-K; f) cross-section L-L. Location of the sections can be found in Figure 5.8.

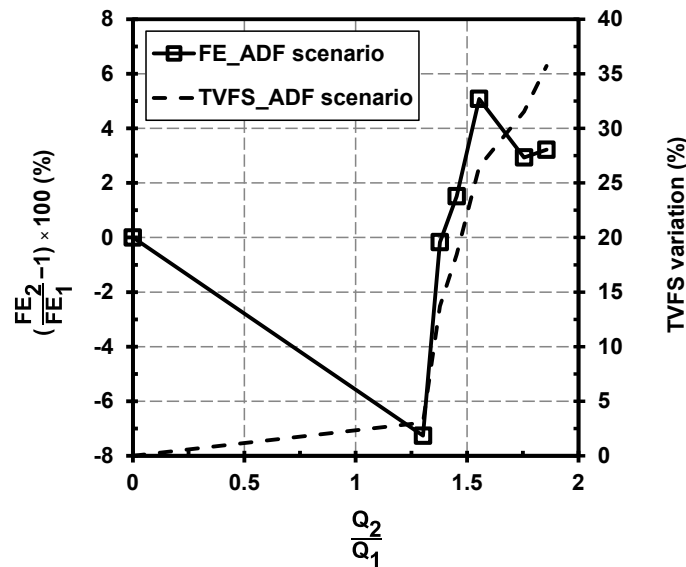


Figure 6.4 Non-dimensional curves showing the relation between TVFS, FE, and used water discharge compared to the reference case under different ADF scenarios.

Although introducing an additional discharge during the free-flow condition enables the flushing channel to develop in lateral and mainly in vertical directions in areas II and III of the reservoir, the flushing channel location is generally close to the right bank in area II and deposited materials along the left bank (i.e. dead zone) still cannot be removed effectively. Therefore, it is recommended to consider other types of soft and hard measures in the future outlook. Changing the water level drawdown rates (i.e., drawdown speed), constructing auxiliary longitudinal channels and using the artificial flow guiding structures may result in a more efficient flushing operation to recover the reservoir capacity.

6.1.2 Introducing a Pulse of Discharge during the Free-flow stage in Dashidaira reservoir (PDF scenario)

As it was mentioned in part 6.1.1, Introducing an additional inflow discharge to Dashidaira reservoir from another reservoir located upstream area is practically feasible during the free-flow flushing operation. Instead of adding a constant discharge to the original discharge values during the free-flow condition in reference case, it is also possible to keep the same used water volume and introduce the additional discharge in a shorter duration with a higher discharge magnitude in the form of a pulse of discharge.

Since the TVFS and FE was the highest in ADF 110 scenario, the total volume of the additional used water during the 18 hours of free-flow stage was calculated and this additional water volume introduced during the 16 hours and 14 hours with a constant discharge of 124 m³/s and 141.5 m³/s (i.e., PDF 124 16 scenario and PDF 141.5 14 scenario), respectively. The concept of the pulse of discharge during the free-flow stage has been illustrated schematically in Figure 6.5.

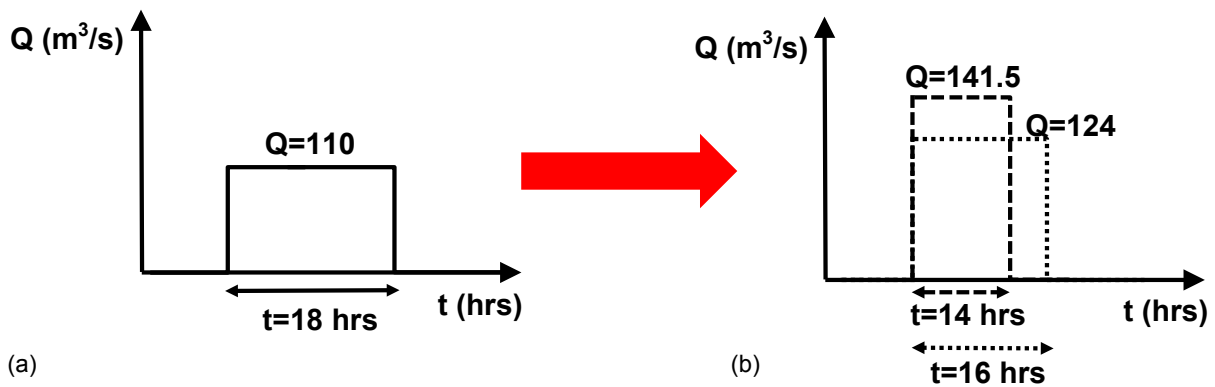


Figure 6.5 (a) Introduced additional discharge under ADF 110 scenario; (b) Schematic figure illustrating the PDF scenario in Dashidaira reservoir.

However, utilizing this scenario with the equal water volume to the ADF 110 scenario showed a smaller TVFS compared to ADF 110 scenario, namely 375.67×10^3 m³ for PDF 124 16 scenario and 374.29×10^3 m³ for PDF 141.5 14 scenario. Table 6.2 shows the average bed level changes compared to the ADF 110 scenario using BCI parameter in upstream, middle and downstream area for PDF 124 16 and PDF 141.5 14 scenarios. The TVFS values and results showed in Table 6.2 pronounces the role of time in morphological process during the flushing operation since the process includes erosion and then transportation. In other words a high discharge would be more effective to erode coarser materials if it lasts for a longer duration. Longer duration of high discharge implies a higher possibility for erosion of coarser materials and also further transporation towards the dam that is favourable for flushing out a higher volume of coarser materilas (e.g., PDF 124 16). Otherwise, the main effect of a PDF scenario would be on erosion and transportation of finer bed materials from downstream of area II and also from area III. In such a condition PDF acenario may not have a robust advantage compared to the ADF 110 scenario (e.g., PDF 124 16). It also should be noted that if coarser material can be transported further downstream area of the reservoir, bed armoring may happen in downstream areas that may decreas the finer material erosion from these areas as can be observed in PDF 124 16 scenario in Table 6.2.

	PDF 124 16			PDF 141.5 14		
Parameter	I	II	III	I	II	III
BCI (m)	0.03	0	0.18	0.29	-0.10	-0.16
TVFS (KCM)	375.67			374.29		

Table 6.2 Average bed level changes in different areas of the reservoir under different PDF scenarios compared to the ADF 110 scenario. The first number after PDF is pulse discharge magnitude in m³/s and second number is the duration of pulse discharge in hours.

Based on the outcomes revealed in Table 6.2, PDF scenario was modified and designed with two discharge pulses, first one in the first half of free-flow stage and second one in the second half. In this way the characteristics of PDF scenarios with high discharge values would be conserved and

meanwhile an enough time would be supplied. Consequently, the modified scenario was designed with a variable duration and magnitude of the first discharge pulse (i.e., P1 Q t) and 110 m³/s discharge pulse for 8 hours in the second half (i.e., P2 110 8). P1 is the first pulse discharge and P2 is the second one. The first number after P1 and P2 is pulse discharge magnitude in m³/s and second number is the duration of pulse discharge in hours. Alternatively, the constant discharge pulse in the first half (i.e., P1 110 8) and a variable type of second discharge pulse (i.e., P2 Q t) can be designed. The concept of the modified PDF scenario has been illustrated schematically in Figure 6.6.

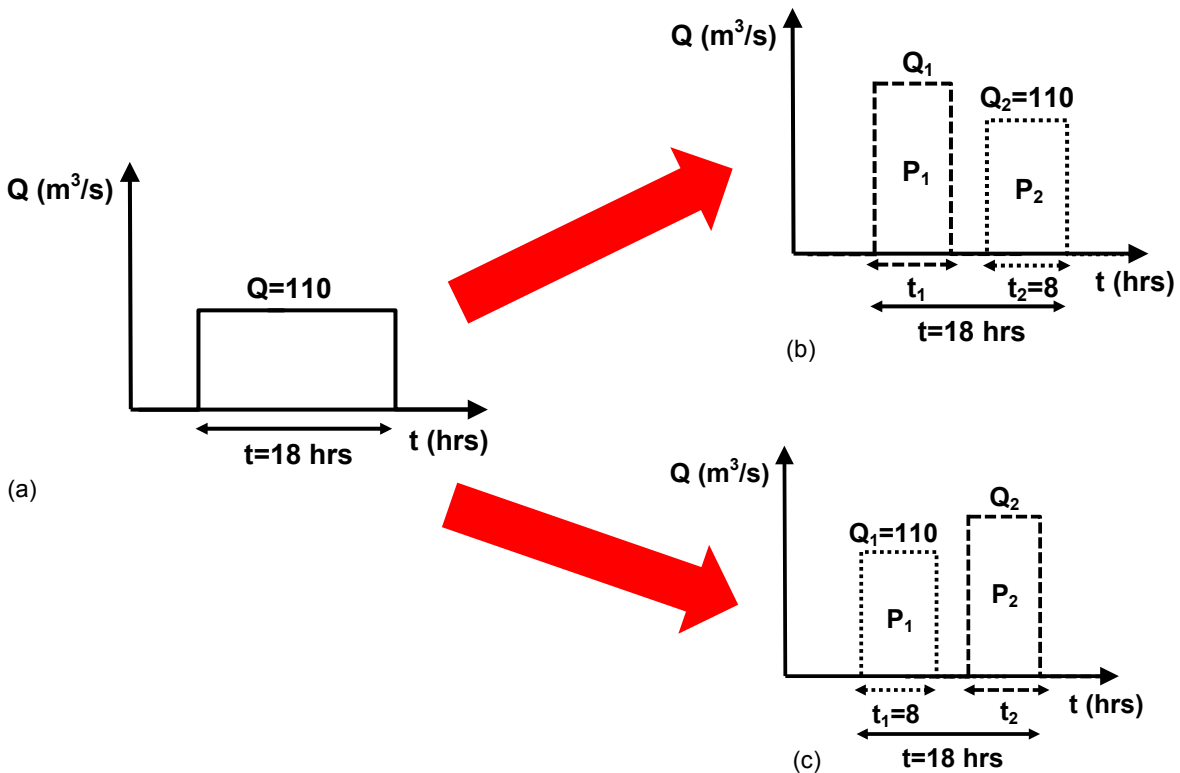


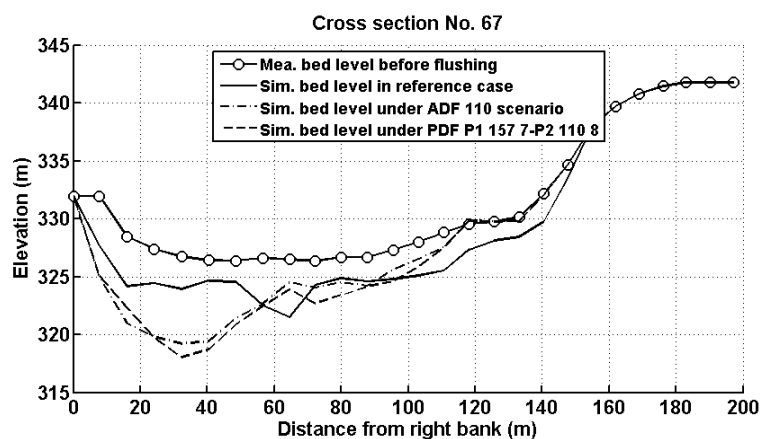
Figure 6.6 (a) Introduced additional discharge under ADF 110 scenario; (b) Schematic figure illustrating the modified PDF scenario with constant discharge pulse in the second half of free-flow stage; (c) Schematic figure illustrating the modified PDF scenario with constant discharge pulse in the first half of free-flow stage.

It should be also taken into account that used additional water volume in all modified scenarios is equal to that of used in ADF 110 scenario. Table 6.3 shows the BCI parameter compared to the ADF 110 scenario for PDF P1 137.5 8-P2 110 8, PDF P1 157 7-P2 110 8 and PDF P1 183.5 6-P2 110 8, respectively. Although using the variable first discharge pulse and fixed second discharge pulse was favourable for enhancing the TVFS through increasing the erosion from area II and III, compared to the ADF 110 scenario, it can not increase the erosion of coarser materials from area I. Moreover, a higher discharge pulse in the first half of free-flow stage may erode coarser materials from far upstream areas, but duration of the pulse should be long enough to transport the eroded materials further downstream. Otherwise, the eroded coarser materials would be deposited in the nearest deeper part where the velocity drops down (i.e., in the lower part of area I). Thus, erosion of coarser materials should be considered with some measures for further transport of these materials to reservoir downstream to

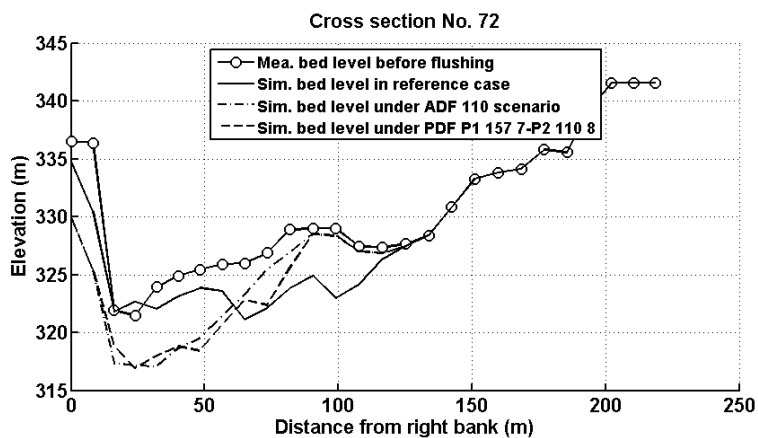
finally flush out from the reservoir. In Figure 6.7, final bed levels in area II and III under PDF P1 157 7-P2 110 8 scenarion have been illustrated and compared to the bed levels in the reference case and ADF 110 scenario to quantitatively show the performance of this scenario for eroding the finer bed materilas in area II and III.

	PDF P1 137.5 8-P2 110 8			PDF P1 157 7-P2 110 8			PDF P1 183.5 6-P2 110 8		
Parameter	I	II	III	I	II	III	I	II	III
BCI (m)	0.08	-0.07	0.02	0.06	-0.21	-0.13	0.13	-0.21	-0.10
TVFS (KCM)	393.69			418.38			409.32		

Table 6.3 Average bed level changes in different areas of the reservoir under different modified PDF scenarios with the constant discharge pulse in the second half of free-flow stage. BCI values are compared to the ADF 110 scenario.



(a)



(b)

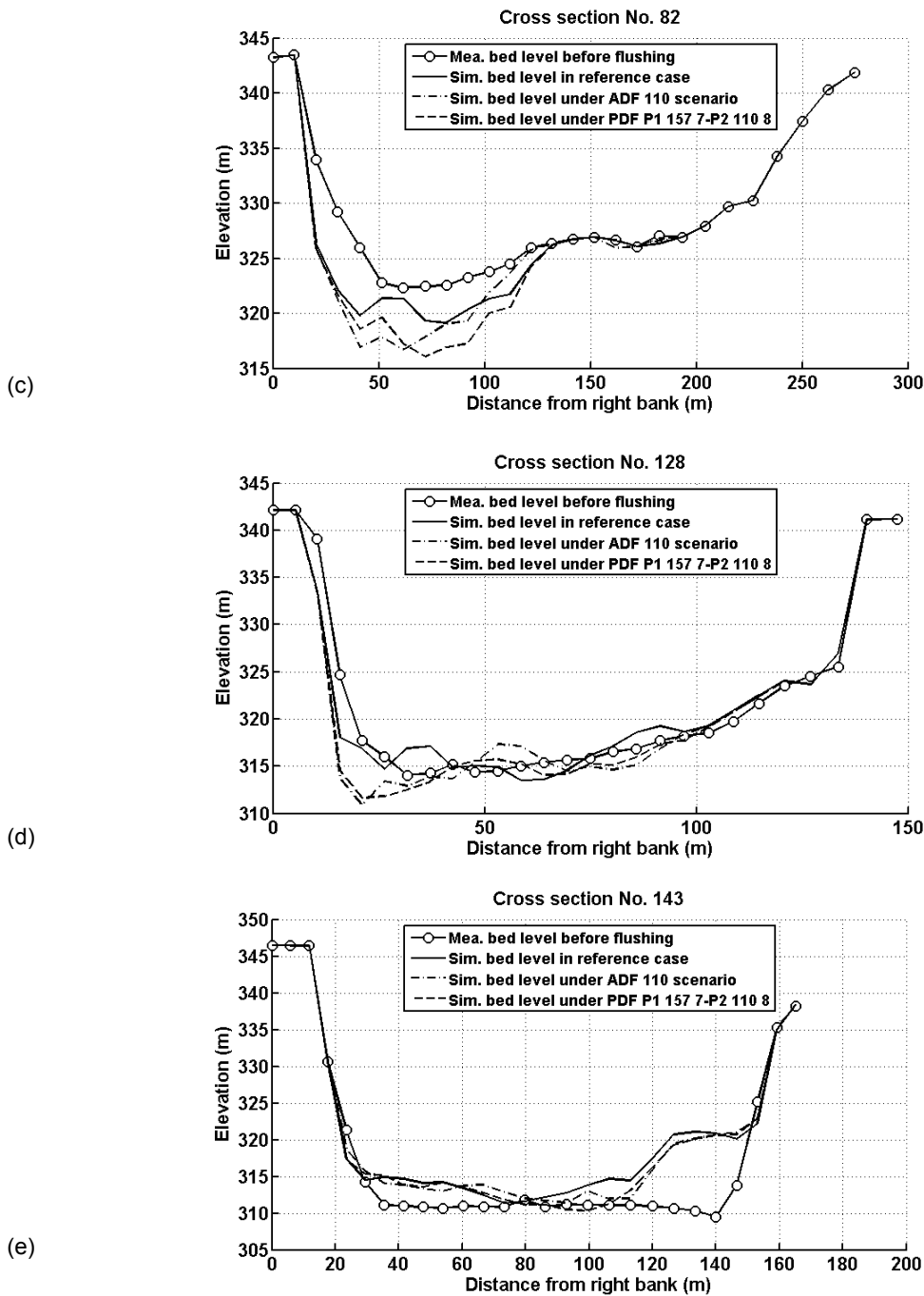


Figure 6.7 Measured bed level before flushing along with the simulated one after flushing in the reference case, under ADF 110, and under PDF P1 157 7-110 8 scenario at the location of a) cross section F-F; b) cross section G-G; c) cross section H-H; d) cross section K-K; e) cross section L-L. Location of the sections can be found in Figure 5.8.

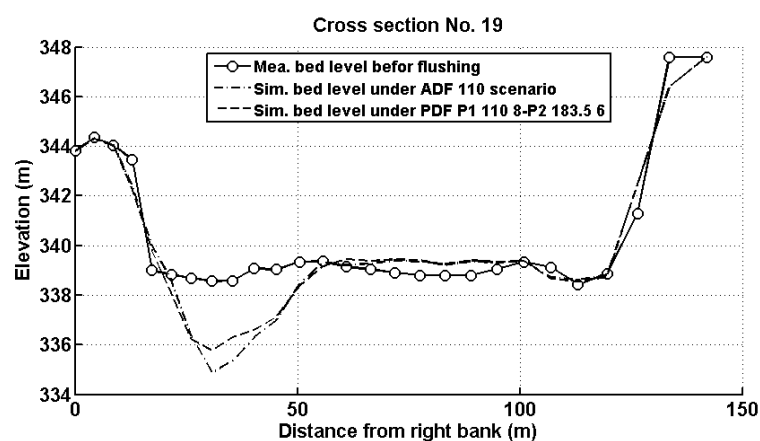
Table 6.4 shows the BCI parameter under the constant discharge pulse in the first half (i.e., P1 110 8) and a variable discharge pulse in the second half (i.e., P2 Q t). Although using the constant first discharge pulse and variable second discharge pulse was favourable for enhancing the TVFS through increasing the erosion from area II and III, compared to the ADF 110 scenario, it can increase the

erosion of coarser materials from area I just when the second discharge pulse is high enough (i.e., in PDF P1 110 8-P2 183.5 6). The reason can be attributed to the further transportation of already eroded coarser materials from area I towards downstream areas during the period of first discharge pulse.

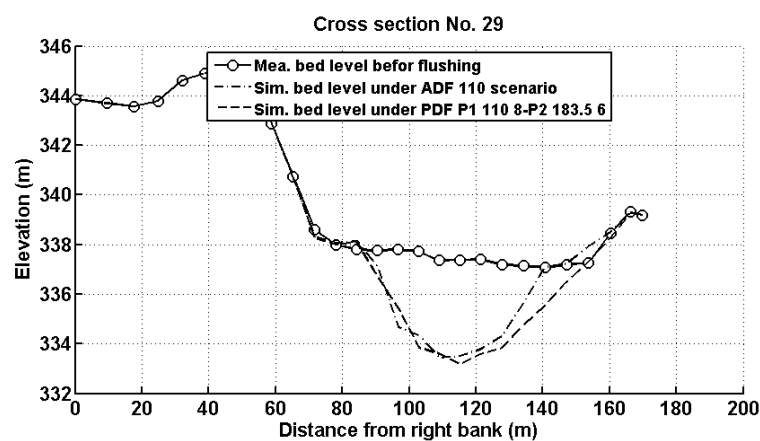
In Figure 6.8, final bed levels in area I under PDF P1 110 8-P2 183.5 6 scenario have been illustrated and compared to the bed levels using ADF 110 scenario to quantitatively show the performance of this scenario for eroding the coarser bed materilas from area I.

	PDF P1 110 8-P2 137.5 8			PDF P1 110 8-P2 157 7			PDF P1 110 8-P2 183.5 6		
Parameter	I	II	III	I	II	III	I	II	III
BCI (m)	-0.01	-0.09	-0.21	0.02	-0.04	-0.20	-0.09	0.01	0.04
TVFS (KCM)	426.24			417.33			410.85		

Table 6.4 Average bed level changes in different areas of the reservoir under different modified PDF scenarios with the constant discharge pulse in the first half of free-flow stage. BCI values are compared to the ADF 110 scenario.



(a)



(b)

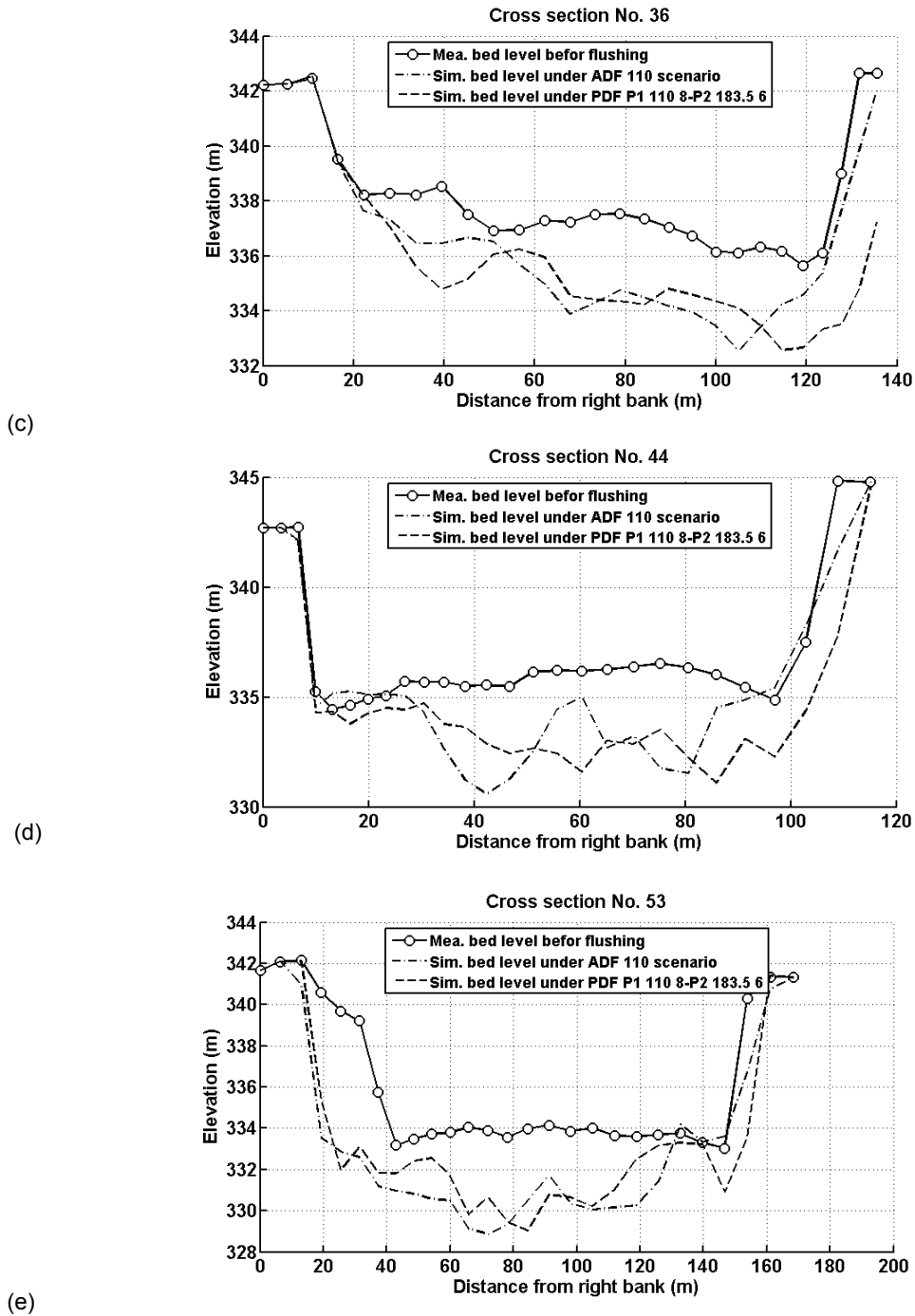


Figure 6.8 Measured bed level before flushing along with the simulated one after flushing under ADF 110, and PDF P1 110 8-P2 183.5 6 scenario at the location of a) cross section A-A; b) cross section B-B; c) cross section C-C; d) cross section D-D; e) cross section E-E. Location of the sections can be found in Figure 5.8.

6.1.3 Introducing an Additional constant Discharge during the Drawdown stage in Dashidaira reservoir (ADD scenario)

In ADD scenario, a constant additional discharge is added to the original discharge values for 14 hours during the drawdown stage (i.e., between 7 to 20 hours after starting the flushing operation) in the reference case while the original water level of reference case remains unchanged. This period was

selected since the drawdown and free-flow stage starts 7 and 20 hours after starting the fishing operation, respectively. For example, ADD 75 scenario represents the application of modified discharge rates by introducing a constant 75 m³/s additional discharge during the mentioned period. In this part, effects of different ADP scenarios on TVFS and bed level changes are evaluated numerically in Dashidaira reservoir. This is performed through comparing the numerical outcomes under different ADP scenarios with the numerical results available from the reference casse (i.e., calibrated model) for Dashidaira resevoir. However, application of this approach may be challenging practically since the water level is still high during the drawdown stage and additional discharge may further increase the water level that implies a risky condition. Figure 6.9 illustrates the original water level and Discharge rates together with discharge modifications for different ADP scnarios in Dashidaira reservoir.

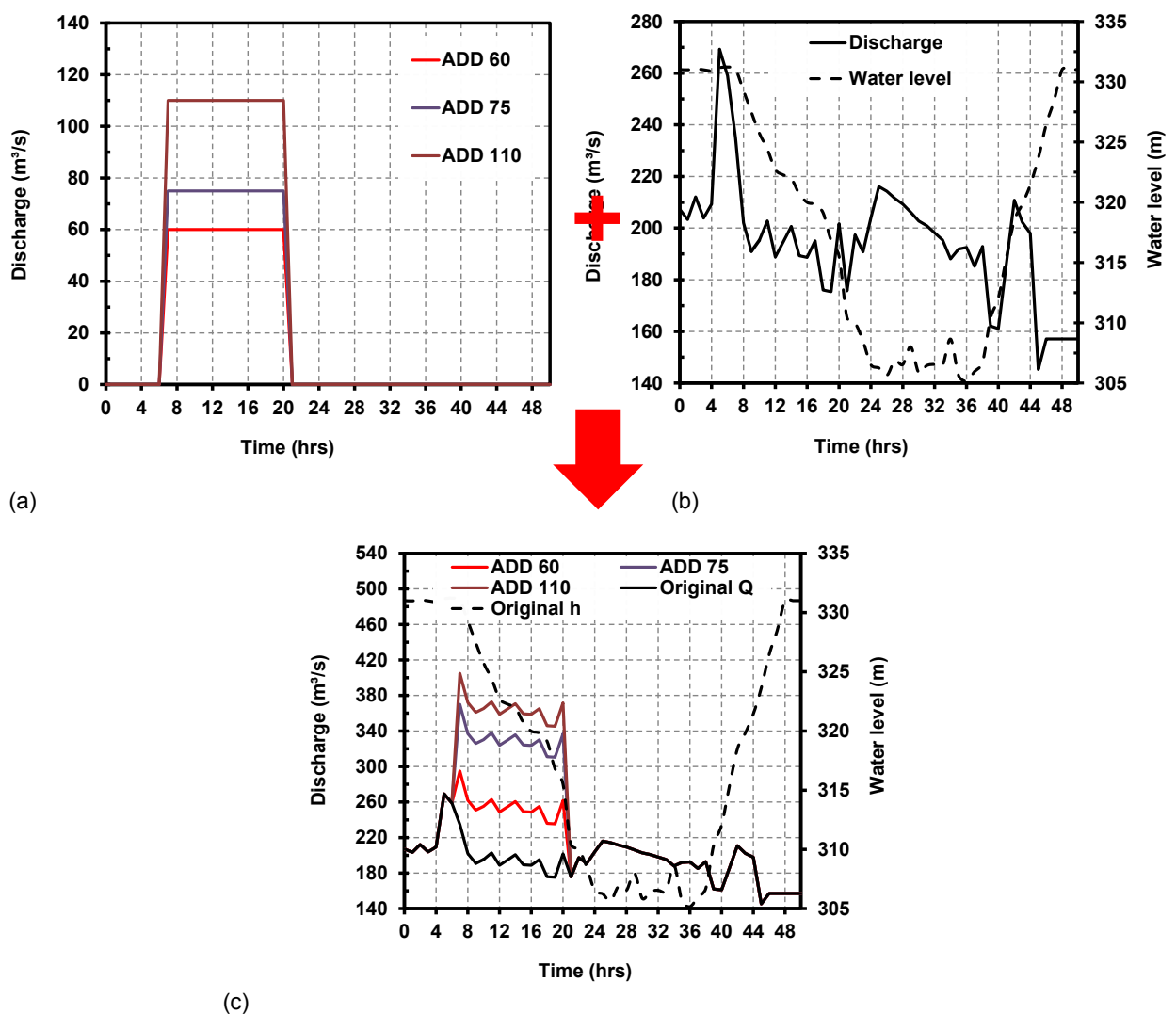


Figure 6.9 (a) constant additional dischrgees for differen ADD scenarios in Dashidaira reservoir; (b) original water level and discharge rates during 2012 flushing in Dashidaira reservoir; (c) Watre level and discharge rates for different ADF scenarios in Dashidaira reservoir.

Similart to the ADF scenarios, introducing an additional discharge during the drawdown stage has a marginal effect on the erosion of coarser sediments from area I and it has a further contribution in ersion of finer materials from area II and III. Likewise the ADF scenario, if deposition of coarser materilas occurs in the main flushing channel, the erosion chance may increase even with a lower additional discharge (i.e., ADD 60 scenario). In ADD scenario, beside the increase in flow discharge the water level also decreases, which can have a further contribution in eroding and transporting the coarser bed materials since the relative bed roughness magnitudes (i.e., ratio of roughness hight to the water depth) changes. This parameter plays a major role in erosion of bed materials (Morris & Fan, 1998). Table 6.5 shows the average bed level changes compared to the simulated reference case using *BCI* parameter in upstream, middle and downstream area under ADD 60, ADD 75, and ADD 110 scenarios.

	ADD 60			ADD 75			ADD 110		
Parameter	I	II	III	I	II	III	I	II	III
BCI (m)	-0.04	-0.11	0.06	0.01	-0.14	0.35	0.09	-0.56	-0.21
TVFS (KCM)	321.73			329.86			374.36		

Table 6.5 Average bed level changes in different areas of the reservoir under different ADD scenarios compared to the reference case.

6.1.4 Introducing an Additional constant Discharge during the Free-flow stage in Unazuki reservoir (ADF scenario)

In this part, effects of different ADF scenarios on TVFS and bed level changes using different ADF scenarios are evaluated numerically in Unazuki reservoir. This is performed through comparing the numerical outcomes under different ADF scenarios with the numerical results available from the reference casse (i.e., calibrated model) for Dashidaira resevoir. Figure 6.10 illustrates the original water level and Discharge rates together with discharge modifications for different ADF scnarios in Unazuki reservoir.

In Figure 6.11, calculated bed changes of reference case using the original discharge rates of 2012 flushing event and calculated bed changes under ADF 150, ADF 250, and ADF 300 scenarios have been depicted. As can be observed from Figure 6.11, introducing the additional discharge contributed in mobilization and redeposition of bed materials within upstream zones of area I and formation of scouring channels in area II. In the study area of reference case (i.e., calibrated model), $7.22 \times 10^3 \text{ m}^3$ sediment deposition was computed. The sediment deposition volume was about $8.2 \times 10^3 \text{ m}^3$ under ADF 150 scenario. Under this scenario, two narrow and deep scouring chanelns appear along the left and right bank of the study area while there is no a serious local scouring in other places. However, the introduced additional discharge caused a thin layer to erode from the lower half of area II due to the existing smaller sediments with a lower critical shear stress for erosion. When the introduced addditional discharge further increases (e.g., under ADF 250 and ADF 300 scenario), the geometry of

scouring channels along the right and left banks develops and meanwhile new scouring channels establish along the centerline of area I and over the downstream half of area II. Therefore, under ADF 250 and ADF 300 scenario, TVFS was about 45.84×10^3 and 55.81×10^3 m³, respectively.

In Figure 6.12, the simulated bed levels with and without introducing the additional discharge (i.e., under ADF 150 and ADF 300 scenario, and in the reference case) have been plotted at cross-sections A-A, B-B, C-C, D-D, E-E, F-F, G-G and H-H, respectively. The location of these cross-sections can be found in Figure 5.17. Table 6.6 shows the average bed level changes compared to the simulated reference case using *BCI* parameter in segments between the cross sections for ADF 150 and ADF 250 scenarios.

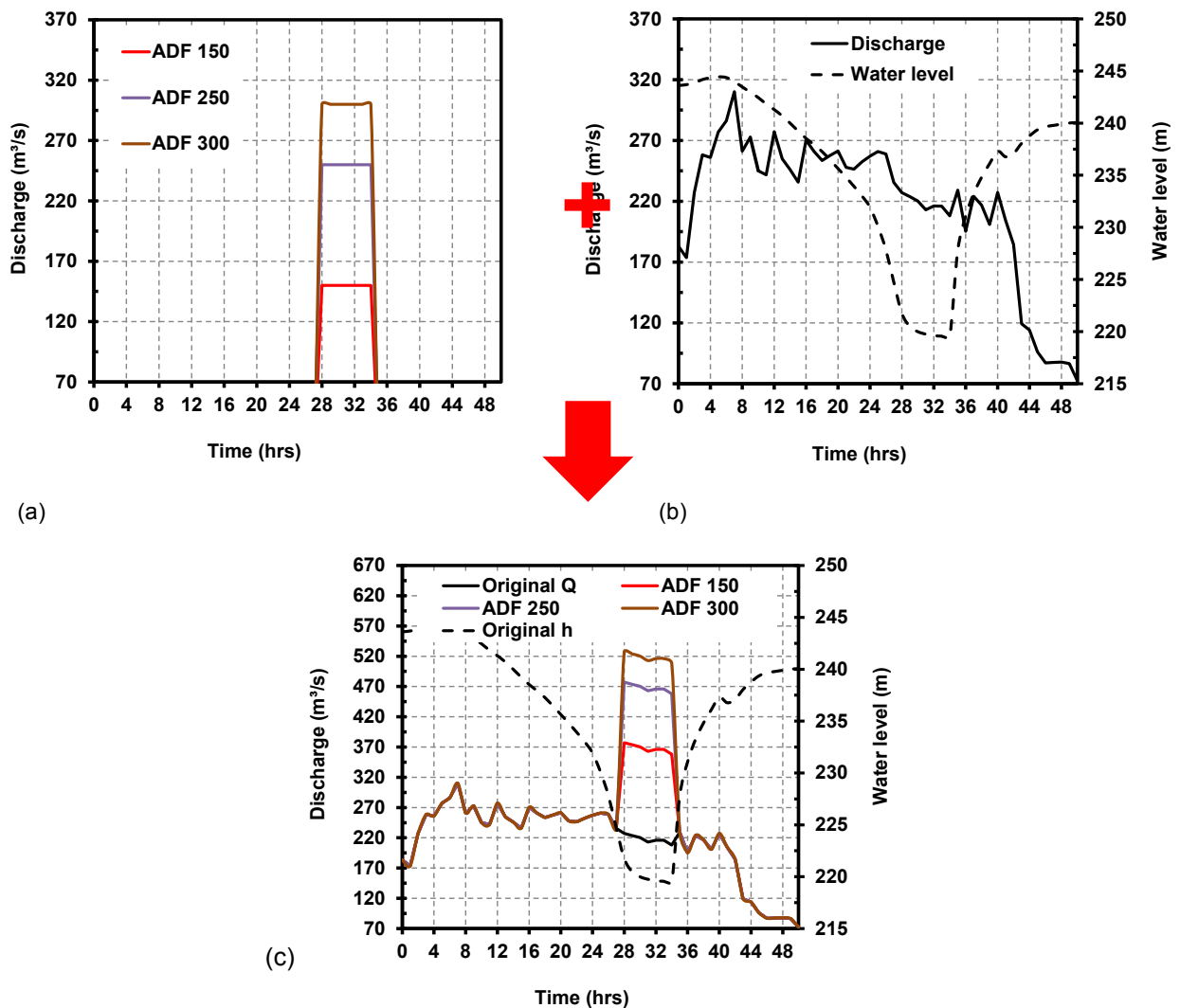


Figure 6.10 (a) constant additional discharges for differen scenarios; (b) original water level and discharge rates during 2012 flushing in Unazuki reservoir; (c) Watre level and discharge rates for different ADF scenarios in Unazuki reservoir.

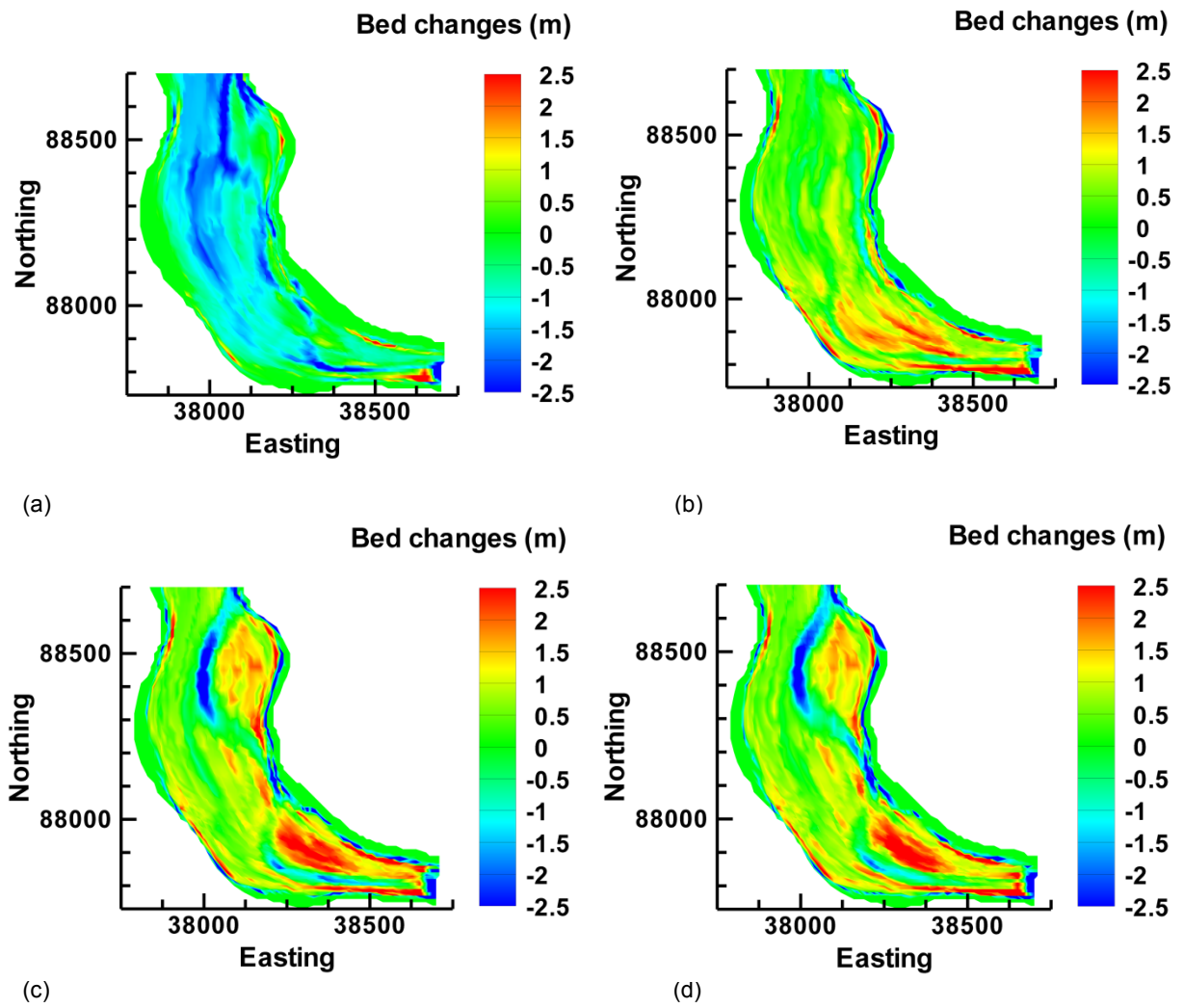
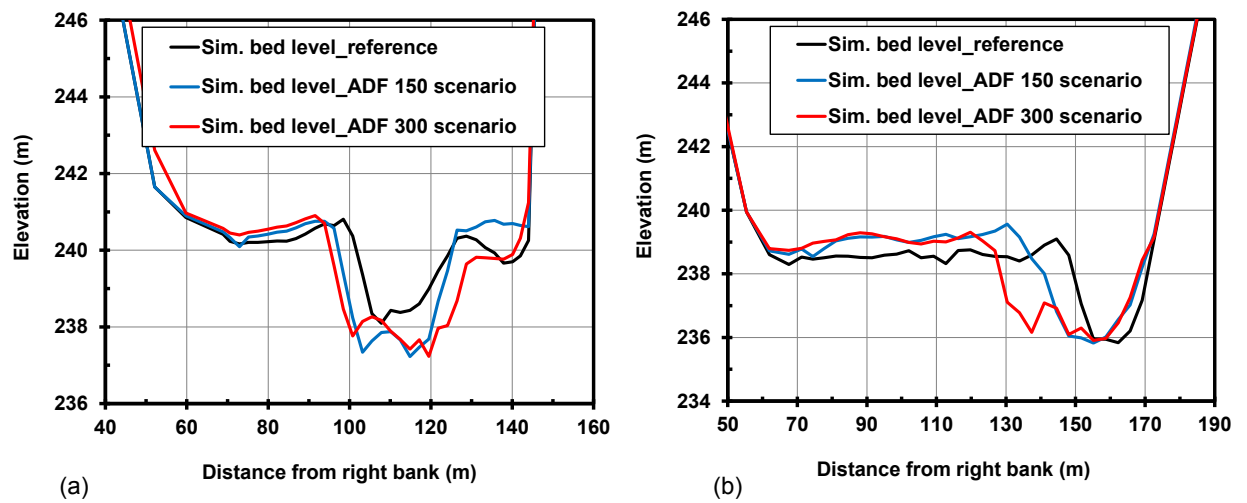


Figure 6.11 Plan view of calculated bed changes in Unazuki reservoir corresponding to (a) reference case without using additional discharge; (b) ADF 150 scenario; (c) ADF 250 scenario; (d) ADF 300 scenario.



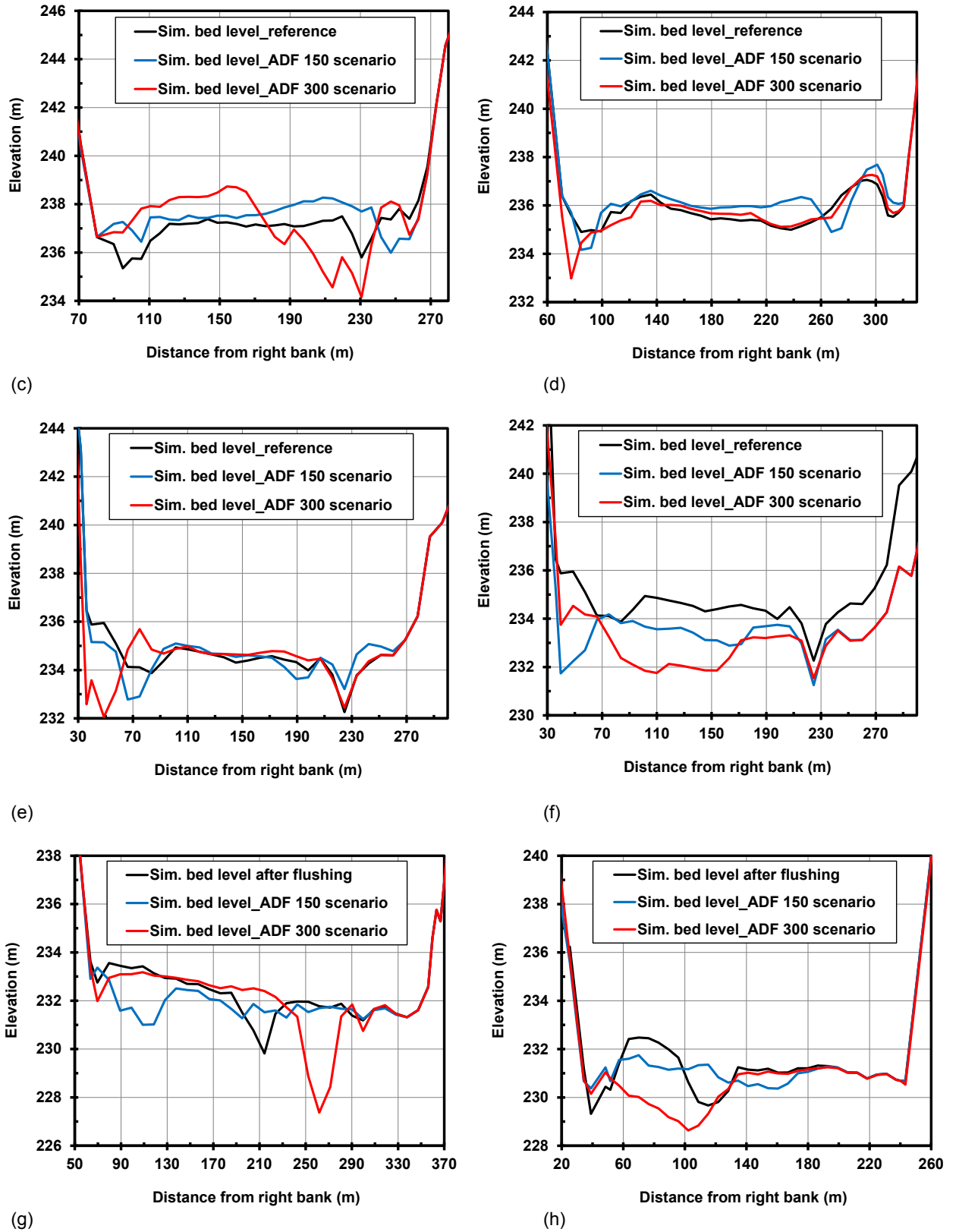


Figure 6.12 Simulated bed levels after flushing in the reference case, after flushing under ADF 150 and ADF 170 scenarios at the location of a) cross section A-A; b) cross section B-B; c) cross section C-C; d) cross section D-D; e) cross section E-E; f) cross-section F-F; g) cross section G-G; h) cross section H-H. Location of the sections can be found in Figure 5.17.

Figure 6.12 and Table 6.6 reveal that when the additional discharge during the free-flow stage is not high enough (e.g., under ADF 150 scenario), sediment erosion just occurs locally at the entrance zone (i.e., before cross section A-A). Then, the eroded sediment deposits just further downstream of area I, in the inner part of the channel bending, where the flow velocity also decreases because of the reservoir geometry (i.e., gradual widening of the channel). Similarly, the entering coarser fractions of sediments into the reservoir during the free flow stage would be deposited in area I. These transported coarser sediments are deposited on smaller fractions of sediments in area I and beyond this area due to the mentioned velocity reduction. Subsequently, bed armoring happens that prevents the smaller underneath bed materials from eroding in the lower half of area I and also upper half of area II. In addition, the mentioned processes (i.e., sediment deposition and bed armoring) may result in dcreasing the bed slope in area I and reducing the induced bed shear stress on the deposited sediments, which lead into simulation of an excessive sediment deposition in this area. Likewise Figure 6.11(b), Figure 6.12 clearly illustrates the formation of two narrow scouring channels along the right nad left bank of the study area under ADF 150 scenario. Once the quantity of introduced additional discharge is high enough (e.g., under ADF 250 or ADF 300 scenario), erosion can take place in both area I and II although the magnitude of erosion would be generally higher in area II. Introducing an additional discharge within the free-flow condition has a marginal effect on the erosion of coarser sediments from area I if its magnitude is not significantly high (e.g., higher than about 70% of free-flow discharge in the reference case). More specifically, Table 6.6 shows that erosion propagates further downstream of area I (i.e., segments between cross sections located in the lower half of area I) when the magnitude of additional discharge increases during the free-flow stage. The inflowing discharge during the free-flow stage is concentrated in the already formed concise scouring channels, which mainly contributes in deepening and slow widening of these channels. This deepening and widening procedure is more effective in areas covered by smaller sized sediments (e.g., area II).

Area	segment	ADF 150		ADF 250	
		BCI (m)	TVFS (KCM)	BCI (m)	TVFS (KCM)
I	Before A-A	-0.09	-8.2	-0.48	+45.84
	Between A-A & B-B	0.03		-0.15	
	Between B-B & C-C	0.22		0.07	
	Between C-C & D-D	0.23		-0.05	
II	Between D-D & E-E	0.02		-0.23	
	Between E-E & F-F	0		0	
	Between F-F & G-G	-0.10		-0.11	
	Between G-G & H-H	-0.17		-0.19	

Table 6.6 Average bed level changes in different segments of Unazuki resevoir under different ADF scenarios compared to the refernce case (i.e., calibrated model). Negative and positive sign of TVFS means deposition and erosion, respectively.

The effect of introducing constant additional discharges under various ADF scenarios on the Flushing Efficiency (i.e., FE) and the Total Volume of Flushed out Sediment from the reservoir (i.e., TVFS), when FE is the ratio of the flushed out sediment volume to the used water volume, has been illustrated in Figure 6.13. Horizontal axis shows the ratio of average discharge during the free-flow stage using different ADF scenarios (i.e., Q_2) to the average discharge during the free-flow stage when no additional discharge is introduced (i.e., Q_1). This axis shows that how much average discharge should be increased to enhance the erosion (i.e., TVFS) in the study area of Unazuki reservoir. FE_2 and FE_1 are the flushing efficiencies when an ADF scenario is employed and when no additional discharge is employed, respectively. Left and right vertical axes figure out the variation of FE and TVFS, respectively. It should be noted that variations of FE and TVFS in vertical axes and changes in the used water discharge during the free-flow condition in the horizontal axis were all calculated relative to the reference simulated case (i.e., 2012 flushing operation).

As can be observed from Figure 6.13, change from deposition mode to the erosion mode in the study area of the Unazuki reservoir occurs when the average discharge during the free-flow stage increased about 75%. This means that total used water volume for the flushing operation should be increased about 11% in Unazuki reservoir. Afterwards, both FE and TVFS increased sharply, according to the increase in the used average discharge during the free-flow stage up to reaching a maximum value. Then, FE and TVFS variation became downward maybe because of the higher erosion of coarser bed materials from the upstream half of area I and subsequently deposition in further downstream areas (i.e., lower half of area I or upper half of area II) that can result in the bed armoring process. According to the diagram shown in Figure 6.13, when the average discharge during the free-flow stage increased about 140% (i.e., $Q_2/Q_1=2.4$), the FE variation had the highest value for the study area of Unazuki reservoir. In such a condition, the total used water volume for the flushing operation increased about 19%. It should be taken into account that when the average discharge during the free-flow stage increases artificially in the study case, the entering sediment load to the reservoir may increase that can lead into sediment deposition and consequently a lower TVFS and FE. Implementing the same scenario on different simulated reference cases (i.e., different years flushing operation) and making their average can be utilized to increase the accuracy of shown diagram in Figure 6.13.

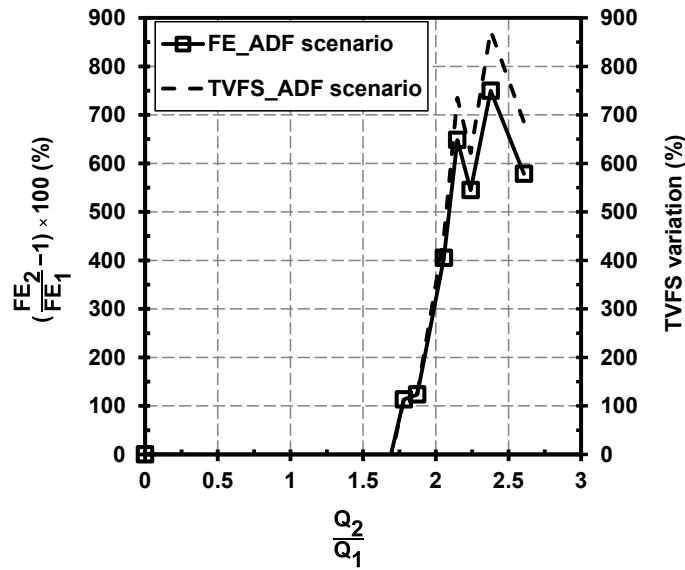


Figure 6.13 Non-dimensional curves showing the relation between FE and TVFS variation, with respect to the used water discharge under ADF scenario. Q_1 and Q_2 are average discharge during the free-flow stage in the reference case and under ADF scenarios, respectively.

6.1.5 Introducing a Pulse of Discharge during the Free-flow stage in Unazuki reservoir (PDF scenario)

Instead of adding a constant discharge to the original discharge values during the free-flow condition in the reference case under different ADF scenarios, it is also possible to keep the same used water volume and introduce the additional discharge in a shorter duration with a higher discharge magnitude in the form of discharge pulses as shown in Figure 6.5 and Figure 6.6.

Due to a better performance of the modified PDF scenario with the constant pulse of discharge in the first half of free-flow stage and variable pulse of discharge in the second half, in Dashidaira reservoir, the same scenario was applied for the study area of Unazuki reservoir. Since the TVFS and FE was the highest in ADF 300 scenario, the totoal volume of the additional used water during the 7 hours of free-flow stage was calculated and this additional water volume introduced under the modified PDF scenarios.

Table 6.7 shows the BCI parameter under the constant discharge pulse in the first half (i.e., P1 300 3) and a variable discharge pulse in the second half (i.e., P2 Q t). BCI parametres are compared to the ADF 300 scenario with the TVFS equals to 55.8 KCM. As can be observed from the table, employed scenarios could not contribute in increasing the TVFS compared to the ADF 300 scenario. However, modified PDF scenario with a higher magnitude of discharge pulse in a shorter period (i.e., P2 600 2) had a better effect on TVFS compared to the scenario with a lower magnitude of discharge pulse in a longer period (i.e., P2 400 3). The reason can be attributed to a higher erosion in the entrance zone of area I (i.e., segment behind the cross section A-A) and the lower half of area II. On the other side, the eroded coarser materials from the entrance zone of area I are deposited in the lower half of this area. This means that coarser materials of area I can be moved forward slightly before redeposition that is favorable, but the bed slope may be decreased over the area where redeposition occurs.

Consequently, erosion of these newly deposited materials may become difficult due to a higher magnitude of the required effective bed shear stress. Nevertheless, when an additional discharge is introduced to the reservoir in a real operation, the waterlevel will increase simultaneously that may facilitate the bed material erosion from the existing flood-plains in the reservoir. Besides, releasing a high discharge from the reservoir to the downstream river channel may result in increasing the watre level that can contribute in washing the deposited materials caused by the sediment flushing from the main channel and flood plains. However, the capacity limitation of the bottom outlets of Unazuki reservoir beisde the unknown effects of releasing a very high discharge in a short period makes the application of this approach practically challenging. It is also necessary to perform a study on the potential environmental impacts of such scenarios on the downstream areas.

Area	segment	PDF P1 300 3-P2 400 3		PDF P1 300 3-P2 600 2	
		BCI (m)	TVFS (KCM)	BCI (m)	TVFS (KCM)
I	Before A-A	0.02	+43.6	-0.25	+55.4
	Between A-A & B-B	0.07		0.24	
	Between B-B & C-C	0.09		0.16	
	Between C-C & D-D	0.03		0.10	
II	Between D-D & E-E	0.14		0.11	
	Between E-E & F-F	0.02		0	
	Between F-F & G-G	-0.07		-0.18	
	Between G-G & H-H	0.05		0	

Table 6.7 Average bed level changes in different segments of Unazuki resevoir under different modified PDF scenarios with the constant discharge pulse in the first half of free-flow stage. BCI values are compared to the ADF 300 scenario and the positive sign of TVFS means erosion.

In Figure 6.14, final bed levels in the area I of the Unazuki reservoir under the PDF P1 300 3-P2 600 2 have been illustrated and compared to the bed levels using ADF 300 scenario to quantitatively show the performance of this scenario for eroding the coarser bed materilas from the segment behind the cross-section A-A.

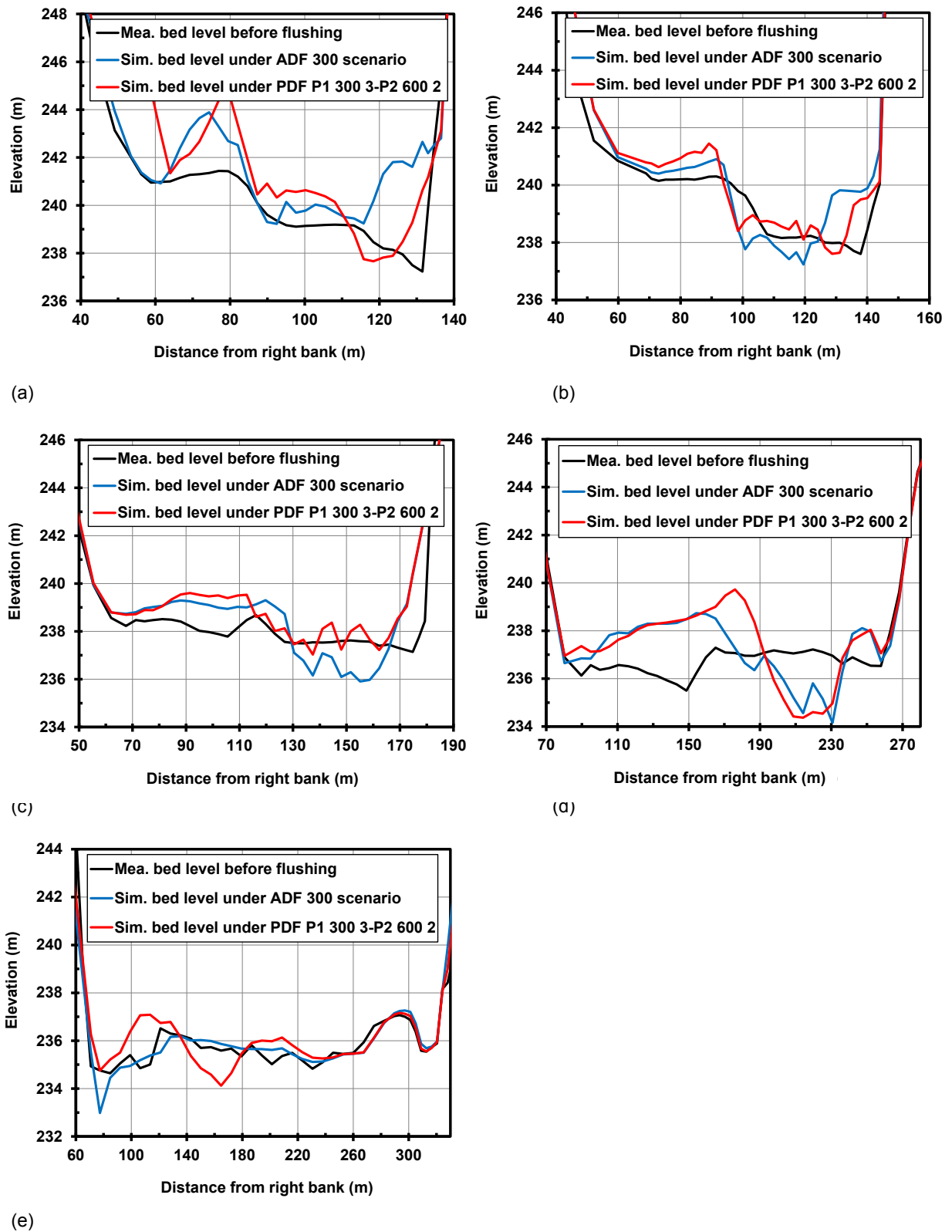


Figure 6.14 Measured bed level before flushing along with the simulated one after flushing under ADF 300, and PDF P1 300 3-P2 600 2 scenario at the location of a) just entrance of the reservoir; b) cross section A-A; c) cross section B-B; d) cross section C-C; e) cross section D-D. Location of the sections can be found in Figure 5.17.

6.2 Water level scenario

In this section, the effect of increasing the water level drawdown speed through introducing an extra water level lowering (i.e., extra drop) for one hour and then keeping the original drawdown rates during the remaining time was evaluated. The purpose was to investigate whether this type of so-called soft measure can increase the flushing efficiency or not and to assess its effect on morphological changes.

6.2.1 Increasing the Water level Drawdown Speed in Dashidaira reservoir (WDS scenario)

In WDS scenario, a target segment during the second half of drawdown stage (i.e., between $t=12$ and 20 hrs after starting the flushing) in the water level variation curve is selected. Then, the original water level drawdown rate is increased significantly for 1 hour time step at the beginning of the segment (i.e., between $t=12$ and 13 hrs after starting the flushing). Afterwards, original drawdown rates are kept unchanged during the remaining time (i.e., from $t=13$ hrs until $t=20$ hrs). As a result, introducing an extra drop in the water level (e.g., 0.5 m, 2.5 m, and 3.5 m) for one hour while the original discharge rates are unchanged causes the velocity to increase abruptly that acts as a strong driving force over the bed materials. For example, WDS -0.5 scenario represents an 0.5 m extra drop in the water level at the beginning of the mentioned segment. This scenario can be performed with gate operations during a flushing event (e.g., temporary increasing the gate opening speed) or with modification of the bottom outlets geometry (i.e., changing the size and location of the bottom outlets). Nevertheless, application of this scenario may be practically and environmentally crucial. From the practical point of view, there would be some limitations for increasing the drawdown speed. First, constraints of mechanical facilities for a safe and quick opening of the gate to increase the drawdown speed. Second, limitations of the discharge capacity of existing bottom outlets that confines the flushed out volume of sediment-laden flow when the gate opening speed increases. Third, some reservations because of the probable sand slide in the reservoir caused by an extra water level drop. From the environmental point of view, increasing the drawdown speed through increasing the gate opening rate will enhance the sediment erosion volume while the used water volume remains constant. This event results in hyper concentration of sediments in the released water-sediment mixture that could be harmful to the habitat structures and aquatic animals life downstream of the dam.

In this part, effects of different WDS scenarios on TVFS and bed level changes are evaluated numerically in Dashidaira reservoir. This is performed through comparing the numerical outcomes under three WDS scenarios with the numerical results available from the reference case (i.e., calibrated model) for Dashidaira reservoir. Figure 6.15 illustrates the original water level and Discharge rates together with water level modifications, in the target segment, for different WDS scenarios in Dashidaira reservoir.

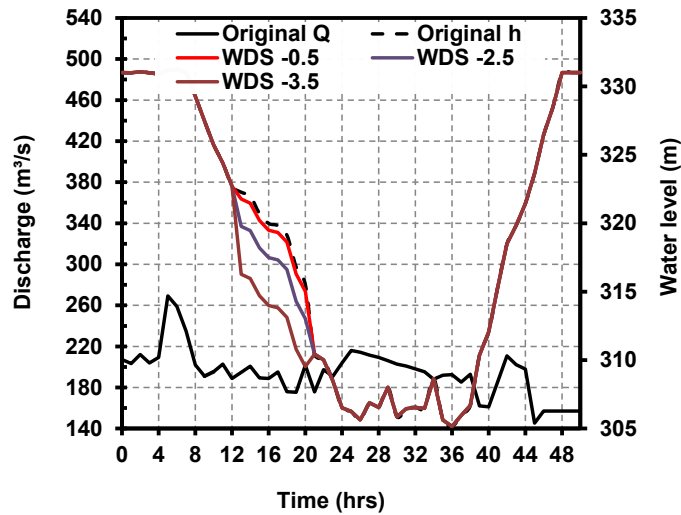


Figure 6.15 Utilized water level and discharge rates for different WDS scenarios.

In Figure 6.16, calculated bed levels of reference case using the original discharge rates of 2012 flushing event and calculated bed levels under WDS -0.5 and WDS -3.5 scenarios have been depicted. The TVFS increased from $313.14 \times 10^3 \text{ m}^3$ in the reference case to $322.78 \times 10^3 \text{ m}^3$ under WDS -0.5, and to 378.85×10^3 under WDS -3.5 scenario. Table 6.8 also shows the average bed level changes compared to the simulated reference case using *BCI* parameter in upstream, middle and downstream area of Dashidaira reservoir after application of WDS -0.5, WDS -2.5, and WDS -3.5 scenarios.

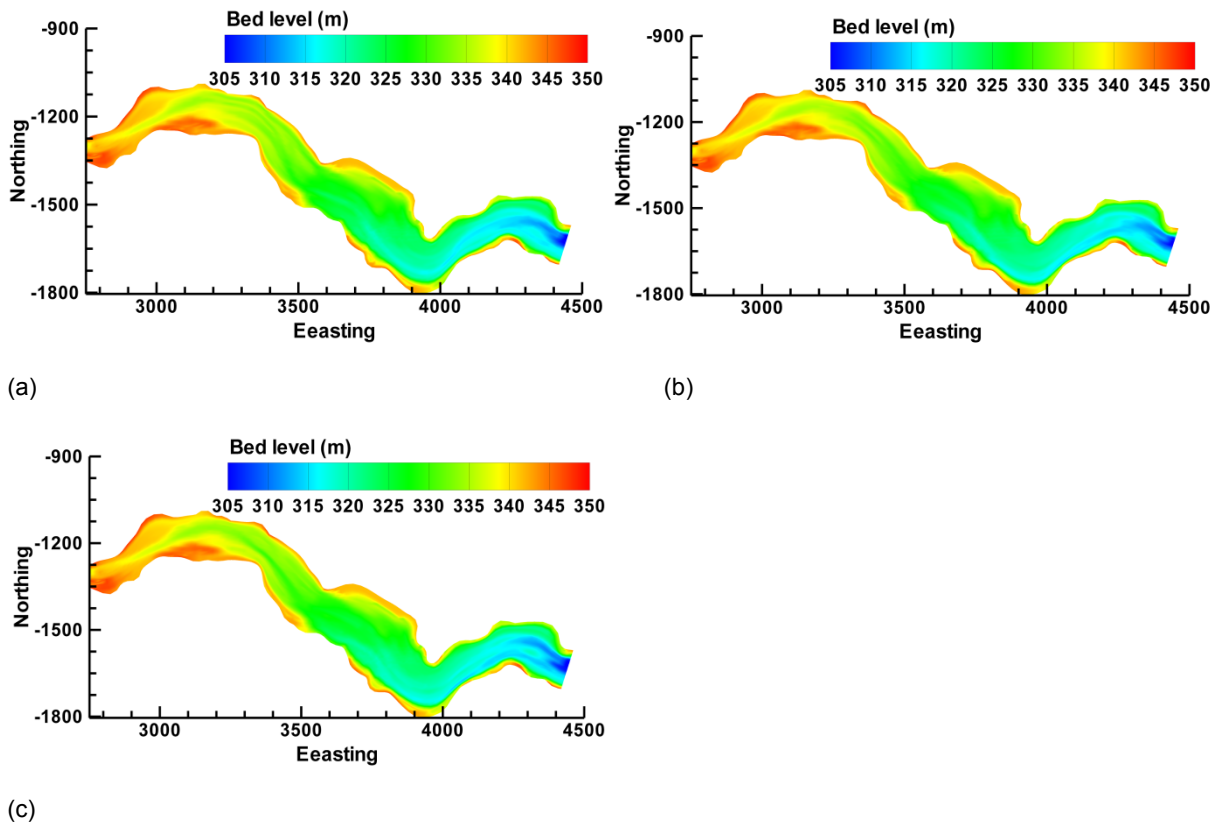
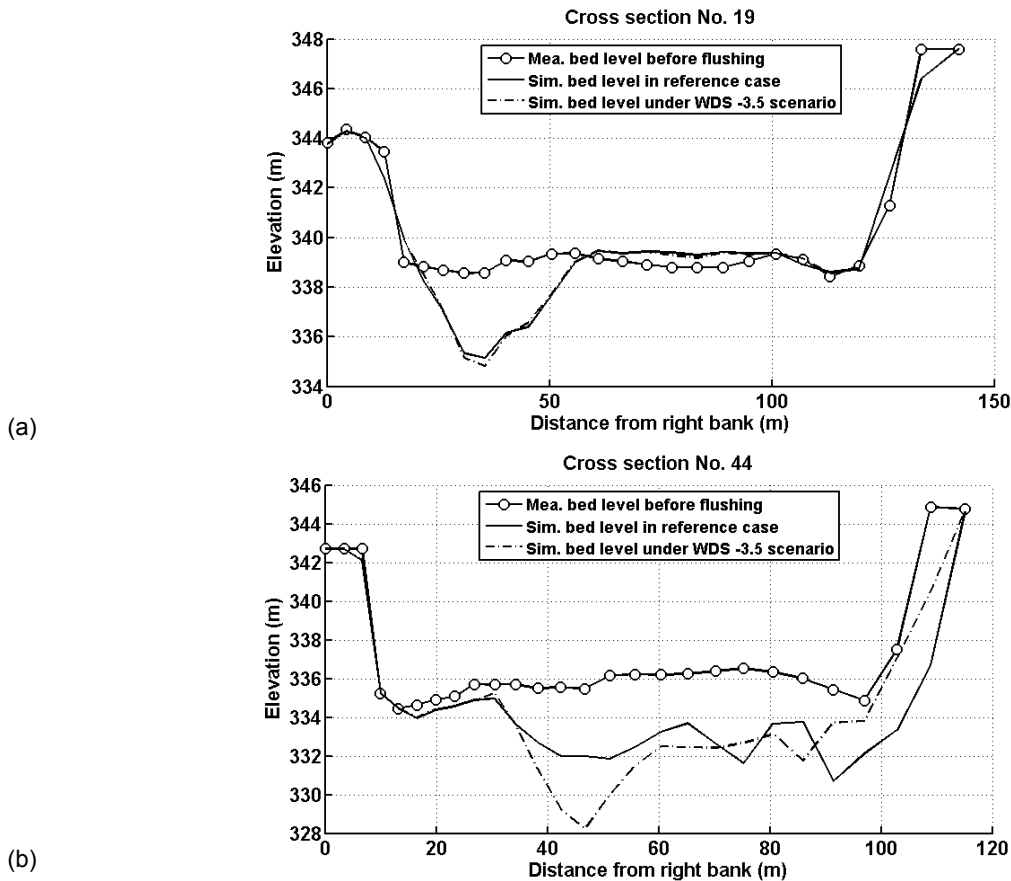


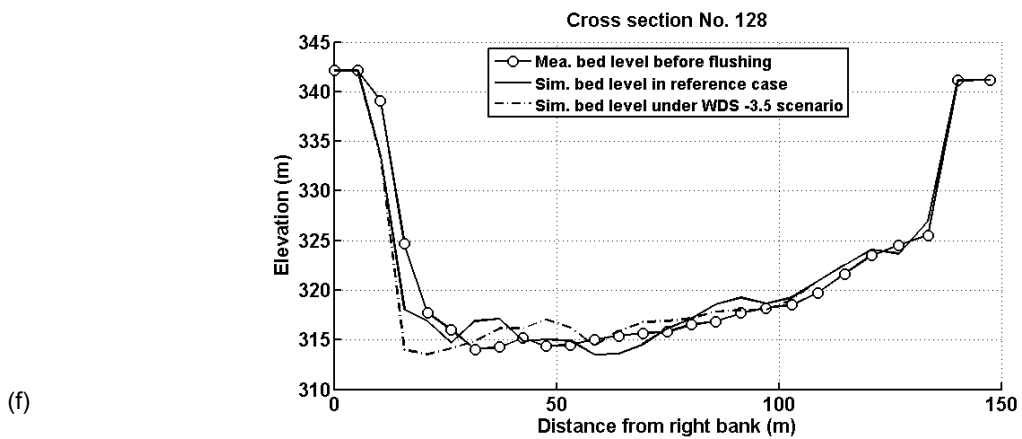
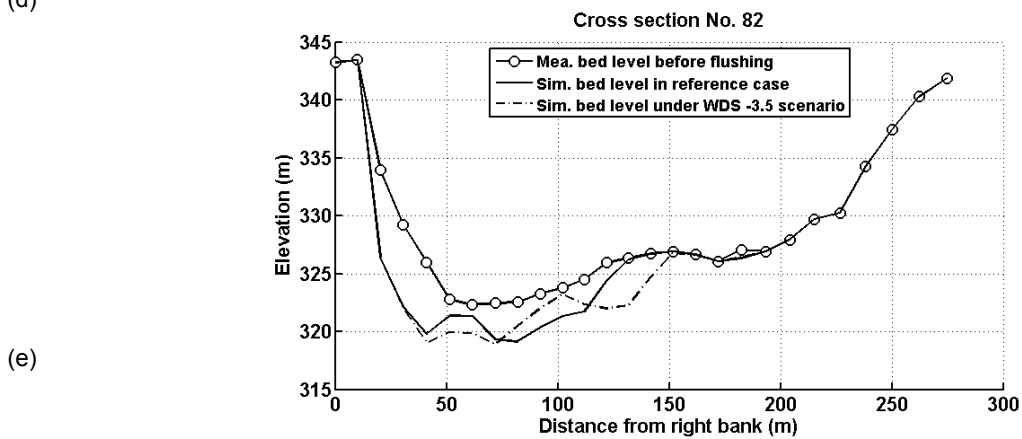
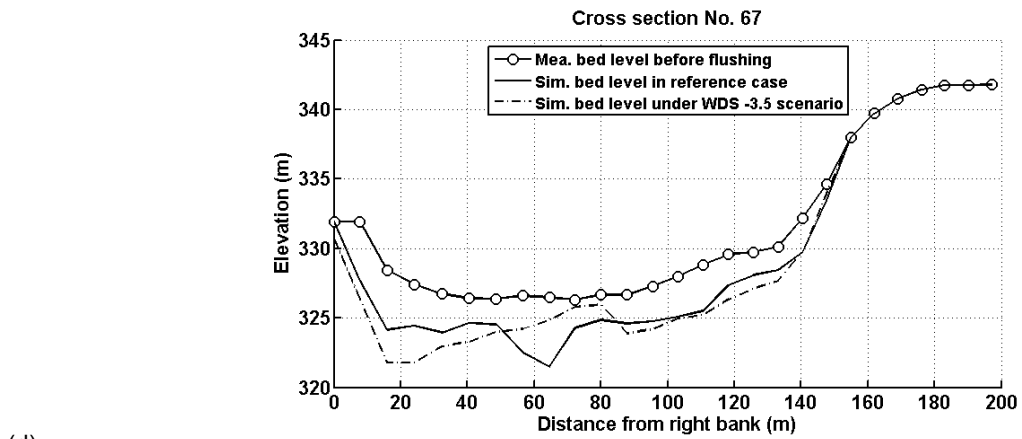
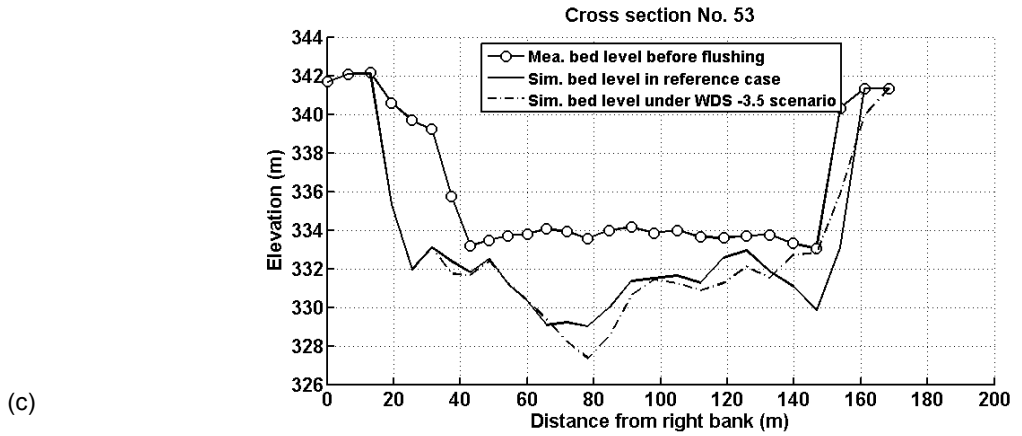
Figure 6.16 Simulated morphological bed levels under (a) the original discharge rates of 2012 flushing event; (b) WDS -0.5 scenario; (c) WDS -3.5 scenario.

	WDS -0.5			WDS -2.5			WDS -3.5		
Parameter	I	II	III	I	II	III	I	II	III
BCI (m)	0.03	-0.14	0.02	0.10	-0.13	-0.23	-0.08	-0.33	-0.22
TVFS (KCM)	322.78			331.87			378.85		

Table 6.8 Average bed level changes in different areas of the reservoir under different WDS scenarios compared to the reference case.

As can be observed from Figure 6.16 and Table 6.8, TVFS increases slightly when the quantity of extra drop in the water level is small (e.g., 0.5 m in WDS -0.5 scenario). When the extra drop in the water level is 2.5 m, TVFS increases but it is not remarkable. Although the water level decrease can enhance the relative roughness (i.e., ratio of roughness height to the water depth), this would not be high enough to erode the coarser materials from upstream areas. Thus, its effect would be limited to the finer materials in WDS -0.5 and WDS -2.5 scenarios. When the extra water level drop becomes bigger in WDS -3.5 scenario, bed erosion occurs over the whole reservoir including the upstream areas covered with coarser materials due to the remarkable enhancement in the relative roughness quantity. In Figure 6.17, final bed levels in area I, II, and III under WDS -3.5 scenario have been illustrated and compared to the bed levels in the reference case to quantitatively show the performance of this scenario for eroding all types of bed materials from the reservoir.





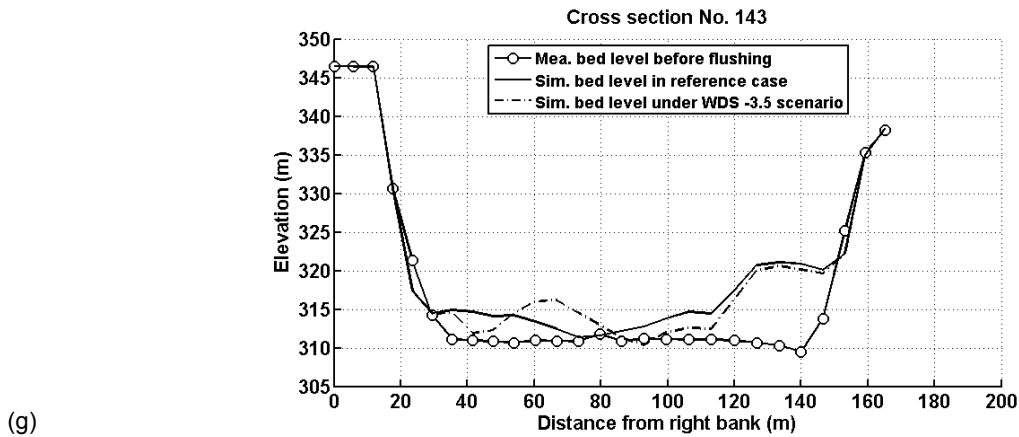


Figure 6.17 Measured bed level before flushing along with the simulated one after flushing in the reference case, and under WDS -3.5 scenario at the location of a) cross section A-A; b) cross section D-D; c) cross section E-E; d) cross section F-F; e) cross section H-H; f) cross section K-K; g) cross section L-L. Location of the sections can be found in Figure 5.8.

The effect of introducing an extra decrease in the water level in various WDS scenarios on the Flushing Efficiency (i.e., FE) and the Total Volume of Flushed out Sediment from the reservoir (i.e., TVFS), when FE is the ratio of the flushed out sediment volume to the used water volume, has been illustrated in Figure 6.18. Horizontal axis shows the ratio of the extra imposed water level drop (i.e., Δh_2) to the original one (i.e., Δh_1) at the beginning of the target segment. This axis shows that how much extra water level drop is required temporarily (i.e., within one hour) to enhance the erosion (i.e., TVFS) in Dashidaira reservoir. FE_2 and FE_1 are the flushing efficiencies when a WDS scenario and when the original water level is employed, respectively. Left and right vertical axes figure out the variation of FE and TVFS, respectively. It should be noted that variations of FE and TVFS in vertical axes and changes in the ratio of water level drop shown in horizontal axis (i.e., $\Delta h_2/\Delta h_1$) were all calculated relative to the reference simulated case (i.e., 2012 flushing operation). As can be observed from this figure, in overall view, TVFS increases when the $\Delta h_2/\Delta h_1$ goes up. Since the used water volume remains unchanged, the FE will increase accordingly. In other words, FE variation has an overall direct relation with the variation of $\Delta h_2/\Delta h_1$. However, removing the coarser materials from the far upstream area of the reservoir requires a high extra drop in the water level (i.e., high $\Delta h_2/\Delta h_1$). Moreover, increasing the $\Delta h_2/\Delta h_1$ in some cases results in a lower TVFS and FE accordingly. This can be attributed to the bed armoring process that takes place as eroded coarser materials move forward from upstream to downstream areas. When the $\Delta h_2/\Delta h_1$ increases the erosion of coarser materials may be higher and at the same time they are likely to be transported more downstream to finally deposit on the finer materials in the deeper parts. If driving forces produced by extra drop in the water level are not high enough to displace the newly deposited coarser sediments on the finer ones, the erosion of finer fractions may be decreased that will result in a lower TVFS and subsequently a lower FE_2 .

Although remarkable drop in the water level enhances the sediment erosion volume from the whole reservoir area, the flushing channel location is still close to the right bank in area II and deposited materials along the left bank (i.e., dead zone) cannot be removed effectively. Therefore, it is

recommended to consider other types of soft and hard measures in the future outlook. Construction of auxiliary longitudinal channels and using the artificial flow guiding structures can be considered as other measures to increase the sediment erosion form the mentioned dead zone.

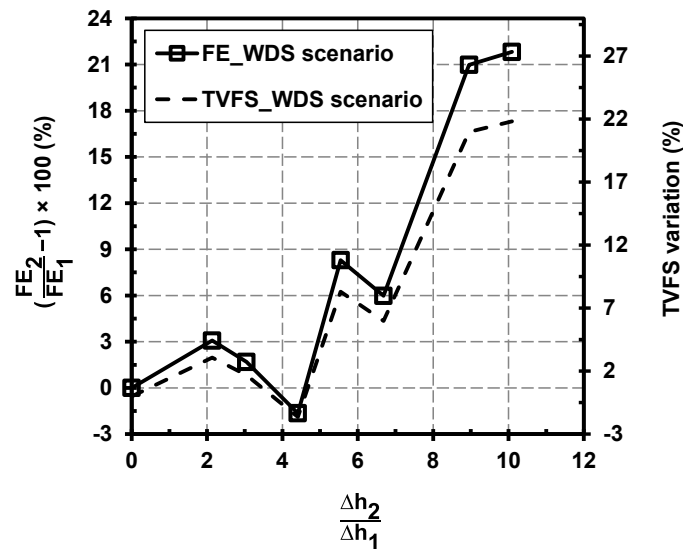


Figure 6.18 Non-dimensional curves showing the relation between TVFS, FE, and Δh . All values are compared to the reference case under different WDS scenarios.

6.3 Hard measure scenarios

Hard-measure scenarios, here, implies the construction of flow-guide structures and auxiliary longitudinal channels in the resevoir to enhance the sediment erosion in specific segments of the reservoir. For instance, to increase the erosion from the dead zone in area II of Dashidaira reservoir or from the upstream area of Unazuki reservoir. Hard measures influence the hydrodynamic characteristics in a specific segment of the reservoir during the whole time of simulation. In this section, first, one example from each mentioned type of hard measure was applied in Dashidaira reservoir and effects were assessed over the target dead zone morphology. Then, performance of the introduced hard measures was tested in Unazuki reservoir.

6.3.1 Construction of flow-guiding structures in Dashidaira reservoir

As it was observed in the real flushing operation and also in numerical modelling of 2012 flushing, flushing channel location is mainly close to the right bank in the wide middle area (i.e., area II). This condition is dominant, although some small chnages in the flushing channel geometry can be happened when diverse type of soft measures are applied. As a consequence, deposited sediments along the lef bank of area II (i.e., dead zone) cannot be be flushed out effectively.

Changing the flow direction at the entrance of area II using a flow guiding structure (e.g., groyne), then, can be considered as a type of hard measure. Figure 6.19 shows the location of implemented flow-guiding structure used for the further assessment in this section. A groyne can deflect a portion of the coming flow to the left hand side of area II, which can contribute in developing an additional new secondary channel along the dead zone. Subsequently, two flushing channels will form in area II, one close to the right bank of arae II and other one along the dead zone. Having two flushing channels in area II is more beneficial rather than having just one flushing channel from different aspects. First, FE increases because the erosion occurs in two flushing channels instead of one that means flushing occurs in longer channel. If just one flushing channel exists, this channel will attarct the whole flow and the channel will mainly deepen that is not an effective approach for increasing the FE. Second, much more amount of deposited sediments can be eroded from the dead zone that could not be eroded before. Third, using this measure can permannently form the additional flushing channel along the dead zone so that a higher TVFS and FE can be expected during next flushing events with higher dicharges or longer durations because the total existing flushing channel length increases.

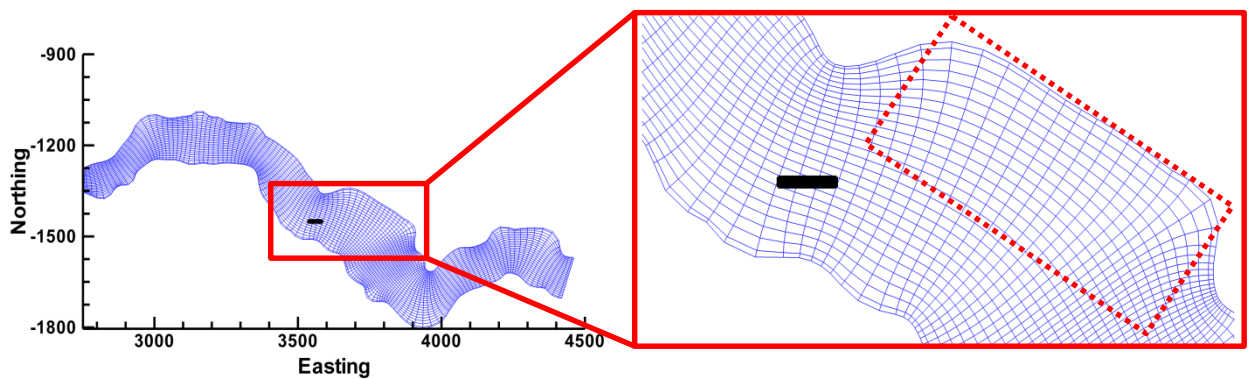
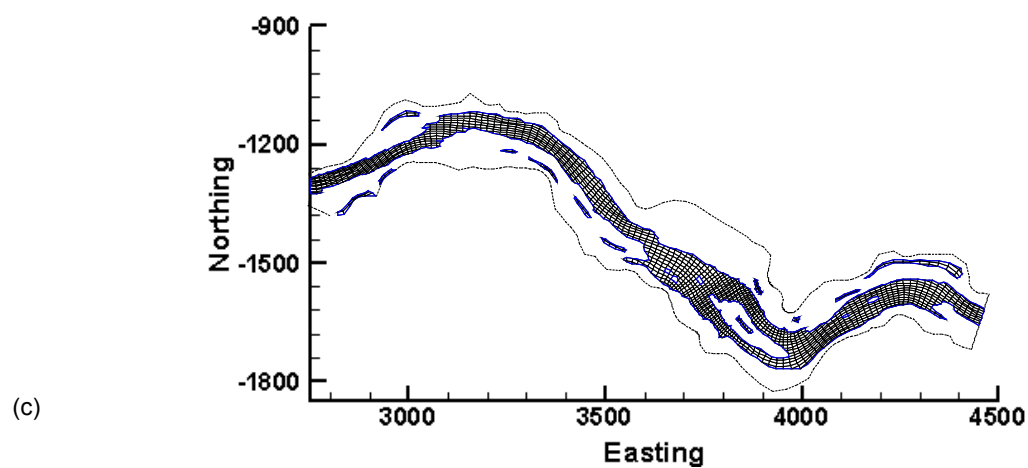
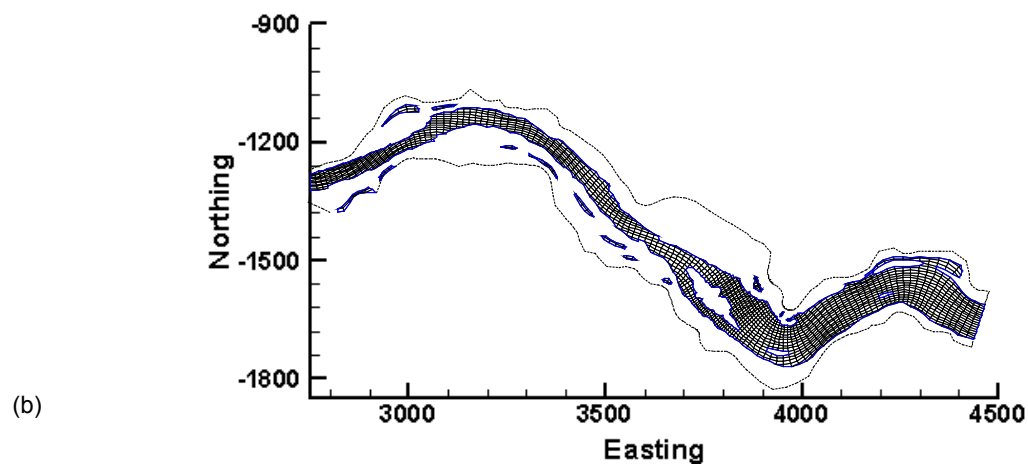
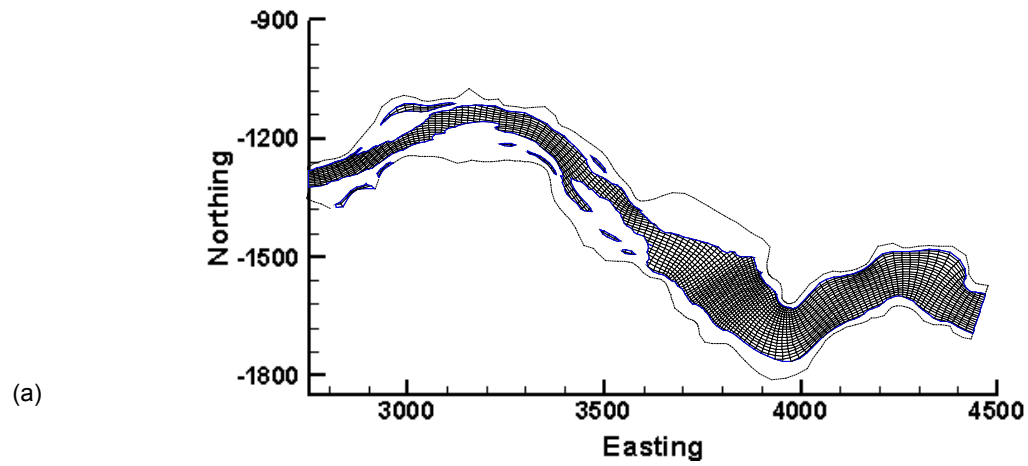
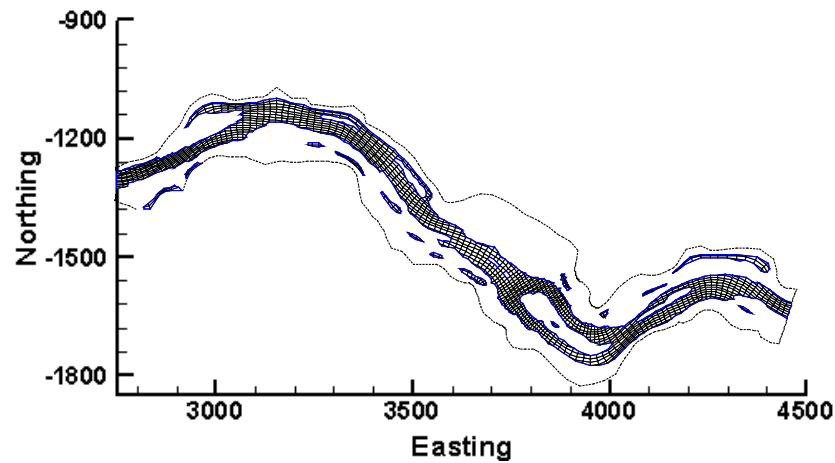


Figure 6.19 Location of the implemented groyne at the entrance of area II to further deflect the water flow towards the dead zone close to the laft bank shown with a dashed line rectangle.

Figure 6.20, depicts the computational grid adjustment at the beginning (i.e., $t=10$ hrs) and middle of drawdown stage (i.e., $t=16$ hrs), during the free-flow condition with a low water head in the reservoir (i.e., $t=32$ hrs), and also at the end of the free-flow stage (i.e., $t=36$ hrs). This figure clearly shows the flushing channel location and its development under the effect of used single groyne at the entarnce zone of area II. As can be seen, at the beginning of drawdown stage (i.e., $t=10$ hrs) the water flow deflects more towards the left bank, at downstream of area II, since computational grid exists in the mentioned zone. Then, at the middle of drawdown stage (i.e., $t=16$ hrs), bifurcation of the flushing channel into two distinctive channels together with a sandbar zone between them can be observed. Afterwards, during the free-flow stage (i.e., $t=32$ hrs) head of the sandbar is eroded that causes the channels to be coupled and form a wide flushing channel in the head area of the sand bar. Meanwhile, the sandbar is shifted further downstream of area II and finally two distictive flushing channel can be just observed in downstream of area II at the end of free-fow stage (i.e., $t=36$ hrs). Beside of the contribution in formation of an additional secondary flushing channel during the drawdown and free-flow

stages, using the groyne has shifted the main flushing channel location from the right bank of area II into the reservoir center.



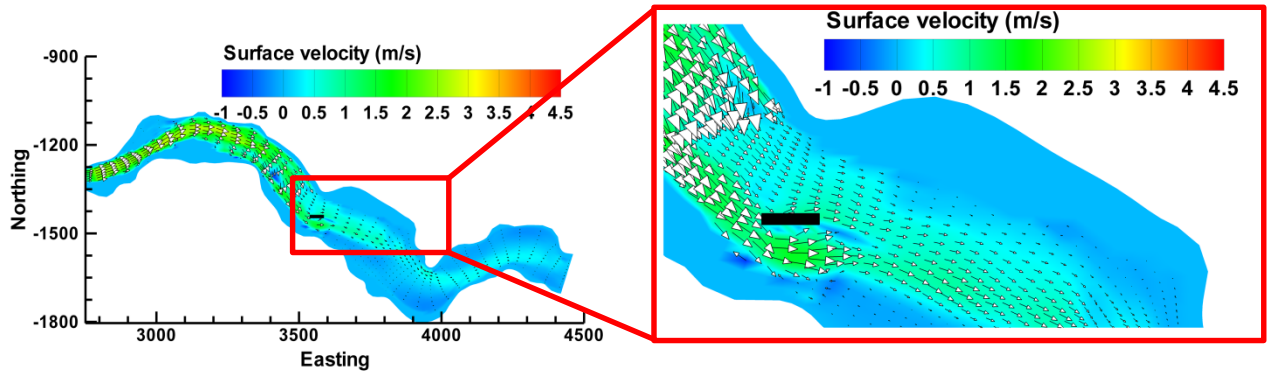


(d)

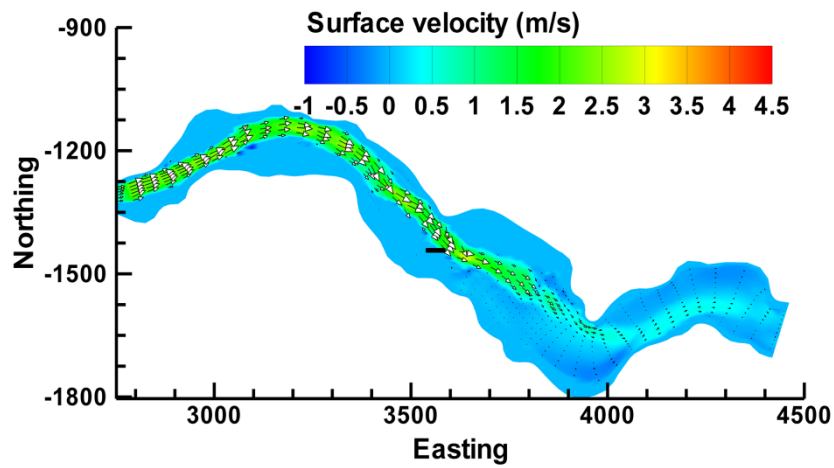
Figure 6.20 Computational grid adjustment showing the flushing channel location at (a) the beginning of drawdown stage (i.e., $t=10$ hrs) ; (b) middle of drawdown stage (i.e., $t=16$ hrs) ; (c) middle of free-flow stage (i.e., $t=32$ hrs) ; (d) end of free-flow stage (i.e., $t=36$ hrs).

In Figure 6.21, the surface velocity vectors before drawdown (i.e., $t=5$ hrs), at the beginning (i.e., $t=10$ hrs) and middle of drawdown stage (i.e., $t=16$ hrs), and also during the free-flow condition (i.e., $t=32$ hrs) have been shown, respectively. The figure demonstrates how the flow velocity distribution is affected by the groyne and consequently the morphological bed changes are manipulated. As can be observed from Figure 6.16 (a), before onset of the drawdown (i.e., $t=5$ hrs), groyne alter the flow velocity characteristics and prevent the whole flow from diversion towards the right bank at the entrance of area II. More specifically, groyne deflects a portion of the water flow towards the left half of area II (i.e., dead zone) and in the meantime other portion of the flow continue its original path close to the right bank of area II since the original thalweg exists in this area. Moreover, the velocity is still higher in the portion of flow close to the right bank. At the beginning of drawdown stage (i.e., $t=10$ hrs), as shown in Figure 6.16 (b), the coming flow is mainly deflected towards the left hand side as flushing channel forms along the left hand side of the groyne in the dead zone of area II. At the middle of drawdown stage (i.e., $t=16$ hrs), flow is bifurcated and consequently bifurcation of the flushing channel into two distinctive channels happens as shown in Figure 6.16 (c). The flow bifurcation can be attributed to the existing original channel thalweg along the right bank that can attract the flow. Flow bifurcation is also continues during the free-flow stage (i.e., $t=32$ hrs) as shown in Figure 6.16 (d).

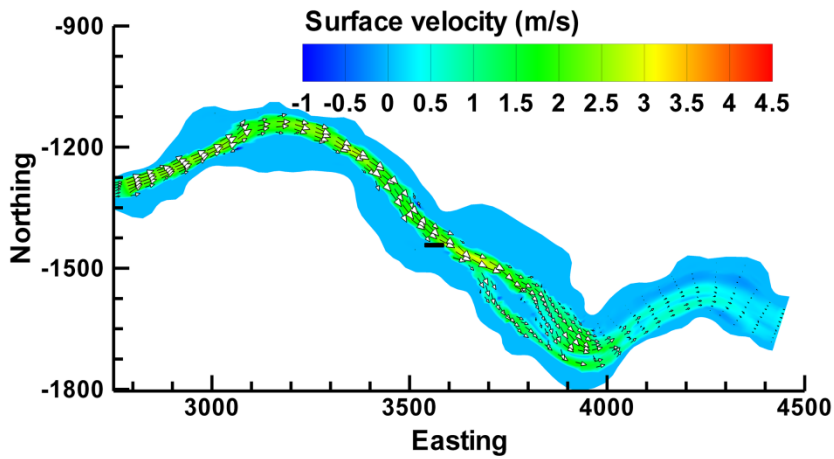
Figure 6.22 illustrates the plan view of final bed morpholgy using a groyne at the entrance of area II. The two side by side flushing channel location and their corresponding thalweg levels can be aparently observed from the figure in area II. Owing to the formation of an additional flushing channel along the left hand side beside the one that is able to be established originally along the right bank of area II, the TVFS increased to $343.36 \times 10^3 \text{ m}^3$ compared to the reference case with $313.14 \times 10^3 \text{ m}^3$. Thus, FE increases 9.65% compared to the reference case. Since area II and III are mainly covered with finer sediments, they can be eroded easily along the two flushing channels that is favorable for increasing the TVFS.



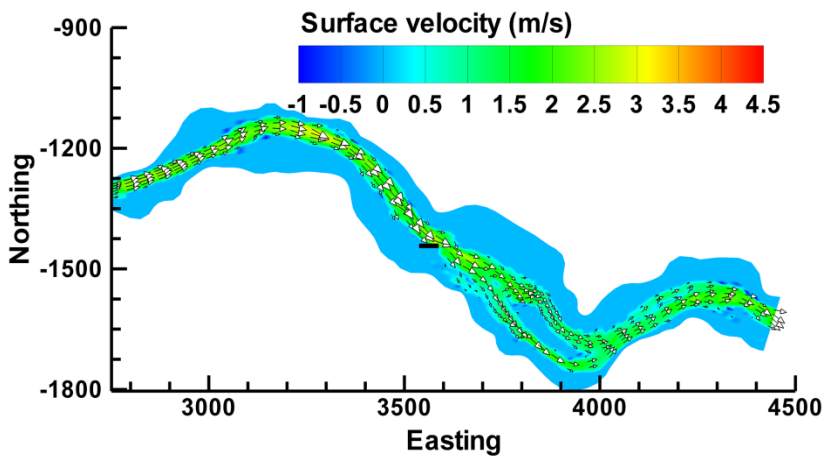
(a)



(b)



(c)



(d)

Figure 6.21 Surface velocity fields at different satges using a groyne in Dashidaira reservoir: (a) before onset of the drawdown stage (i.e., $t=5$ hrs); (b) at the beginning of drawdown stage (i.e., $t=10$ hrs); (c) at the middle of drawdown stage (i.e., $t=16$ hrs); (d) during the free-flow condition (i.e., $t=32$ hrs).

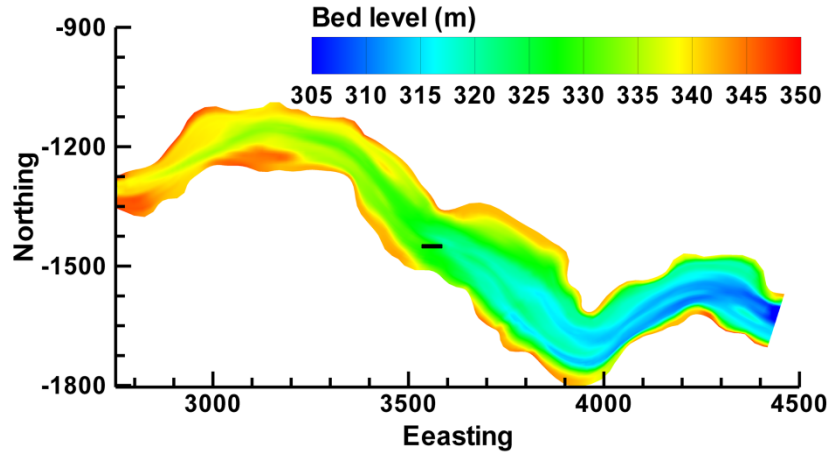
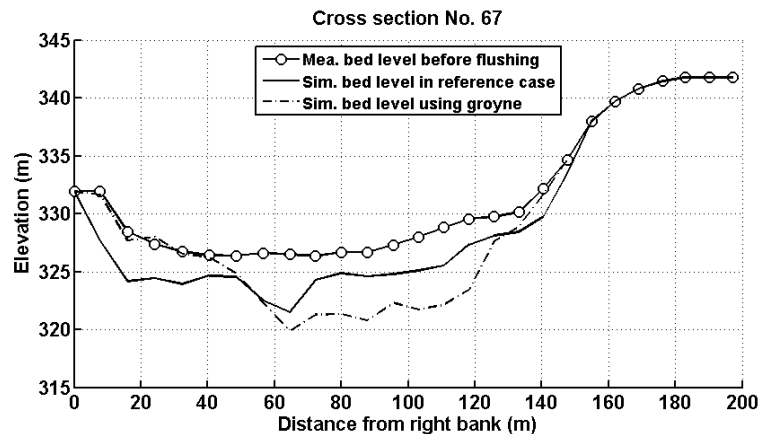
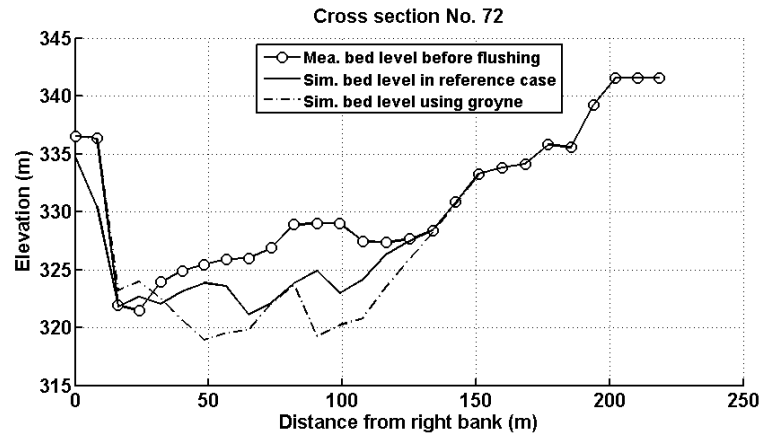


Figure 6.22 Plan view of final simulated bed levels using a groyne at the entrance of area II. Two side by side flushing channels track can be seen in area II and III.

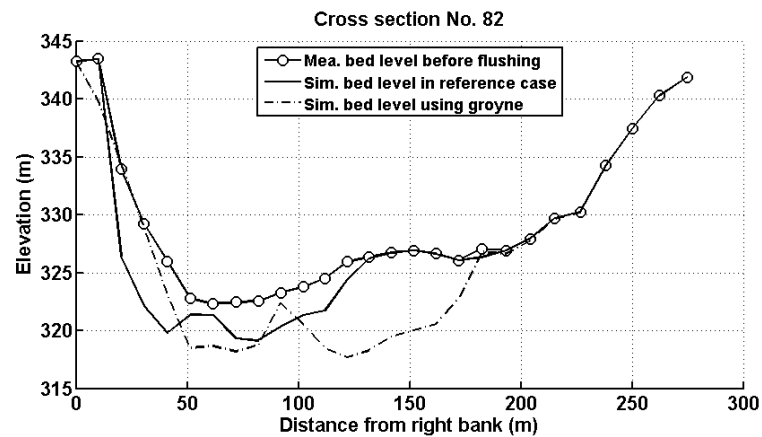
In Figure 6.23 final bed levels after flushing in area II and III were shown when groyne is utilized and they were compared to the bed levels in the reference case to quantitatively show the performance of this type of hard measure for increasing the bed erosion mainly from the left side of area II. One can clearly found from this figure that bed levels lowered remarkably along the left bank of area II. Furthermore, bed levels are lowered slightly in area III due to the significant velocity reduction and sediment deposition in this area caused by the large water depth.



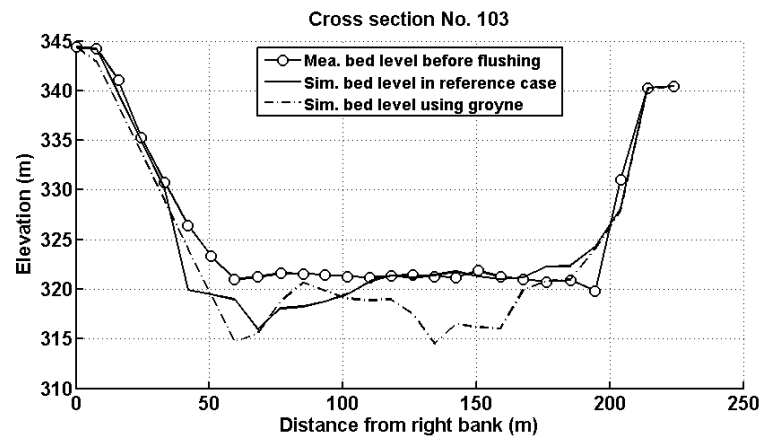
(a)



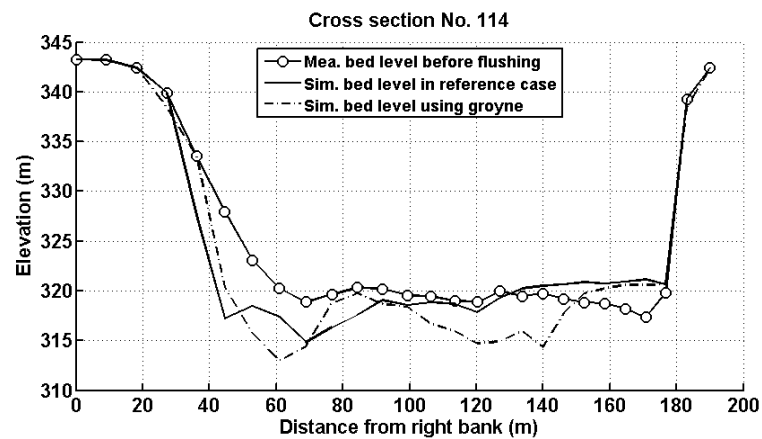
(b)



(c)



(d)



(e)

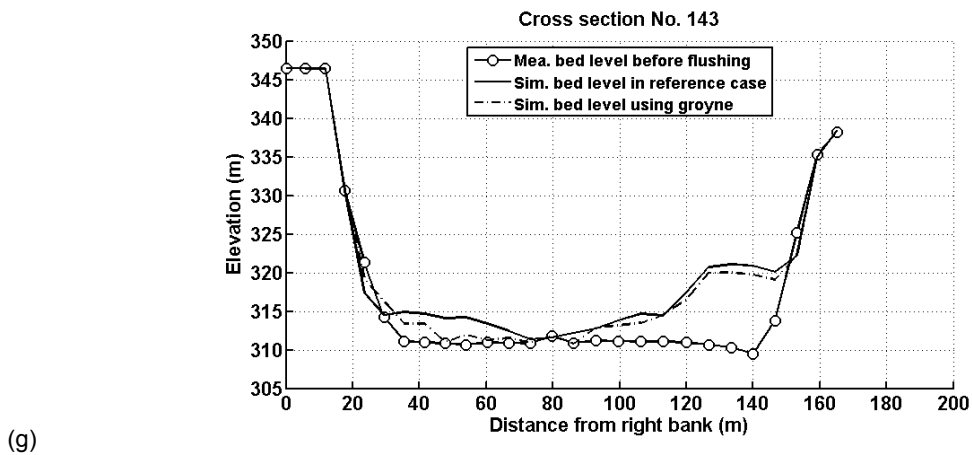
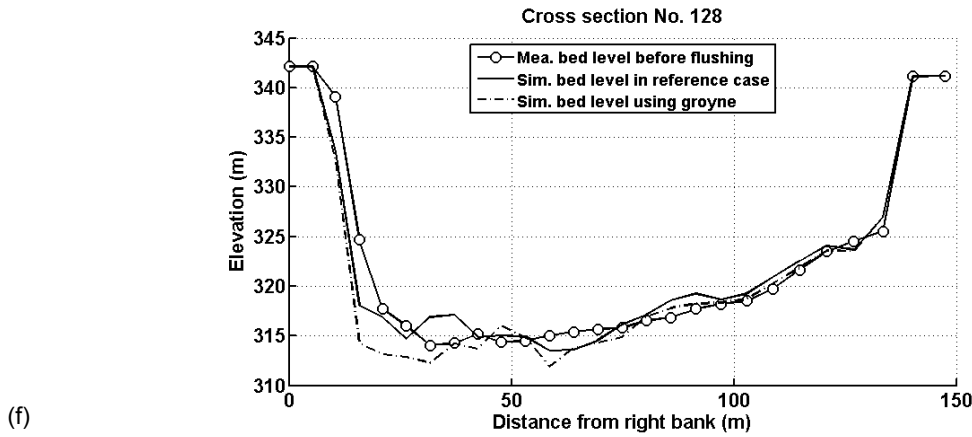


Figure 6.23 Measured bed level before flushing along with the simulated one after flushing in the reference case, and when groyne is applied at the location of a) cross section F-F; b) cross section G-G; c) cross section H-H; d) cross section I-I; e) cross section J-J; f) cross section K-K; g) cross section L-L. Location of the sections can be found in Figure 5.8.

Using the groyne contributed in increasing the TVFS because it changed the location of flushing channel to scour the deposited sediemnt from the target dead zone and becuase it caused the channel bifurcation. If groyne just changes the flushing channel location, the TVFS may not be increased significantly and the advantage would be reduced solely to a higher erosion from the target dead zone area. Thus, a wider study on the position of the groyne or group of groynes under different hydrdynamic boundary conditions may be considered for further development of this study. Besides, the economical assessment for constructing the groynes should be conducted due to the risk of failure caused by accelerated laden flow and local scouring phenomena around the groynes. Application of groynes may also increase the deposition locally during the deposition period in the reservoir and causes the secondary problems. Therefore, their effect on morphological bed changes should be analyzed during the sediemnt deposition period.

6.3.2 Construction of an auxiliary longitudinal channel in Dashidaira reservoir

Another alternative for improving the erosion from the left side of area II in Dashidaira reservoir (i.e., dead zone) can be application of a longitudinal auxiliary (i.e., secondary) channel beside the main channel. This auxiliary channel contributes in removing a portion of the deposited material from the

dead zone and can be excavated artificially using mechanical devices when the water level in the reservoir is low enough. In this method, the flushing flow is diverted from the main channel into the auxiliary channel and again enters to the main channel by a confluence further downstream of the diversion point. Longitudinal auxiliary flushing channel concept has been illustrated schematically in Figure 6.24 along with the location of the implemented auxiliary channel used for the further assessment in this section.

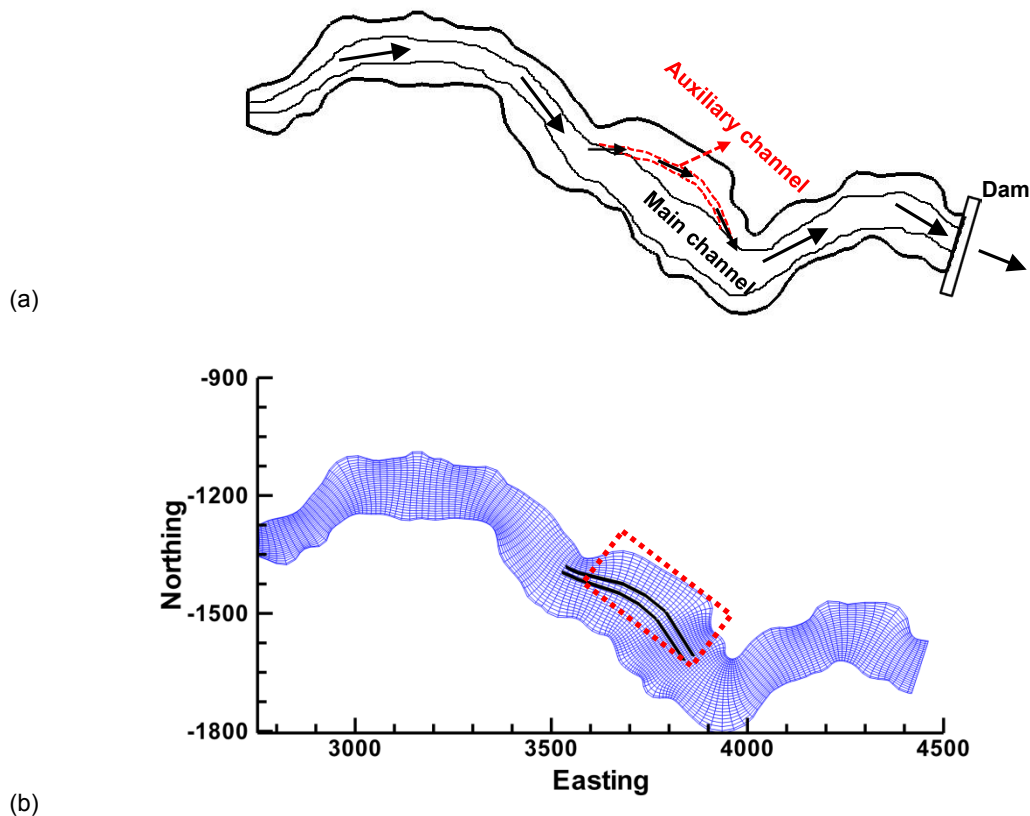
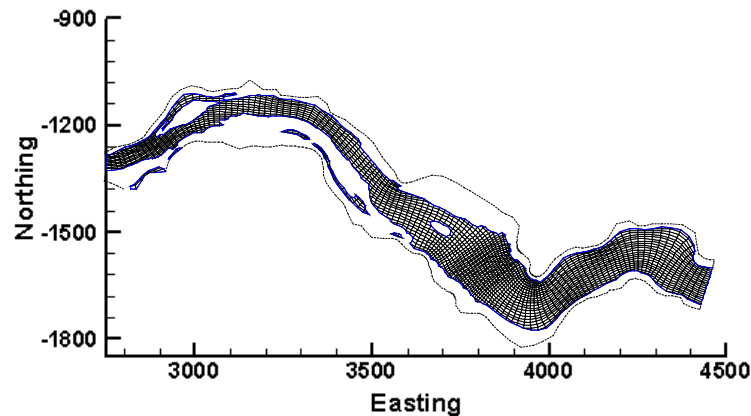


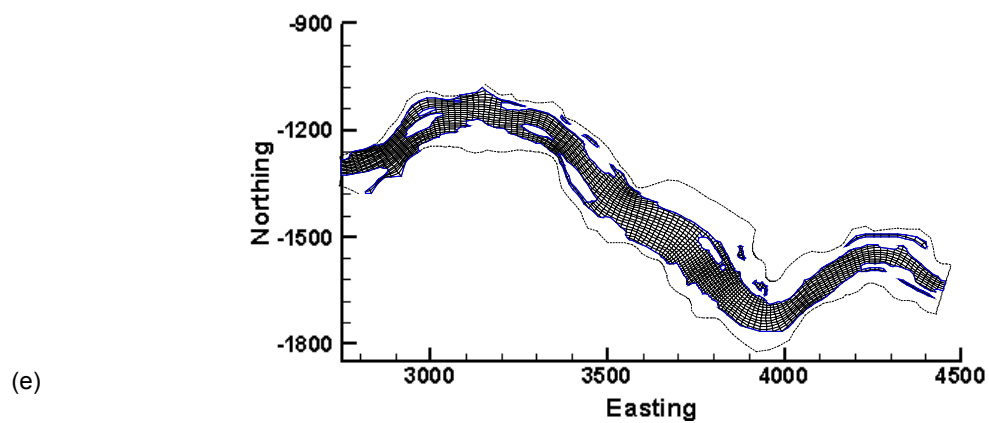
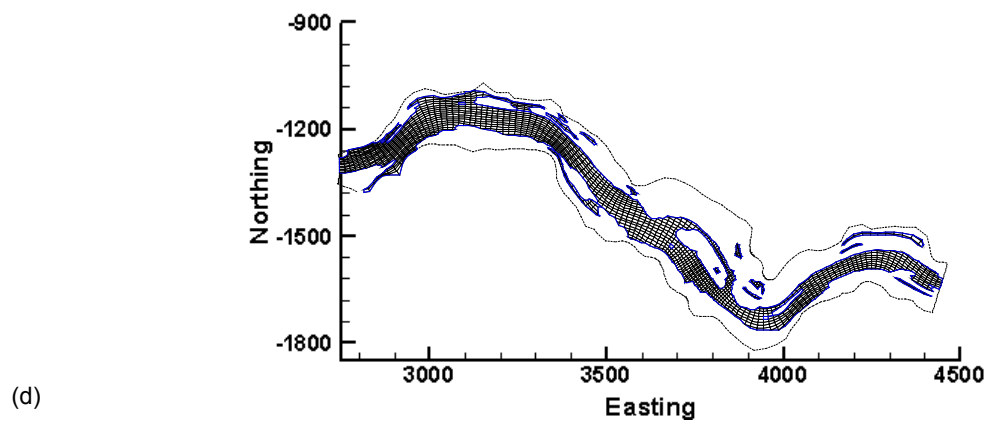
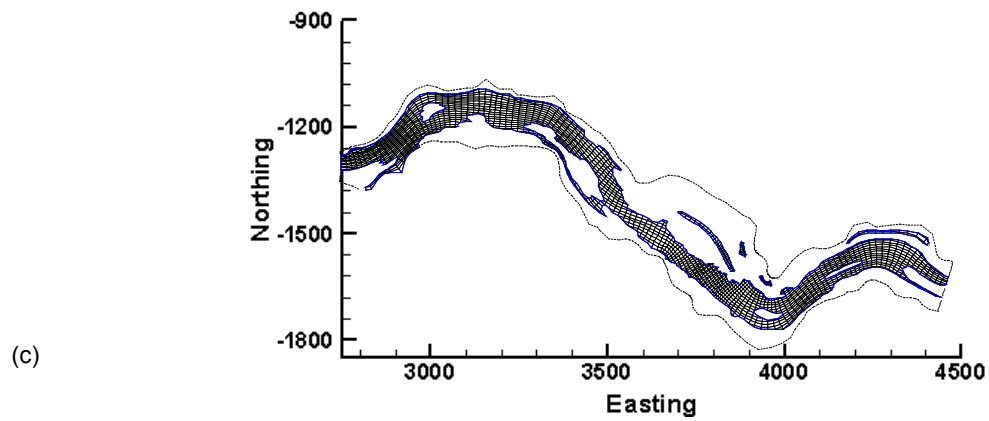
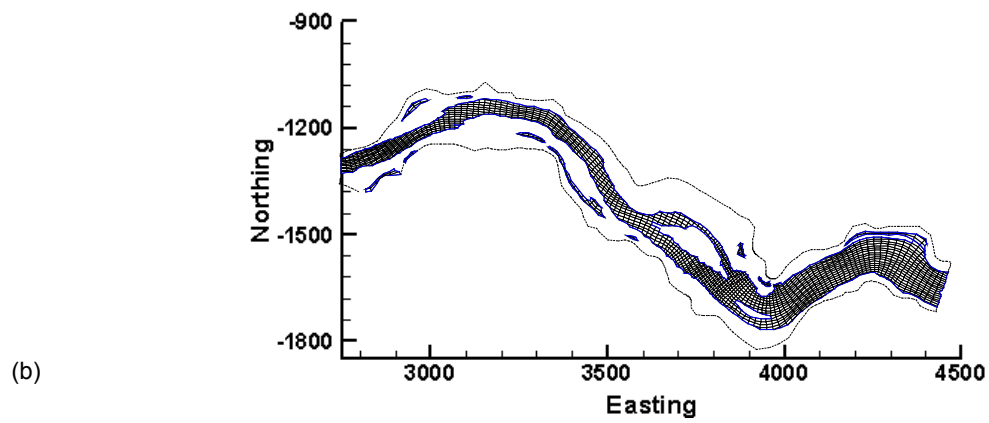
Figure 6.24 (a) Schematic illustration of the concept of longitudinal auxiliary flushing channel; (b) location of the implemented auxiliary flushing channel in the dead zone of the reservoir; borders of the channel showed with bold black line and dead zone has been shown with the dotted red rectangle.

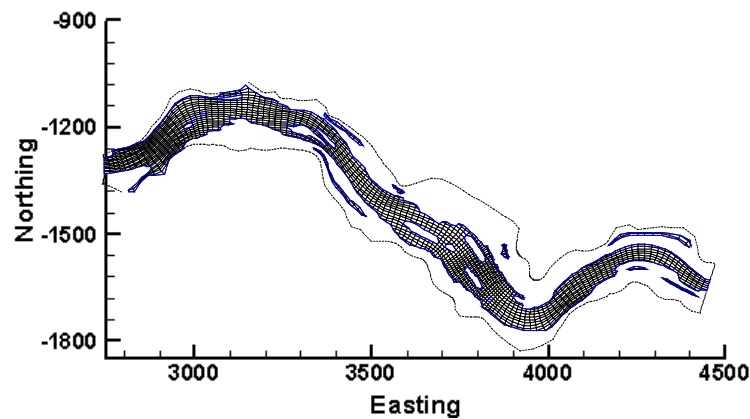
An auxiliary longitudinal channel along the dead zone can attract a portion of the coming flow to the left hand side of area II. Subsequently, morphological process will take place along two flushing channels in area II, namely, along the original flushing channel close to the right bank of area II and along the auxiliary flushing channel along the dead zone in area II. Having two flushing channels in area II is more beneficial rather than having just one flushing channel from two aspects. First, FE increases because the erosion occurs in two flushing channels along a longer length instead of one channel. If just one flushing channel exists, this channel will attract the whole flow and the channel will mainly deepen that is not an effective approach for increasing the FE. Second, much more amount of deposited sediments can be eroded from the dead zone that could not be eroded when just one original flushing channel develops close to the right bank of area II.

Figure 6.25, depicts the computational grid adjustment at the beginning (i.e., $t=10$ hrs), middle (i.e., $t=16$ hrs) and end ((i.e., $t=21$ hrs) of drawdown stage, during the free-flow condition with a low water head in the reservoir (i.e., $t=26$ hrs), and also at the end of the free-flow stage (i.e., $t=36$ hrs). This figure clearly shows how the original flushing channel along the right bank of area II, and the auxiliary flushing channel along the dead zone contribute in the sediment transportation process. As can be seen, at the beginning of drawdown stage (i.e., $t=10$ hrs) the water flow deflects more towards the left bank, at downstream of area II, since the auxiliary channel attracts a portion of incoming flow and computational grid exists in the mentioned zone. Then, at the middle of drawdown stage (i.e., $t=16$ hrs), bifurcation of the flushing channel into two distinctive channels (i.e., a main wider flushing channel and a narrower auxiliary channel) together with a sandbar zone between them can be observed. Following that, at the end of drawdown stage (i.e., $t=21$ hrs) auxiliray flushing channel disappears due to the sudden decrease in the water level and wetting/drying algorithm effect. Afterwards, during the free-flow stage (i.e., $t=26$ and 32 hrs), the auxiliary flushing channel appears again and simultaneously the sandbar erosion is started. Thus, water exchange between the main flushing channel and the auxiliary flushing channel became able to take place especially when the small water level fluctuations are performed for remobilization of eroded materials during the free-flow stage. Consequently, the morphological process takes place in a unified wider channel along the center of area II whereas two flushing channels are still working. This condition continues until the end of of free-fow stage (i.e., $t=36$ hrs). Likewise the groynes, using a longitudinal auxiliary channel has shifted the main flushing channel location from the right bank of area II into the reservoir center through attracting a portion of incoming flow and thereby dfecting the velocity direction from the entrance of area II.



(a)

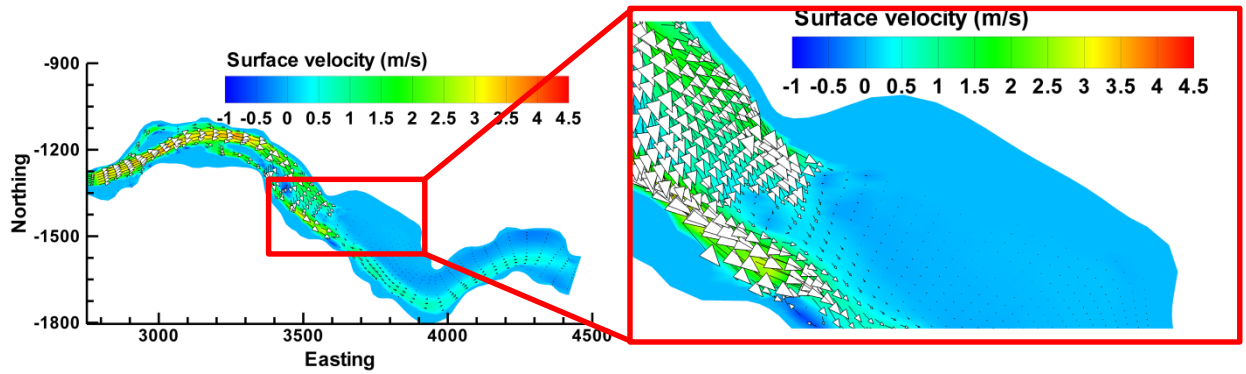




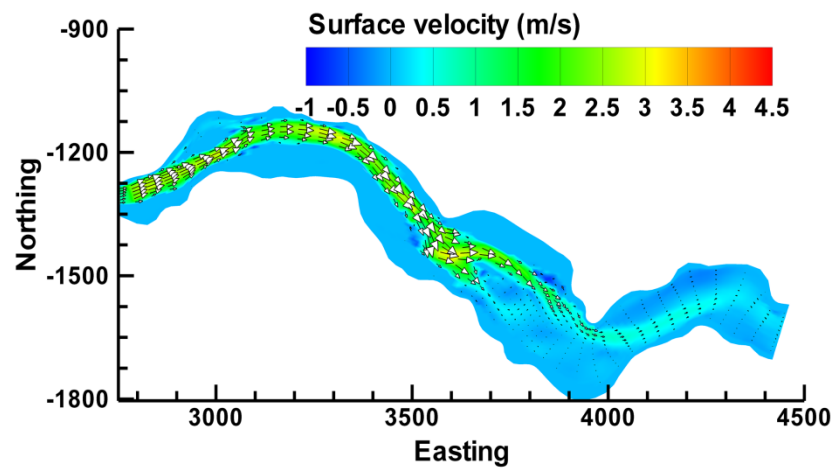
(f)

Figure 6.25 Computational grid adjustment showing the flushing channel location at (a) the beginning of drawdown stage (i.e., $t=10$ hrs) ; (b) middle of drawdown stage (i.e., $t=16$ hrs) ; (c) end of drawdown stage (i.e., $t=21$ hrs) ; (d) early free-flow stage (i.e., $t=26$ hrs) ; (e) middle of free-flow stage (i.e., $t=32$ hrs) ; (f) end of free-flow stage (i.e., 36 hrs).

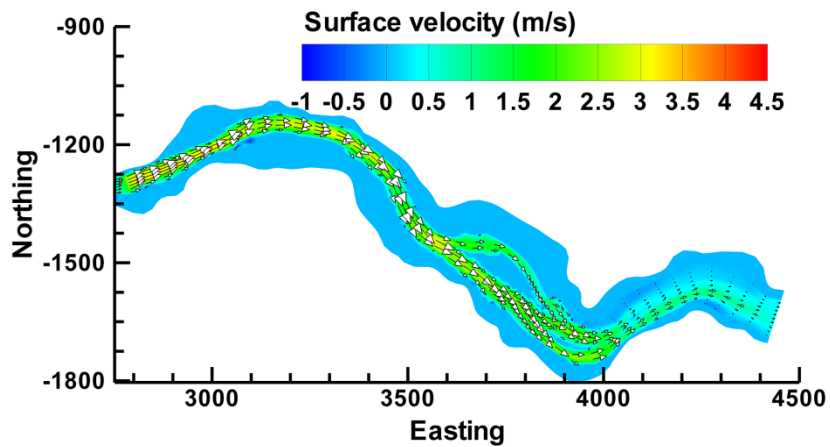
In Figure 6.26, the surface velocity vectors before drawdown (i.e., $t=5$ hrs), at the beginning (i.e., $t=10$ hrs) and middle of drawdown stage (i.e., $t=16$ hrs), and also during the free-flow condition (i.e., $t=32$ hrs) have been shown, respectively. The figure demonstrates how the incoming flow is diverted towards the auxiliary channel and consequently the morphological bed changes are manipulated in area II of the reservoir. As can be observed from Figure 6.21 (a), before onset of the drawdown, (i.e., $t=5$ hrs), auxiliary channel in the dead zone attracts a portion of the incoming flow and prevent the whole flow from deflection towards the right bank at the entrance of area II where the original flushing channel thalweg exists. In other words, auxiliary channel deflects a portion of the water flow towards the left half of area II (i.e., dead zone) and in the meantime other portion of the flow continue its original path close to the right bank of area II over the original thalweg of the main flushing channel. Moreover, the velocity is still higher in the portion of flow close to the right bank. At the beginning of drawdown stage (i.e., $t=10$ hrs), as shown in Figure 6.21 (b), the incoming flow is mainly deflected towards the auxiliary channel in the left hand side of the dead zone of area II. At the middle of drawdown stage (i.e., $t=16$ hrs), flow is bifurcated completely so that a major portion of incoming flow is deflected towards the main channel and the other portion is deflected towards the auxiliary channel as shown in Figure 6.21 (c). After erosion of the sandbar and the consequent water exchnage between the main and auxiliary flushing channels, the flow velocity will be distributed over a wider flushing channel and this condition continues during the free-flow stage (e.g., $t=32$ hrs) as shown in Figure 6.16 (d).



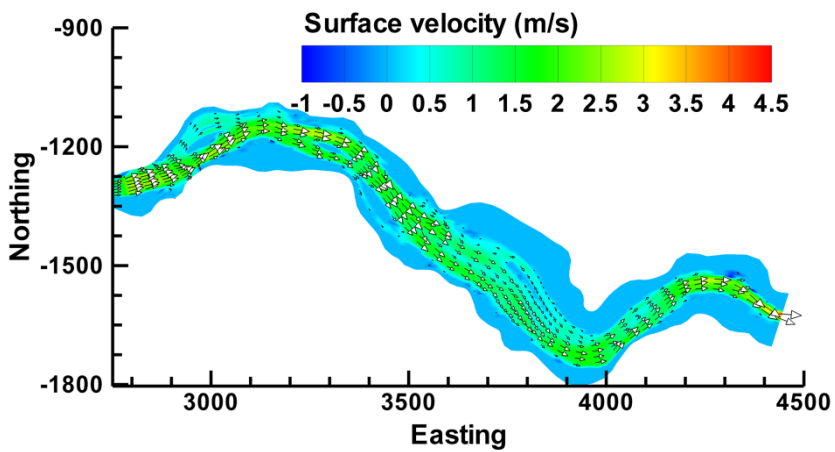
(a)



(b)



(c)



(d)

Figure 6.26 Surface velocity fields at different satges using a groyne in Dashidaira reservoir: (a) before onset of the drawdown (i.e., $t=5$ hrs); (b) at the beginning of drawdown stage (i.e., $t=10$ hrs); (c) at the middle of drawdown stage (i.e., $t=16$ hrs); (d) during the free-flow condition (i.e., $t=32$ hrs).

Figure 6.27 illustrates the plan view of final bed morpholgy employing an auxiliary flushing channel along the dead zone of area II. The thalweg of the main flushing channel along the right bank of area II and also thalweg of the auxiliary flushing channel along the dead zone of area II can be aparently observed from this figure. Owing to the flow diversion into the auxiliary channel along the left hand side beside the original main channel along the right hand side of area II, the total length of the flushing channel is enhanced and the TVFS increased to $340.36 \times 10^3 \text{ m}^3$ compared to the reference case with $313.14 \times 10^3 \text{ m}^3$. Since area II is mainly covered with finer sediments in lower half, bed materials can be eroded easily along the two flushing channels (i.e., main and auxiliary) that is favorable for increasing the TVFS. Thus, FE increases 8.7% compared to the reference case.

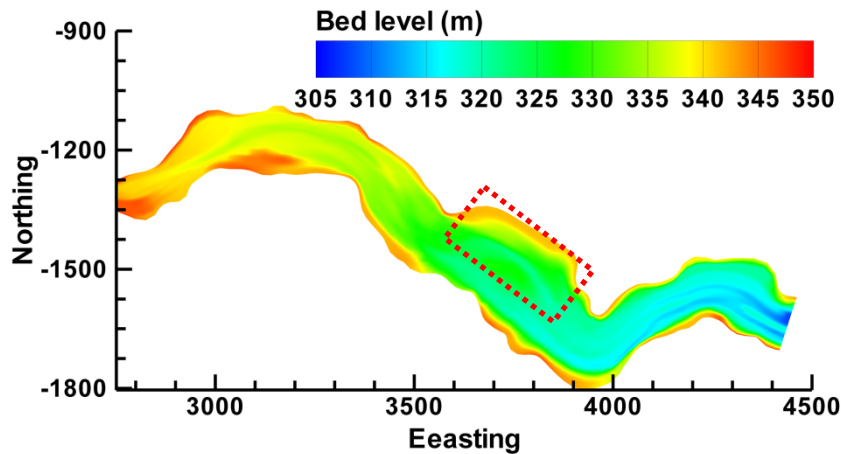


Figure 6.27 Plan view of the final simulated bed levels using an auxiliary flushing channel along the dead zone of area II. Auxiliary flushing channel track has been shown in the red dashed rectangle.

In Figure 6.28 final bed levels after flushing in area II were shown when the auxiliary flushing channel is utilized in area II and they were compared to the bed levels in the reference case to quantitatively show the performance of this type of hard measure for increasing the bed erosion mainly from the left side of area II. One can clearly found from this figure that bed levels lowered noticeably along the auxiliary flushing channel thalweg located in the left bank of area II.

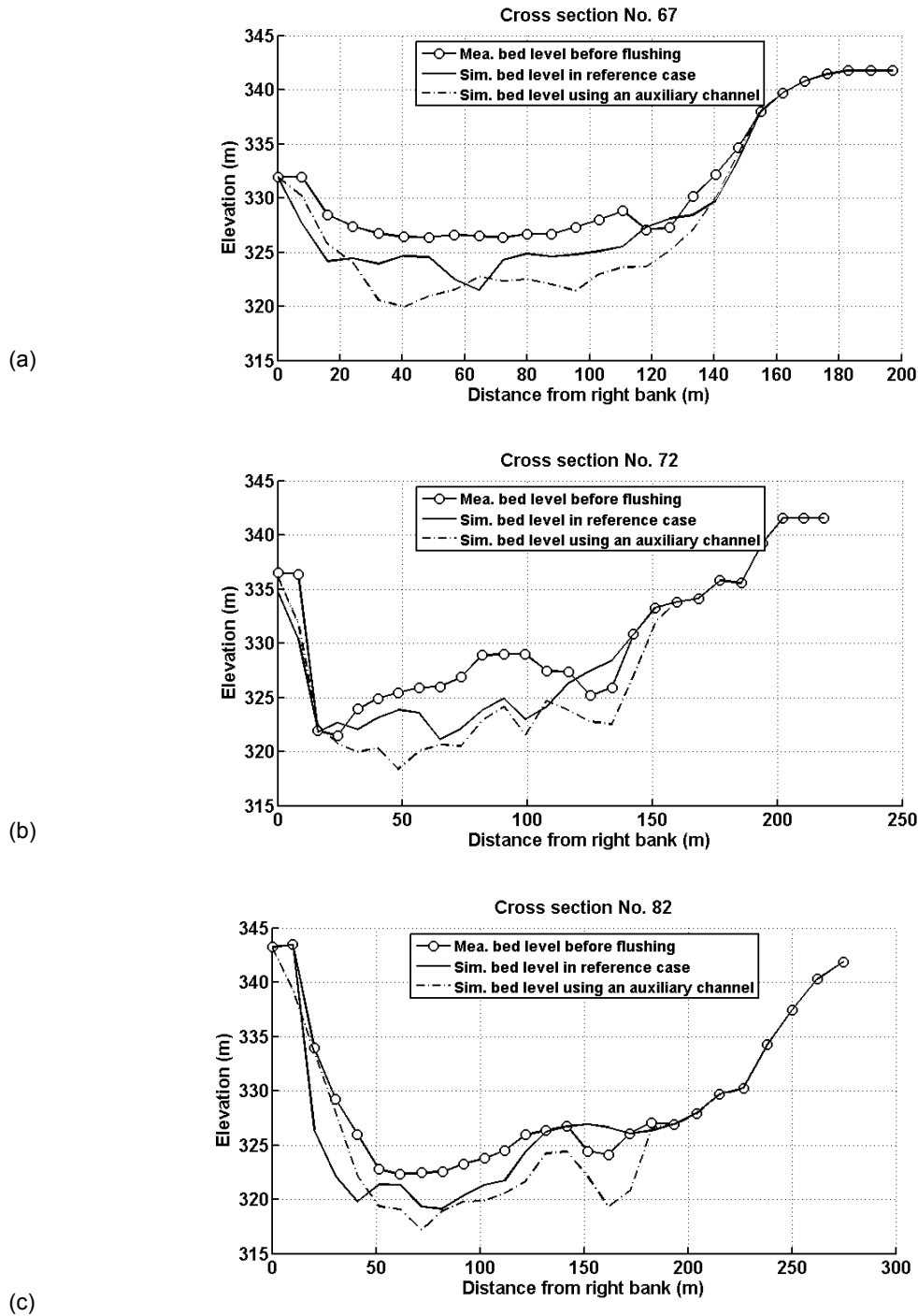


Figure 6.28 Measured bed level before flushing along with the simulated one after flushing in the reference case, and also when the auxiliary flushing channel is utilized at the location of a) cross section F-F; b) cross section G-G; c) cross section H-H. Location of the sections can be found in Figure 5.8.

Chapter 7 Conclusions and recommendations

This study mainly focused on the 3-D numerical modelling of flushing in reservoirs. Since shallow flows play a significant rule during the free-flow flushing, in the first step, hydrodynamic and sediment transportation in physical model of rectangular shaped reservoirs with shallow flows were considered for numerical modeling to figure out the governing processes during the flushing in simplified conditions (e.g., simple geometry and hydrodynamic boundary). In the second step, 3-D numerical model was applied to simulate the free-flow sediment flushing in Dashidaira and Unazuki reservoirs located across the Kurobe River in Japan. Dashidaira reservoir has been constructed earlier than Unazuki reservoir and in contrast to Unazuki resevoir, its bed has been reached to equilibrium condition. In addition, the shape of Dashidaira reservoir is narrower than Unazki reservoir and flushing operation is very efficient in this reservoir except in one wide area. On the contrary, flushing efficiency is low especially in upstream half where the bed materials could not be eroded remarkably and coming sediments from Dashidaira reservoir are partailly deposited here that is not favorable for dam owners. In the final step, measures and scenarios focusing on modifying the discharge and water level rates (i.e., soft measures) and those focusing on reservoir bed manipulation (i.e., hard measures) were suggested for both reservoirs to assess their effects on increasing the flushing efficiency in target areas of each study case.

Here, the outcomes and conclusions obtained from each aforementioned step are presented. Then, practical recommendations based on the current study results for further development of the study are provided accordingly.

7.1 Conclusions

The following results were acheived regarding the flow field simulation in recangular shallow reservoirs under clear water condition :

- Different flow patterns, and consequently velocity distribution patterns, may lead to different morphological processes in the reservoir. 3-D numerical modelling enables us to simultaneously simulate a wide range of flow patterns (i.e., shallow to deep) with a resolution over the flow depth that is necessary to capture 3-D features of the flow (e.g., changing the velocity magnitude and direction over the flow depth, circulation zones in the vertical direction, and upwelling or downwelling fluid movements) when they emerge.
- In this study, various aspects and hydrodynamic characteristics of shallow waters such as jet trajectory, recirculation zones, eddies and flow distribution pattern, were represented by

the numerical model on both flat and misshaped beds in different experimental geometries setup. Two set of hydraulic parameters (e.g., water depth, Froude number, roughness) corresponds to each bed type (i.e., flat or misshaped) was used in the simulations. Similar to the observations, the model reproduces both symmetric and asymmetric flow patterns in the symmetric geometry setup of the reservoirs after introducing a slight disturbance in the inflow boundary condition (i.e., perturbation) for all cases with flat bed. More specifically, 2.5% difference of the velocity magnitude in one side of the inlet channel compared to the other side represented the perturbation in the inflow boundary condition. The numerical results show that the flow pattern is insensitive to the small disturbance in the inflow boundary condition of the geometries with lower defined shape factors. In this condition, the numerical model converges to a steady symmetric flow pattern. In contrast, the calculated flow pattern in geometries with higher defined shape factors converges to a steady asymmetric flow pattern. Regardless to the bed condition, the computed flow velocity magnitudes are reasonably consistent with the experimental observations. In some geometry setups with an asymmetric flow pattern and a lower defined shape factors, contribution of other hydraulic parameters (i.e., bed roughness) beside the perturbation was observed. However, Froude number in all mentioned cases were smaller than 0.2 and different flow pattern may be developed for higher Froude numbers.

- Some discrepancy exists between the numerical results and the experimental measurement in the upstream vortex dimensions and flow field in this area on both bed types (i.e., flat and misshaped). This discrepancy is mainly because of the simulated longer flow and reverse flow jet pattern, and also due to the simulated concentrated flow and reverse flow jet pattern with lower flow diffusion in the numerical outputs. The lower diffusion can be attributed to the application of the standard $k-\epsilon$ turbulence model. Besides, bed and side walls may have small roughnesses due to the fabrication process or installation setting of the reservoir stands while they were neglected in numerical modelling for flat beds. In case of misshaped beds, the varying bed friction over the reservoir geometry may increase the discrepancy.

The following results were obtained concerning the simulation of flushing channel formation in rectangular shallow reservoirs :

- When the water level is drawdown significantly in a short time, two erosion patterns are initiated (i.e., progressive and retrogressive). Thus, the water level drawdown rate (i.e., drawdown speed) remarkably affects the initiation of erosion patterns that forms the flushing channel. Progressive pattern propagates from inlet towards the downstream and retrogressive one from outlet towards the upstream. The progressive pattern was faster than retrogressive one. While the eroded flushing channel by the progressive pattern is stretched, deepened, and widened rapidly due to the strong jet flow and subsequent erosion, the retrogressive pattern develops due to a lateral velocity beside the outlet that

contributes in development of a local half cone just in the vicinity of the outlet. Thus, it can be concluded that the progressive pattern plays a more important role in both sediment erosion and then sediment transportation compared to the retrogressive one when free-flow condition exists.

- Results disclosed that difference between the computed and measured total volume of flushed out sediment is between 18%-39%, when flushing channel location is along the center line, showing the capability of 3-D numerical model for reasonably representing the free-flow flushing process in physical models. Extracted numerical results regarding the flushed out sediment discharge reveal, first, a sharp rising limb in a shorter time period up to reaching a maximum value and then a milder recession limb in a longer time period. Rising limb corresponds to gathering up the erosion caused by progressive and retrogressive pattern and recession limb corresponds to decreasing rate of flushing channel stretching, widening and deepening as flushing channel development is progressing. In other words, as the flushing channel is stretched, widened, and deepened, the flow velocity and the consequent erosive forces are reduced. After formation of the flushing channel over the whole reservoir, this channel attracts the coming flow into the reservoir whereas the channel cannot be noticeably developed in lateral direction using this flow. Therefore, introducing an additional discharge during the free-flow condition can increase the erosive forces, and of course the sediment erosion, which can further develop the flushing channel geometry. However, concentration of flow just in the flushing channel contributes in deepening rather than widening that means deposited sediments outside the main flushing channel (i.e., flood plains) cannot be flushed out properly. For real cases, water level will increase if the introduced additional discharge during the free-flow condition is high enough that can enhance the erosion and transportation of already deposited materials from the flood plain that may result in a further widening of the flushing channel.

Through the simulation of free-flow flushing process in Dashidaira reservoir, the following conclusions are obtained :

- Sensitivity analysis of TVFS to the mainly empirical parameters in the Dashidaira reservoir showed when a larger amount of bed material can be eroded in one time step (i.e. assuming a thicker active layer), a higher volume of erosion from deposits is expected. Moreover, assuming a higher water content in the unit volume of bed material, decreases the submerged density of the bed material, and leads to a higher sediment entrainment. Both mentioned outcomes means that erodibility of the bed material would be higher when the consolidation rate of bed material is lower. Active layer thickness less than 3 times the coarsest bed material size (i.e., about 1 m) and water content of the bed material in the range of 40%-50% showed reasonable performance to represent the Total Volume of Flushed out Sediment and bed geometry changes.

- From the sensitivity analysis, it was found that a higher critical angle of repose keeps a steeper side bank of the flushing channel after each time step of simulation that results in a further deepening of the channel, which is not an efficient approach for increasing the TVFS. In contrast, a lateral development of the flushing channel, that is favorable for increasing the TVFS, can be achieved by a lower critical angle of repose. Side bank erosions can be modeled properly, utilizing the explained sand slide algorithms, if there is no cohesion influence between sediment particles that means they can move independently from each other on a side bank. The critical angle of repose smaller than 33 degrees can be considered for the whole reservoir according to the used approach in this study for modeling the side bank erosion.
- Both MPM and Van Rijn formulas showed a satisfying performance in simulation of bed changes in specific segments of the reservoir. While using MPM formula showed a better prediction of bed changes in the upper half of the reservoir, using Van Rijn formula revealed a better performance in area close to the dam. Upstream areas of the reservoir are covered by the coarser sediment fractions and areas close to dam are deeper and mainly covered by the finer fractions. Each sediment transport formulas has been developed empirically to calculate the sediment transportation for a certain range of sediment sizes and hydrodynamic boundary conditions. Nonetheless, the bed sediment size distribution, bed roughness, and hydrodynamic boundary condition change dynamically during the free-flow flushing process. Such significant changes cannot be handled by empirical sediment transport formulas due to their inherent inabilities, which introduce some uncertainties for simulating the sediment transport during a flushing event. However, MPM bed load formula qualitatively and quantitatively showed a better performance compared to the Van Rijn formula for the whole reservoir. It is possible to reach more than 75% of the measured TVFS using MPM formula and meanwhile it is also possible to represent a general agreement between the simulated and measured bed topography changes after a flushing event.
- Bed forms (e.g., dunes and ripples) which are developed over the computational domain is taken into account by an empirical formula that is solely appropriate for small Froude numbers. Consequently, the alluvial roughness could not be estimated appropriately, which introduces an additional uncertainty for representing the morphological bed changes during the flushing simulation.
- Finer sediments tend to be eroded and flushed out earlier than coarser ones during a flushing operation. In addition, the concentrations of smaller finer sediments are sensitive to both water level and discharge fluctuations. On the other hand, coarser sediments are more sensitive to water level changes rather than discharge variations. Thus, coarser bed materials are mainly flushed out at the end of drawdown stage and during the free-flow stage. One can conclude that when water level just starts to decrease in drawdown period,

coarser sediments start to move further downstream. Afterwards, at the end of drawdown stage and during the free-flow stage, coarser fractions transported nearby the bottom outlets are flushed out. Lengthening the free-flow stage with enough discharges, can contribute in increasing the erosion and transporation of coarser sediments. This measure can be reinforced with introducing water level fluctuations, especially during the free-flow stage, to enhance the sediment remobilization and tranportation chance of coarser sediments.

- Considering the studied cases in the literature and also the current cases, it can be concluded that SSIIM program has an acceptable performance for simulating the sediment transport during the flushing operation as an extreme and complex event. Nevertheless, a robust verification of the empirical and numerical parameters are required for each case owing to the lack of the measured input data with a high resolution (e.g., vertical and lateral sediment size distribution of the bed materials over the whole computational domain) and also due to the simplifications in the model (e.g., assuming the outflow discharge equal to the inflow discharge). Therefore, this program can be used for simulating the simpler events such as sediment deposition in a reservoir even during a long period. Utilizing an implicit solver in SSIIM makes it possible to use large time steps (i.e., compared to the well-known Curren criteria) and it is one of the main advantages of SSIIM program. As long as large number of inner iterations is set for each time step in time dependent camputations, for stability and convergence issues, large time steps can be employed. Moreover, SSIIM program can be used for simulating the sediment transportaion process in downstream channel of the reservoir to figure out the effect of different reservoir operation conditions on morphological bed changes during a specific time period (e.g., after a flushing operation). Then, outcomes can be used for optimizing the river engineering works in downstream channel of the dam as a part of the frame work of comprehensive sediment management (e.g., detecting the areas prone to the side bank erosion, or assessing the sediment replenishment effects).

Simulation of free-flow flushing process in Unazuki reservoir, highlights the conclusions below :

- The study area of Unazuki reservoir (i.e., the upstraem half) is wider compared to the other parts that makes the flow condition shallower here, especially during the free-flow stage. Since bed has not been reachd to equilibrium condition and there is no a distinctive flushing channel to attract and stabilize the flow in this area, flow instability takes place and flow deviates from left side to the right side of the study area as a consequence of small disturbances during a flushing operation. This flow instability occurs when free flow stage is initiated and results in formation of irrigrular and distributed scouring channels in the numerical outputs. The arrangement of these scouring channels can not be observed explicitly from the bed topography measured almost a long time after the flushing. Thus, it is important to measure the bed topography soon after the flushing operation to have a

more accurate assessment of morphological bed changes caused by the flushing in the study area. This condition is pronounced for Unazuki reservoir, where the bed topography in the study area (i.e., upstream half) changes dynamically during and after the flushing.

The conclusions below are achieved based on the numerical application of different type of scenarios and measures in Dashidaira and Unazuki reservoirs aimed to enhance the Flushing Efficiency (i.e., FE):

- Amongst the applied soft scenarios, introducing an artificial additional discharge during the free-flow stage is more reasonable from different aspects for reservoirs located across the Kurobe River. Practically, artificial extra discharge can be supplied from reservoirs located upstream and because it is introduced when the flushing gates are fully opened and the water level is low, it can be passed through the bottom outlets if its value is less than the maximum capacity of the outlets. This extra discharge will increase the induced bed shear stress and bed erosion, and supplies an additional driving force to transport the eroded sediments further downstream of the reservoir to finally flush out them from the reservoir. The extra discharge also causes the water level to increase in downstream river channel, which can be beneficial from environmental point of view through washing the fine materials from the downstream channel terraces. However, it was found that introducing the extra discharge in the form of two scattered discharge pulses with a bigger discharge pulse placed in the second half of the free-flow stage would be more efficient to enhance the FE and meanwhile increase the bed degradation in upstream areas covered with coarser materials. This result is mainly dominant in case of Dashidaira reservoir, which can highlight the role of driving forces to further transport the eroded bed materials further downstream to flush out them from the reservoir. In case of Dashidaira reservoir, introducing about 20.5% additional water from upstream reservoirs (i.e., about 56% increase in the average free-flow discharge) can enhance the FE about 4%-13% compared to a reference case, depends on how this additional discharge is used. Since numbers were extracted after comparing to a reference case, application of same scenarios for other reference cases (e.g., for different years operation) and redraw the graph with more results, will provide a more accurate range for numbers (i.e., number of increase in the flushing discharge magnitude and FE).
- If a flow guiding structure (e.g., groyne) is constructed in a suitable place in Dashidaira reservoir, as a type of hard measure, it can contribute in increasing the flushing efficiency through : 1- changing the main flushing flow direction and consequently developing a new secondary flushing channel to scour the deposited sediments from the target dead zone of the reservoir, and 2- causing bifurcation and unification of main and secondary flushing channel that locally shapes wider and longer flushing channel in the affected area. If groyne just changes the flushing channel location, the Total Volume of Flushed out Sediment (i.e., TVFS) may not be increased significantly and the advantage would be solely a higher

erosion from the target dead zone area. Bifurcation happens due to the existing original flushing channel thalweg that can attract flow from the secondary flushing channel at the downstream of groyne. Unification of original flushing channel and the newly formed secondary channel result in a total longer and wider flushing channel, which enhances the flushing efficiency. Thus, placement of the groyne is very important to have a proper performance of this structure. If bifurcation and unification process can be developed through installing the groyne, it was shown that FE can be enhanced about 10% in one test case, since TVFS increased while used water volume remained constant.

- An auxiliary longitudinal flushing channel along the dead zone area of the Dashidaira reservoir, causes a portion of the flushing flow to deviate from the main channel into the auxiliary channel and again to enter the main channel by a confluence further downstream of the diversion point. In the meantime, other portion of the flow continue its original path over the thalweg of the main flushing channel. Likewise the case with a flow guiding structure, the mentioned processes caused by the auxiliary longitudinal channel result in a total longer and wider flushing channel. Hence, the flushing efficiency increases. It was shown that FE can be enhanced about 9%, in one test case, since TVFS increased while used water volume was constant.
- Introducing an artificial additional discharge during the free-flow stage in the Unazuki reservoir will contribute in changing the dominant morphological process from deposition to erosion in the study area if its magnitude is high enough (i.e., the additional discharge should be about 75% of the average discharge during the free-flow stage). Thus, total used water volume for the flushing operation should be increased about 11% in Unazuki reservoir to change the dominant morphological process. If additional discharge becomes bigger than the mentioned value, both TVFS and FE increase sharply due to the excessive erosion. For instance, if the introduced additional discharge magnitude reaches about 87% of the average discharge during the free-flow stage, FE and TVFS increases about 126% and 124% respectively. Nonetheless, application of the extra discharge in the form of two scattered discharge pulses with a bigger discharge pulse placed in the second half of the free-flow stage was not efficient except when the discharge pulses are too big (e.g., first pulse equals to 300 m³/s discharge for three hours and the second pulse equals to 600 m³/s for two hours) that can be practically difficult to perform due to the limitation of the capacity of bottom outlets. This pronounces the necessity of providing sufficient duration for free-flow stage with an enough discharge to supply the required driving forces aimed to erode and flush out the sediments.

7.2 Recommendations for future developments

- The version of SSIIM program that uses the structured grid (i.e., SSIIM 1) is recommended to use for numerical simulation of experimental cases or real case studies with simple geometries where a huge water level variation does not take place. Mainly because

wetting/drying algorithms have not been included. Due to the type of utilized approaches for computing the free-water surface, the applicability domain is mainly subcritical flows. This version is also simpler to understand, apply, modify, and also extract the outputs for analyzing. Other version of the SSIIM program that uses the unstructured grid (i.e., SSIIM 2) is recommended to use when significant water level changes exist in the computational domain (e.g., reservoir flushing or refilling). The wetting/drying algorithms have been included and subsequently the lateral movement of the computational domain can be simulated in the plan view. In addition, the applied advanced algorithms for computing the free-water surface also enables the model to deal with critical flows. Thus, this version is suitable for modelling the flow and sediment transportation in complex and natural geometries of real cases. Nevertheless, more robust and stable approaches are recommended to develop for SSIIM 2 to increase its performance for computing the free-water surface when supercritical flows with Froude numbers much more bigger than unity emerge on movable beds.

- Behaviour of shallow flows on both type of beds (i.e., flat and misshaped) is required to be analyzed experimentally, and numerically using 3-D models when Froude number is higher than 0.2. Conducting this analysis will enrich this field of study since the considered cases here were under Froude numbers lower than 0.2. Analyzing shallow flows behaviour in a wider range of Froude numbers would be useful for practical purposes when the shallow flow condition emerges under different flow regimes (i.e., filling or emptying the reservoir of low-head power plants). Contribution of other hydraulic parameters, such as roughness, beside the perturbation should be assessed on development of asymmetric flow patterns also under different flow regimes.
- 3-D numerical model is suggested to simulate the shallow-flow behaviour with suspended load under different flow regimes (e.g., different Froude numbers) in physical model studies. Since sedimentation is the main cause for conduction the flushing, this approach will provide a broader insight to manage the sedimentation in preferential zones for practical cases. Through further generalizing the outcomes, it would be possible to prepare a simple guideline aimed to optimally design sand trap facilities or reservoirs of low-head power plants to increase or decrease the sedimentation.
- To reduce the existing discrepancy between the simulated and measured flow velocity field in shallow reservoirs presented in this study, changing the constant empirical values in standard k - ϵ turbulence model (e.g., c_μ) is proposed. Changing these values may contribute in modeling a higher diffusion that may present a more similar flow pattern to the measured one.
- Although MPM formula showed a reasonable performance in this study, it is recommended to set other sediment transport formulas in 3-D numerical model that are suited for simulating the sediment transportation on steep slopes, especially for transporation of the

coarser fractions of sediments. Assessing the pros and cons of a set of sediment transport formulas developed for sediment transportation on steep slopes as well as developing appropriate formulas for modeling bed forms under higher Froude numbers will increase the quality of represented morphological bed changes caused by a flushing event.

- 3-D numerical simulation of sedimentation is proposed mainly for case of Unazuki reservoir to monitor the morphological bed changes and sedimentation characteristics since this reservoir has not been reached completely to equilibrium condition and the bed morphology dynamically changes in the study area. This would be beneficial to assess the morphological bed changes of the reservoir after conducting the annual free-flow flushing operation until the next year's operation. As a result, effect of some measures to increase or decrease the sediment deposition in specific areas can be assessed. This approach works as a complementary measure in addition to the measures for enhancing the flushing efficiency to conduct a more comprehensive sediment management plan in this reservoir. For instance, effect of flow guiding structures or auxiliary flushing channels can be assessed on sedimentation condition in the reservoir. Monitoring the morphological bed changes and sedimentation characteristics are recommended for newly constructed reservoirs, with or without scheduled flushing, aimed to assess the effect of implementing some soft or hard measures on sediment management issues in the reservoir. For example, it would be possible to assess the consequence of sediment yield reduction strategies from watershed area (e.g., using upstream sediment trapping structures) or modifying the dam operation rule curves on sedimentation in the reservoir. However, incoming sediment load data or at least reasonable estimation of such data would be necessary accordingly.
- Taking aerial photos with reasonable time intervals from different areas of the reservoir during the flushing operation, and also measuring the surface velocity using available techniques (e.g., LSPIV) together with measuring the bed topography data are recommended to perform. Aerial photos are useful for qualitative assessment of the flow direction and for recognizing the flushing channel location in different stages of flushing. This type of real time-based data can be very useful for calibration purpose of the 3-D numerical model at the basic stage. Performing the surface velocity measurements together with the bed topography data collection, can be employed for calibrating the 3-D numerical model using the flow hydrodynamics. In case of Unazuki reservoir, measured bed topography data immediately after performing the flushing operation will be helpful to assess the numerical outcomes more accurately because bed morphology will be disturbed significantly in the study area long time after the flushing operation (e.g., one month later) due to the reservoir management tasks (e.g., partial drawdown or sediment sluicing). After calibration of the model using measured surface velocity, it can be recalibrated with the coupled computation of flow and sediment field during the flushing operation through

comparing the measured bed changes caused by flushing with the simulated one. This process, results in a more accurate calibration process.

- Since progressive erosion pattern, widening, and deepening of the flushing channel have significant effect on Total Volume of Flushed out Sediment (i.e., TVFS), close attention should be paid on these factors to enhance the Flushing Efficiency (i.e., FE). When the problem is flushing a higher volume of coarser bed materials, progressive pattern should be reinforced in far upstream areas for two purposes : 1- increasing the erosion of coarser bed materials, and 2- further transporting them inside the reservoir towards dam to increase their flushing chance. Owing to the higher impact of water level changes on mobilization of coarser bed materials, it is recommended to reinforce the progressive pattern with increasing the drawdown speed during the second half of drawdown stage through introducing an extra drop in the water level for a short period (e.g., 1 hour) and then keeping the original drawdown rates. Besides, introduction of small water level fluctuations during the free-flow stage is proposed for further remobilization and transportation of coarser bed materials. However, if there are some environmental and technical limitations to do so, it is recommended to implement additional flushings in both Dashidaira and Unazuki reservoirs even with partial drawdown condition beside the annual free-flow flushing operation. This can be performed with additional supplied water from upstream reservoirs and may have two advantages : 1- since the consolidation rate of bed materials is lower, their erodibility is higher that means lower discharge magnitude can supply enough induced bed shear stress to erode the bed materials. 2- coarser bed materials can be transported further downstream in the reservoir towards the bottom outlets that may increase their chance to be flushed out during the subsequent free-flow flushings.
- When environmental implications are highlighted and also enough water can be supplied from upstream reservoirs, application of additional discharge during the free-flow stage is proposed to enhance the flushing efficiency. Although this approach showed a higher contribution on flushing out the finer bed materials, it causes the water level to rise in river channel downstream of a dam that can further contribute on washing the trapped fine materials on channel terraces. Thus, negative impacts due to the additional flushed out fine materials can be offset to some extent. To enhance the flushing of coarser materials when additional discharges are introduced the following measures are suggested : 1- using couple of discharge pulses during the free-flow stage with bigger discharge pulses in the second half of free-flow stage that may contribute in further transporting the coarser materials towards dam to finally flush out from the reservoir. 2- employing small water level fluctuations even when discharge pulses are working, which facilitates the remobilization of coarser sediments for further transportation towards the bottom outlets. 3- In case of Unazuki reservoir, supplying an additional discharge during a longer free-flow stage is recommended to enhance the eroded volume of the sediments from the upstream areas (i.e., target area in this study).

- In case of the study area of Unazuki reservoir, the applied soft measures in this study can not contribute in remarkable erosion from the upper half (i.e., reservoir entrance zone). The problem is further magnified due to the reservoir geometry in this area that is widen at the channel bend. Thus, it is suggested to mainly focus on employing the hard measures (e.g., construction of auxiliary longitudinal channels) to increase the sediment erosion from this area. Nonetheless, employing a constant additional discharge during the free-flow stage (i.e., a soft measure) can be considered together with the used hard measures.
- Application of flow guiding structures and auxiliary longitudinal channels are strongly recommended after conducting an intense numerical and experimental study on performance of these hard measures under different hydrodynamic conditions with different geometrical arrangements (e.g., shape, size and location). Moreover, their influence should be checked during the sedimentation period to figure out wheather they make adverse affect or not (e.g., increasing the local erosion or deposition). In addition, it should be taken into account that their construction and maintenance implies high costs. However, if the urgent remedy for a local problem is necessary (e.g., excessive deposition in the dead zone of Dashidaira reservoir), and performance of these measures can be convinced to some extent through preliminary studies, it is recommended to use these hard measures temporarily with a lower cost and afterwards, implement the main construction after being sure that they can satisfy the intended purpose.

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List of published papers

Journal papers:

- Esmaeili T, Sumi T, Kantoush SA, Haun S and Rüther N (2015) Three-dimensional numerical modelling of flow field in shallow reservoirs. *Proceedings of the Institution of Civil Engineers-Water Management* (Available online, DOI: 10.1680/wama.15.00011).
- Esmaeili T, Sumi T, Kantoush SA, Kubota Y and Haun S (2015) Numerical study on flushing channel evolution, case study of Dashidaira reservoir, Kurobe River. *Journal of Japan Society of Civil Engineers, Ser. B1 (Hydraulic Engineering)*, **71(4)**: I_115-I_120.
- Esmaeili T, Sumi T and Kantoush SA (2014) Experimental and numerical study of flushing channel formation in shallow reservoirs. *Journal of Japan Society of Civil Engineers, Ser. B1 (Hydraulic Engineering)*, **70(4)**: I_19-I_24, DOI: 10.2208/jscejhe.70.I_19

Conference papers:

- Esmaeili T, Sumi T, Kantoush SA, Kubota Y and Haun S (2015) Three-dimensional numerical modeling of sediment flushing: case study of Dashidaira reservoir, Japan. In *E-Proceedings of the 36th IAHR World Congress, Delft- The Hauge, the Netherlands*.
- Esmaeili T, Sumi T, Kantoush SA and Schleiss AJ (2014) Three dimensional numerical modelling of flow field in rectangular shallow reservoirs. In *Proceedings of the 7th River Flow conference, Lausanne, Switzerland* (Schleiss et al. (eds)). CRC Press, London, UK, pp. 11-19.

Appendix A Measured cross sections in Dashidaira reservoir

Locations of the measured cross sections before and after the flushing operation of June 2012 in the Dashidaira reservoir, and measured cross sections geometry.

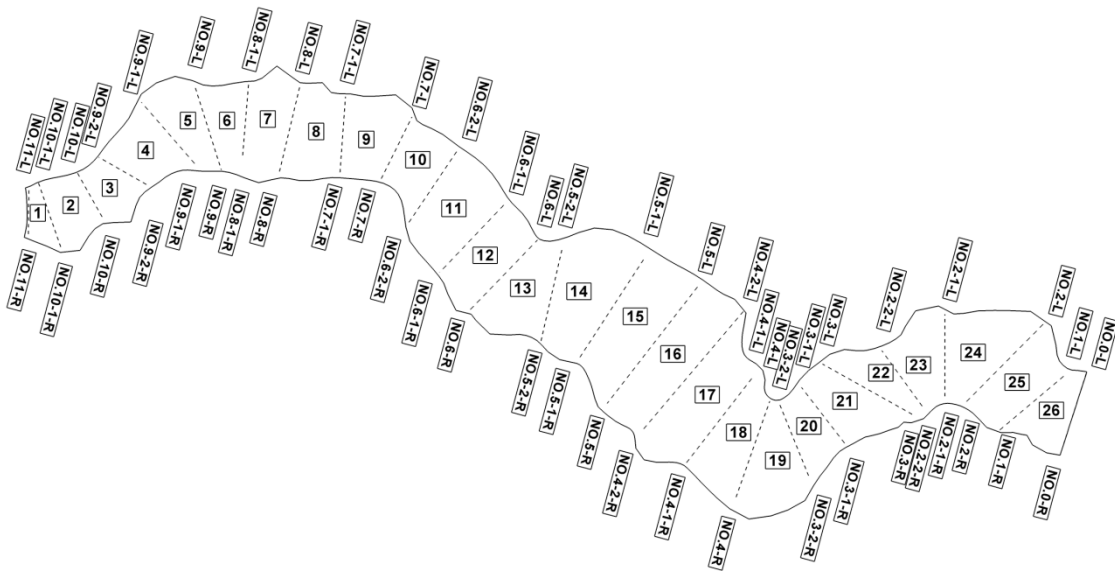
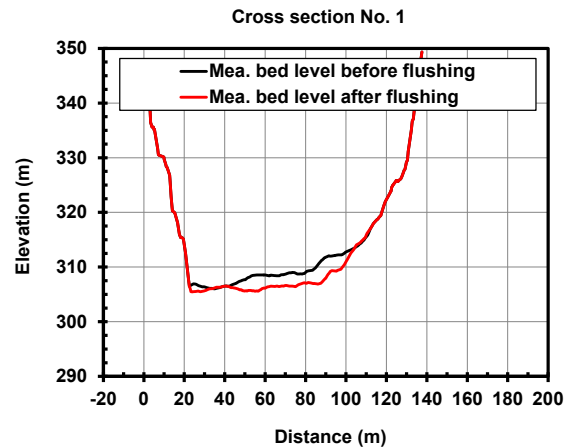
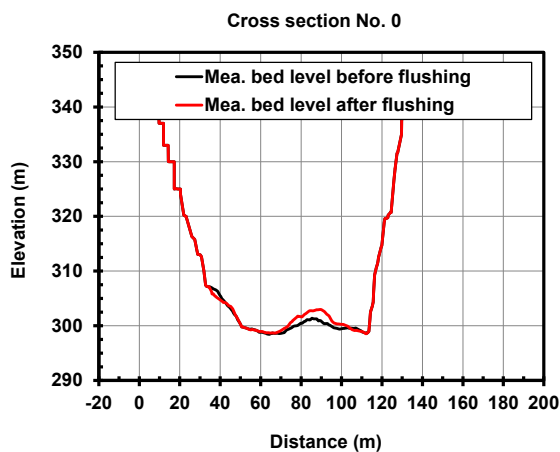
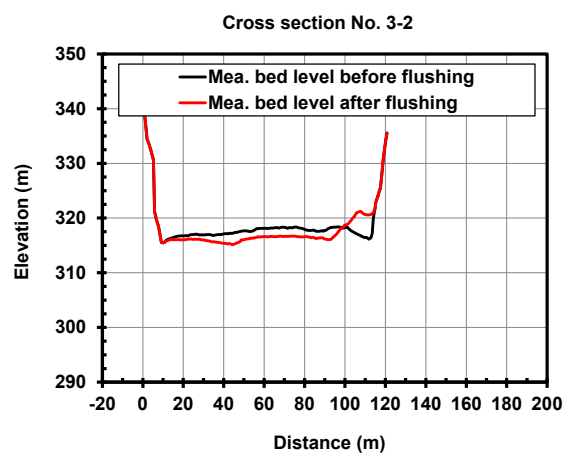
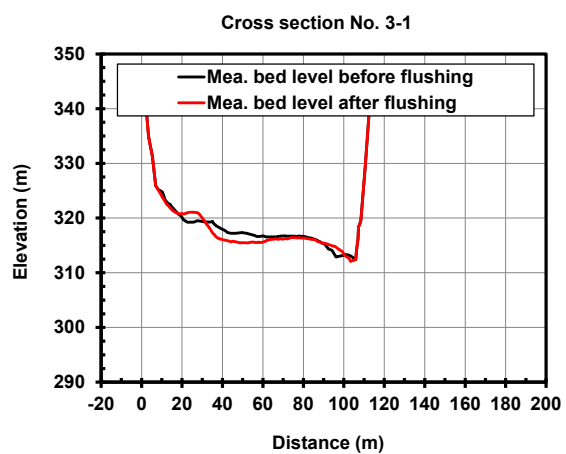
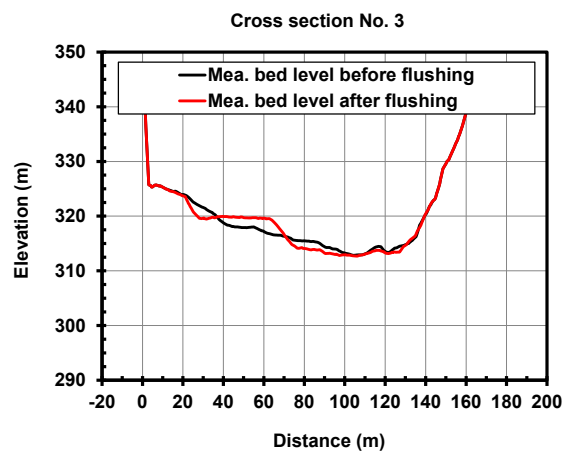
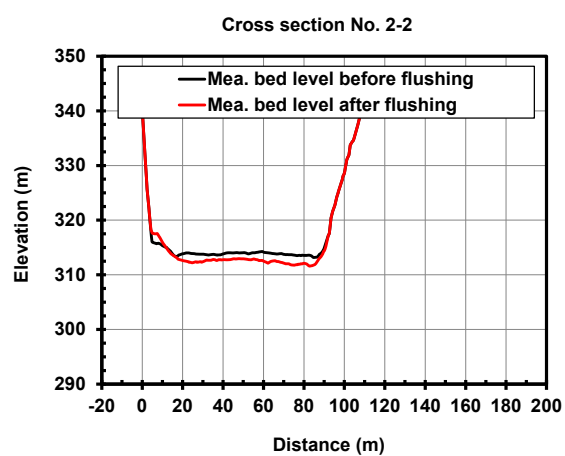
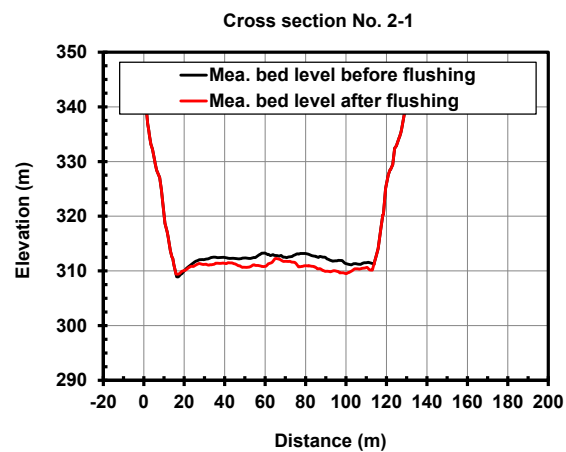
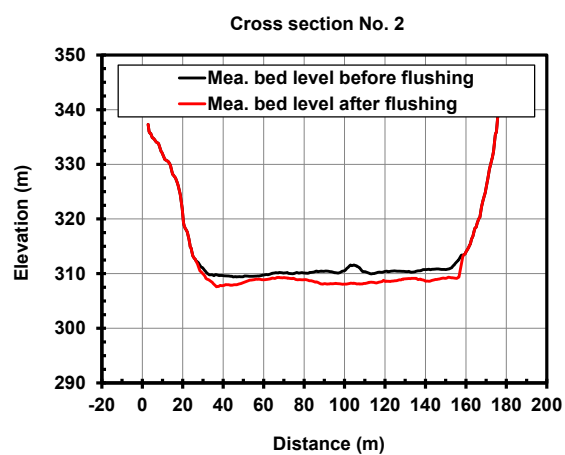
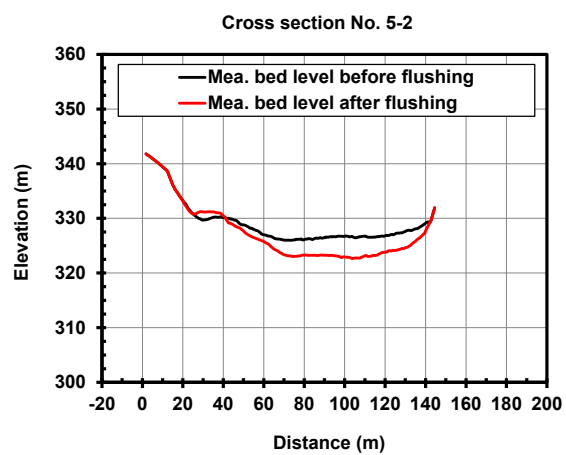
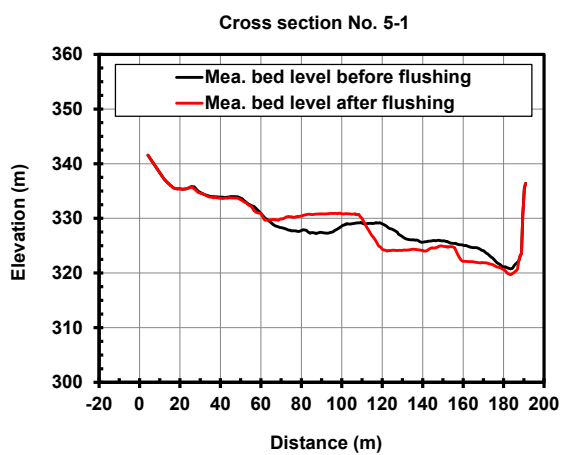
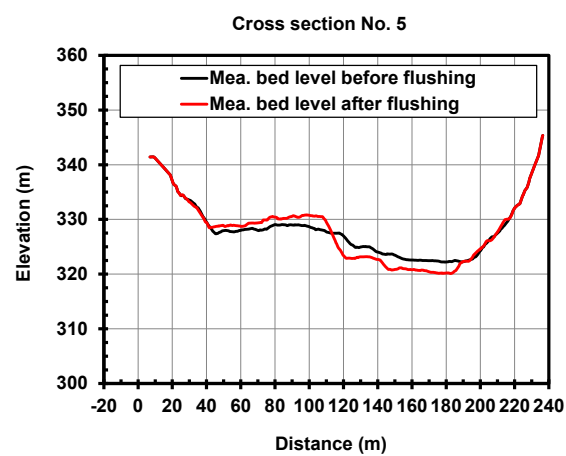
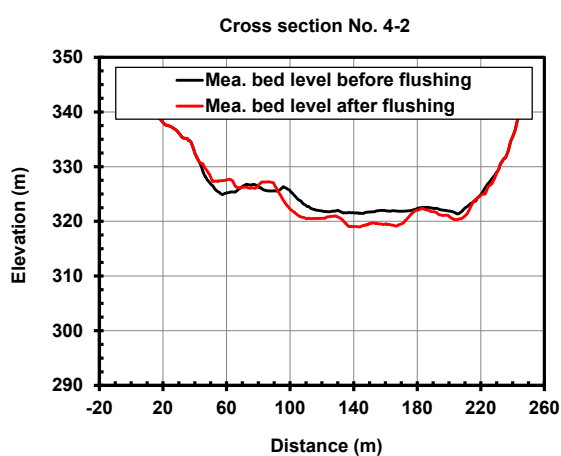
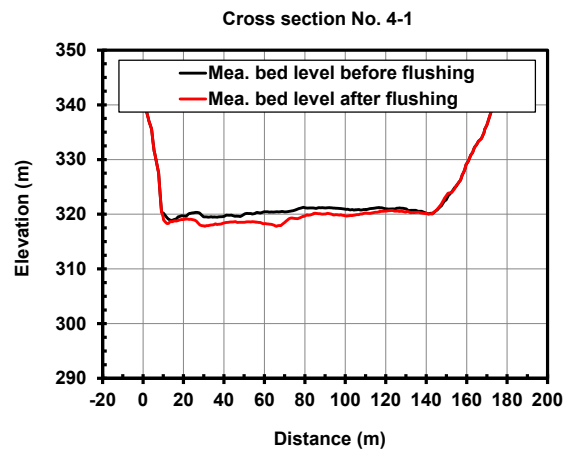
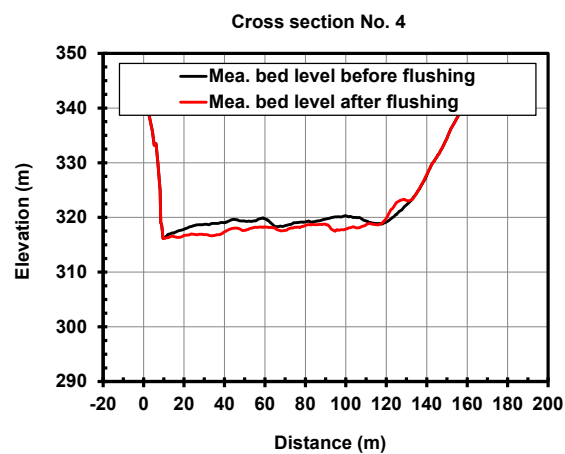
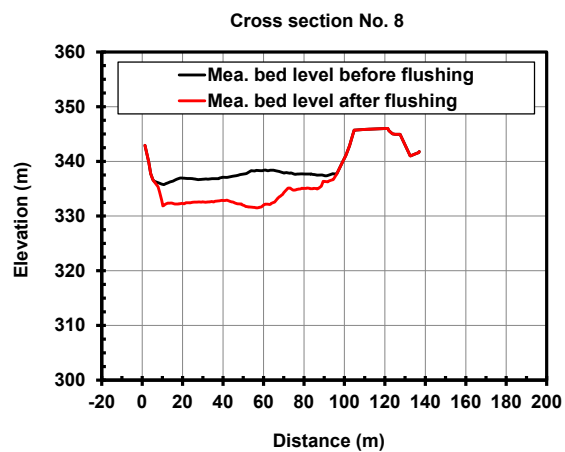
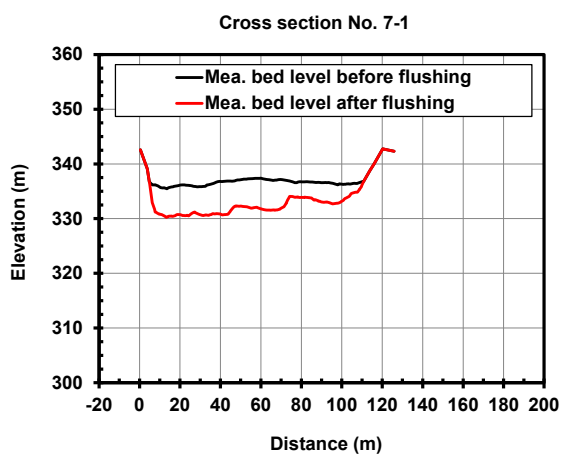
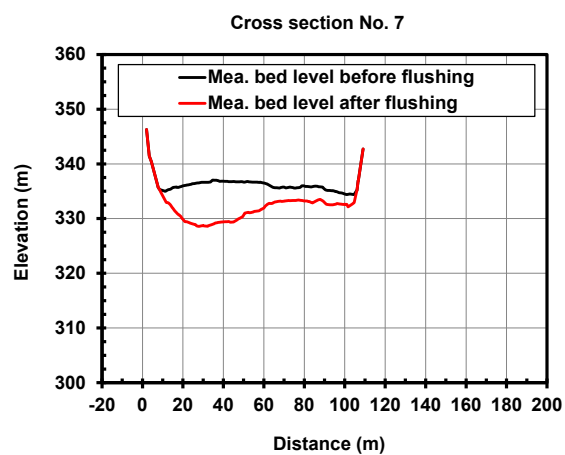
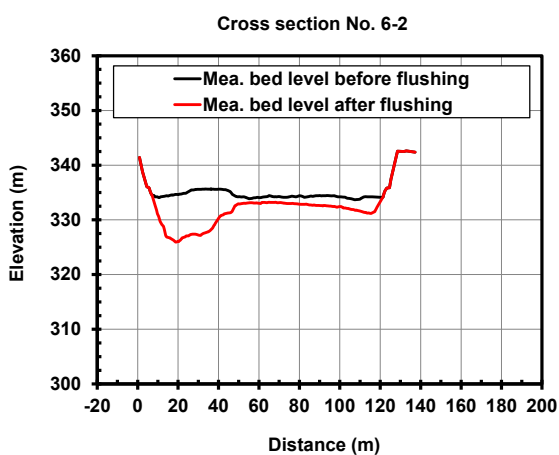
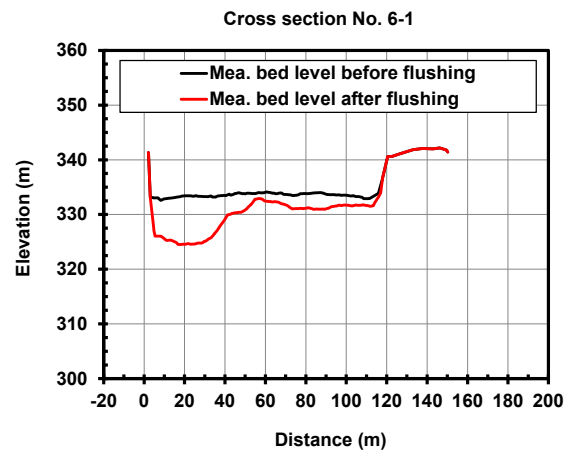
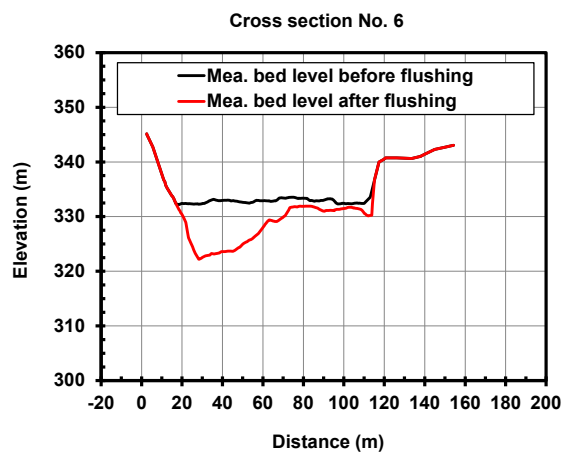


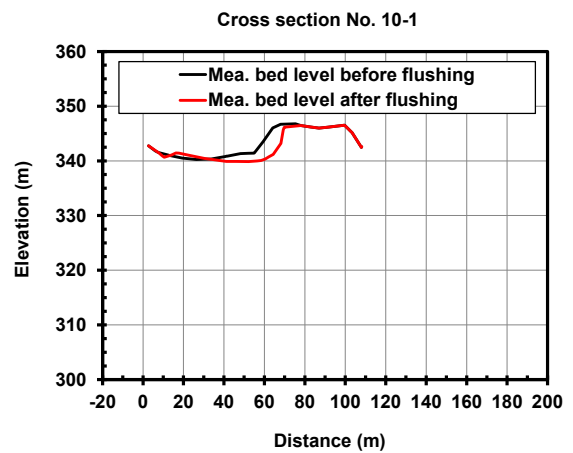
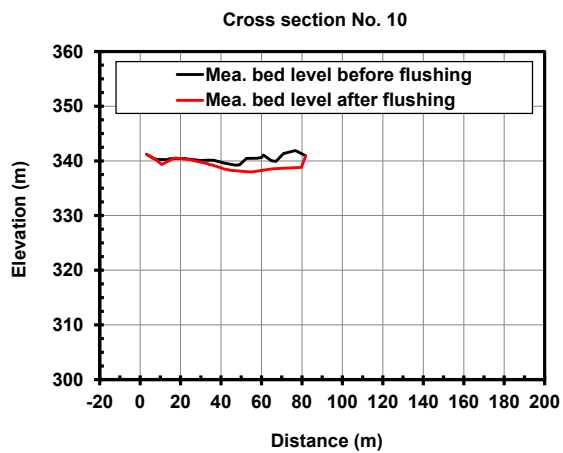
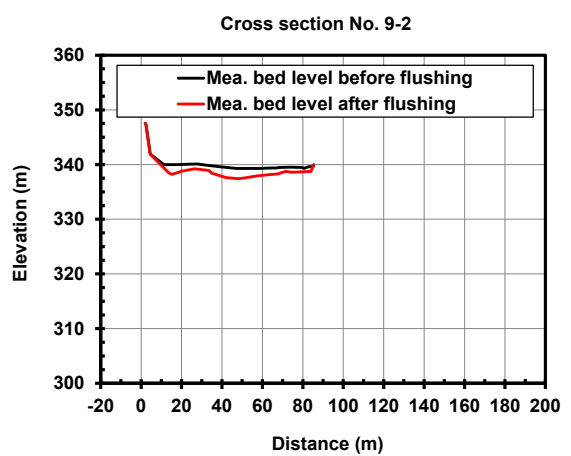
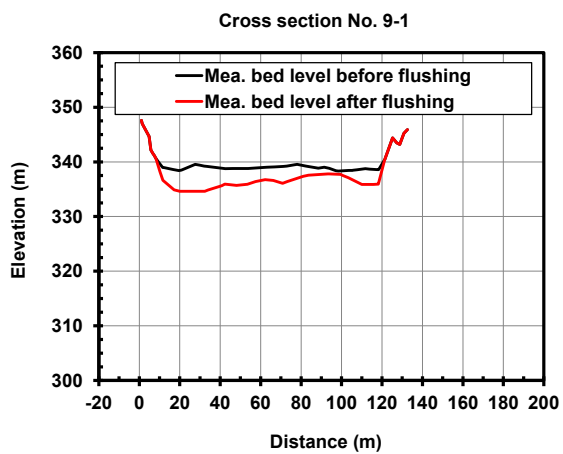
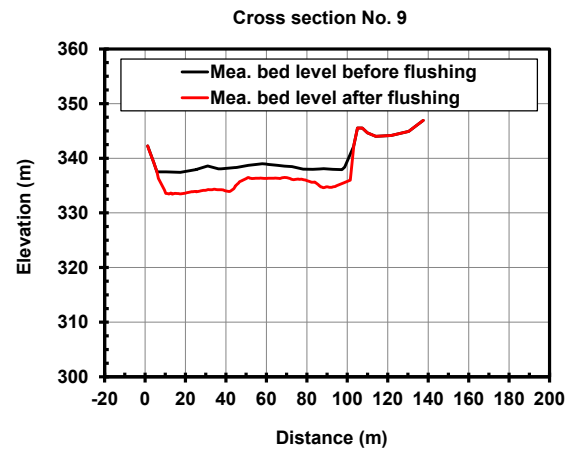
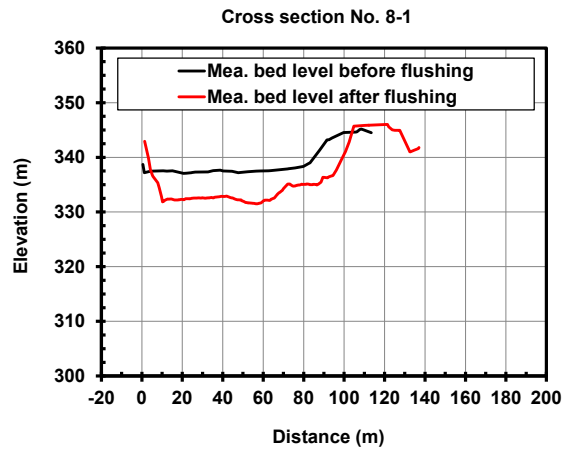
Figure A.1 Plan view of the location of measured cross sections before and after the flushing operation of June 2012 in the Dashidaira reservoir. Tags between each two cross sections show segment numbers where identical non-uniform sediment size distribution was used.











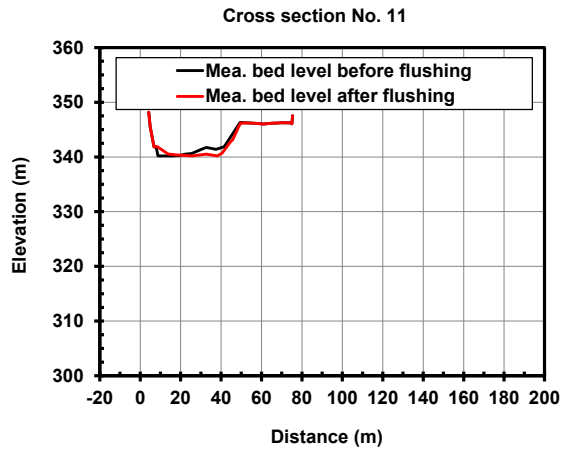
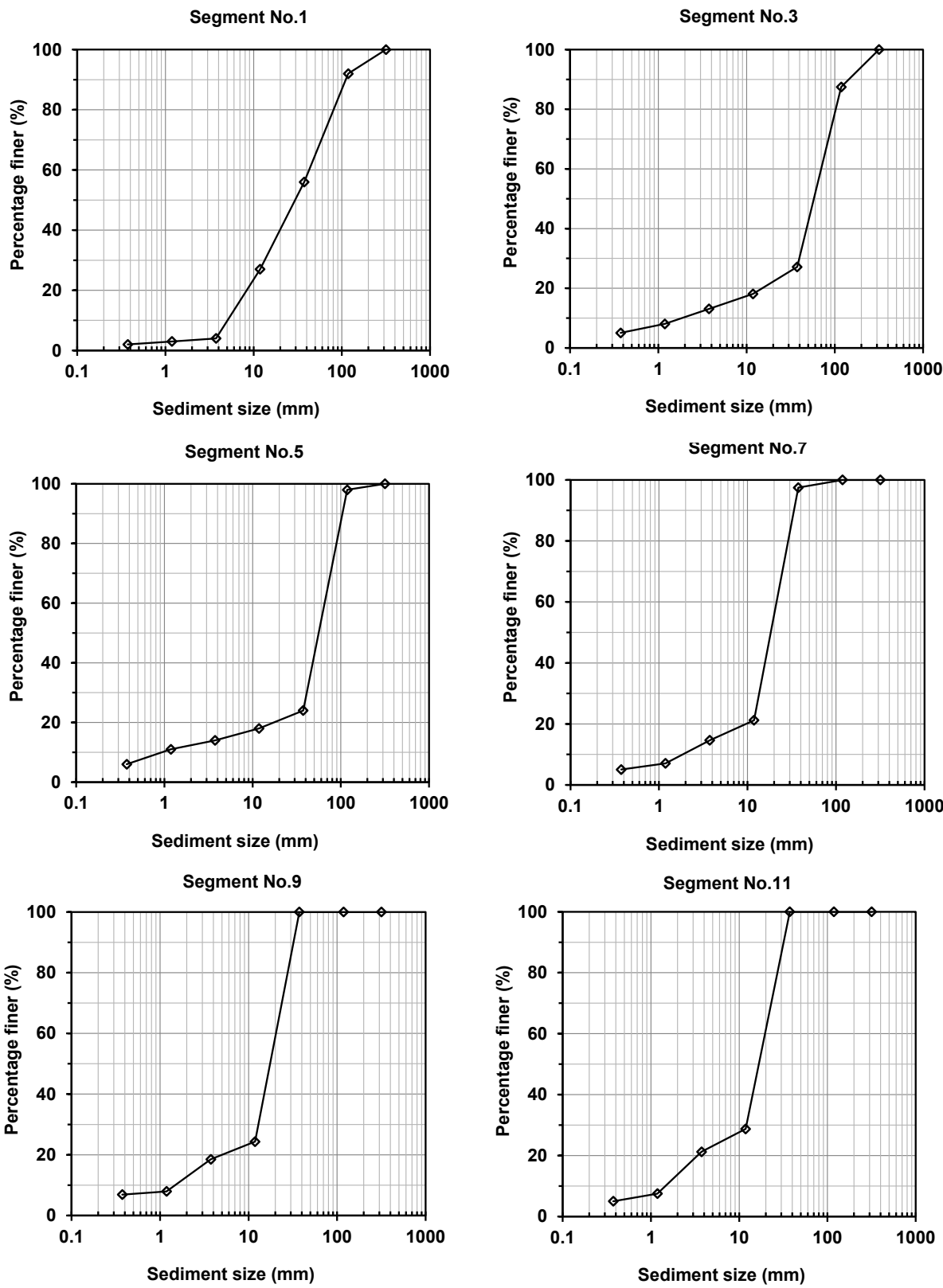


Figure A.2 Measured cross sections before and after the flushing operation of June 2012 in the Dashidaira reservoir.

Appendix B Sediment size distribution in the Dashidaira reservoir

Average sediment size distribution in the selected segments of Dashidaira reservoir, with identical non-uniform sediment size distribution, shown in Table 5.1.



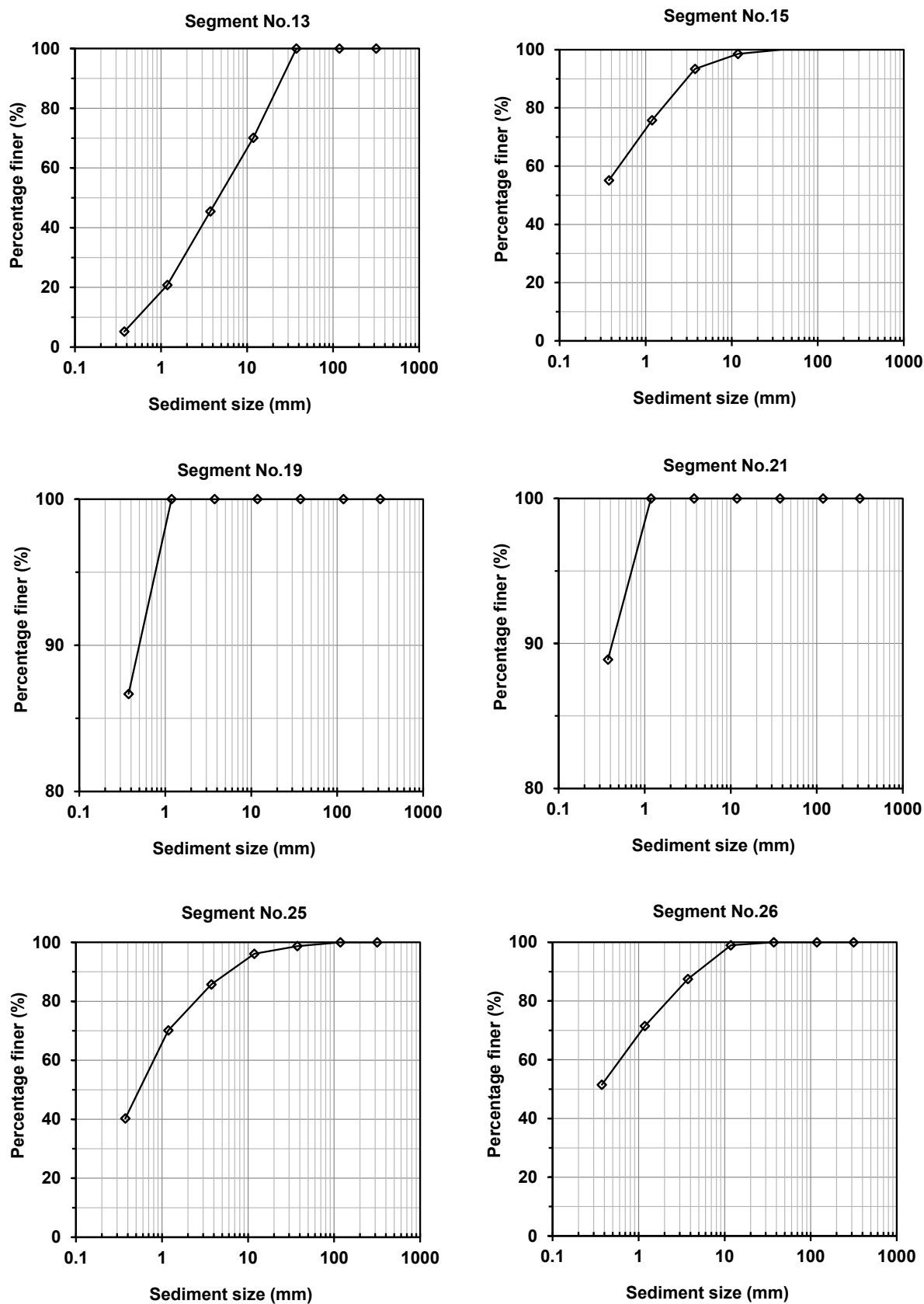


Figure B.1 Sediment size distribution extracted using onsite bed samples in the selected segments of Dashidaira reservoir shown in Figure A.1.

Appendix C Measured cross sections in the study area of Unazuki reservoir

Locations of the measured cross sections before and after the flushing operation of June 2012 in the Unazuki reservoir, and measured cross sections geometry.

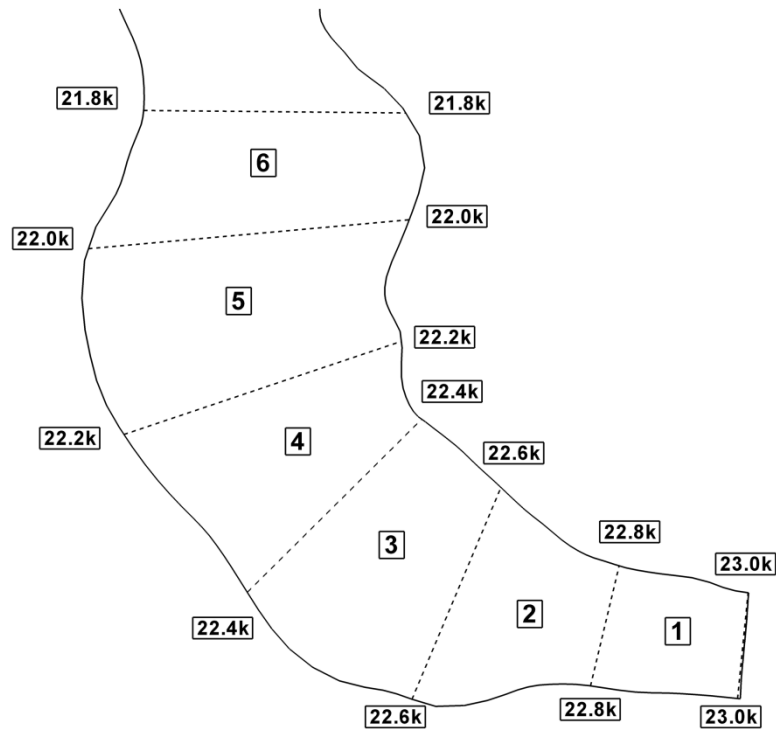
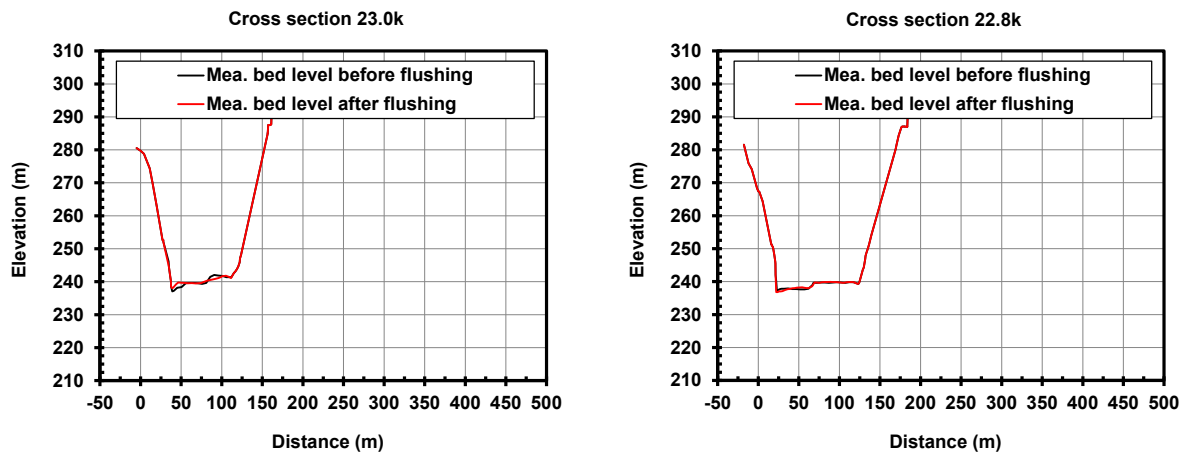


Figure C.1 Plan view of the location of measured cross sections before and after the flushing operation of June 2012 in the Unazuki reservoir. Tags between each two cross sections show segment numbers where identical non-uniform sediment size distribution was used.



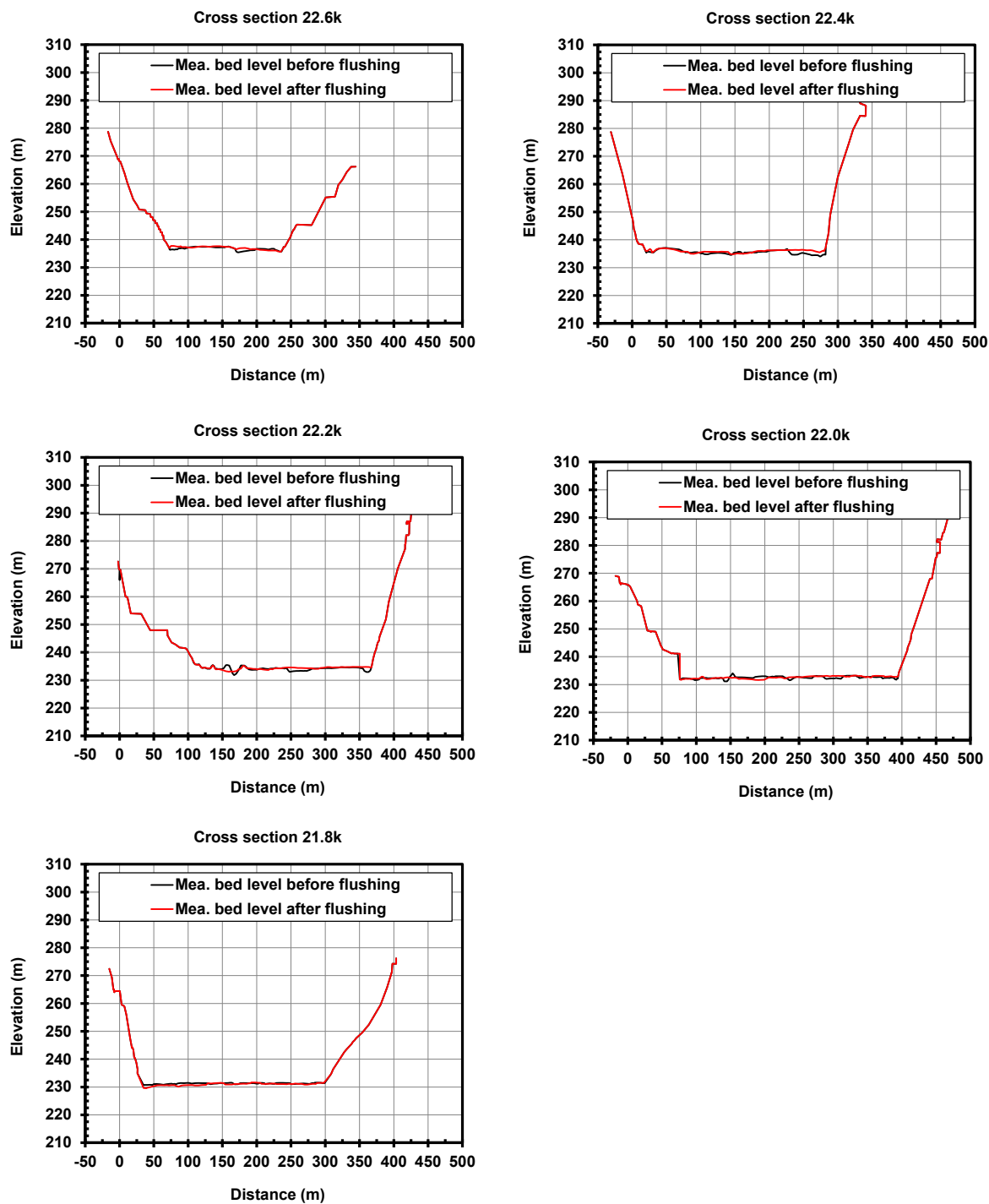
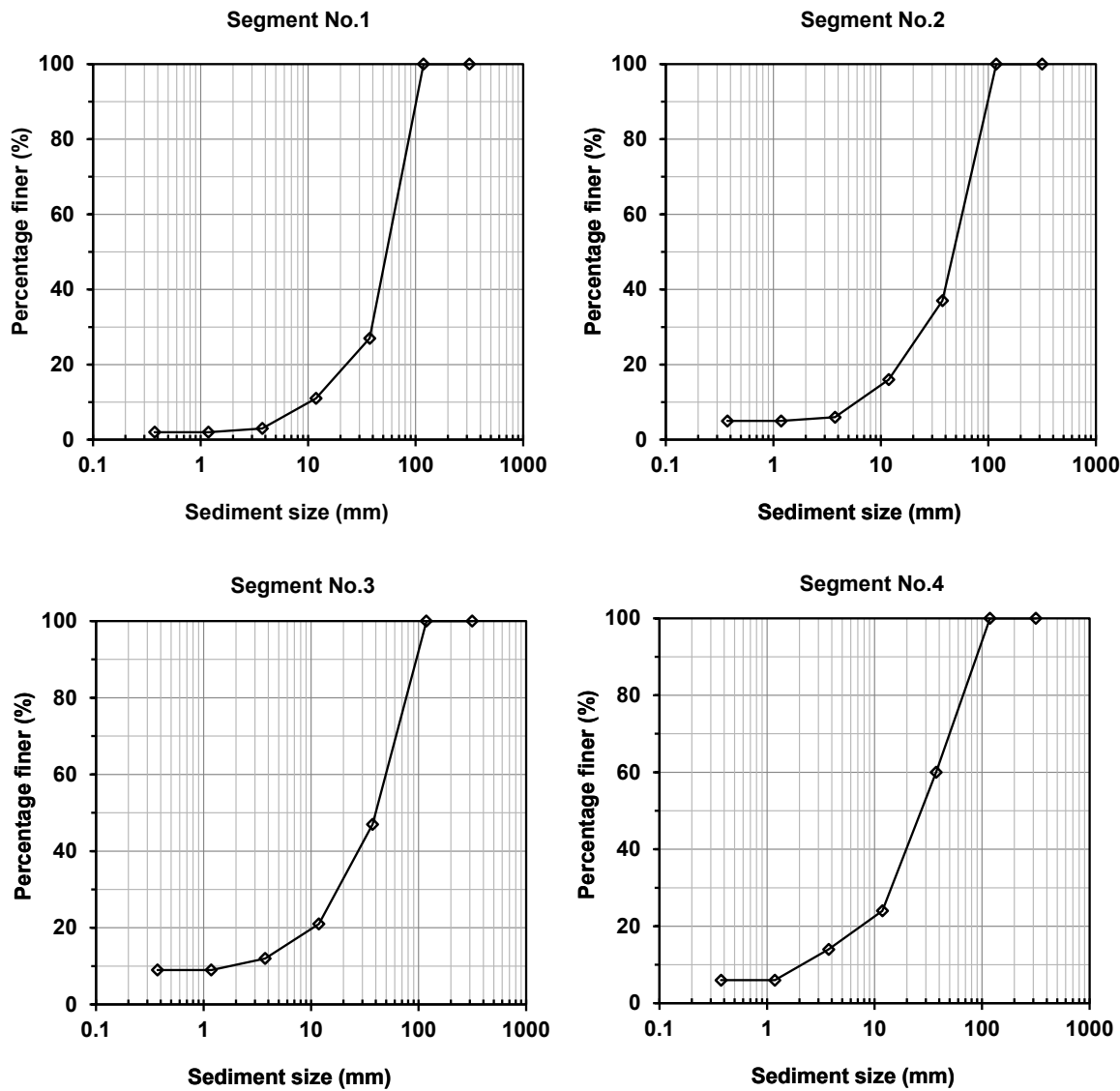


Figure C.2 Measured cross sections before and after the flushing operation of June 2012 in the Unazuki reservoir.

Appendix D Sediment size distribution in the study area of Unazuki reservoir

Average sediment size distribution in the selected segments of Dashidaira reservoir, with identical non-uniform sediment size distribution, shown in Table 5.2.



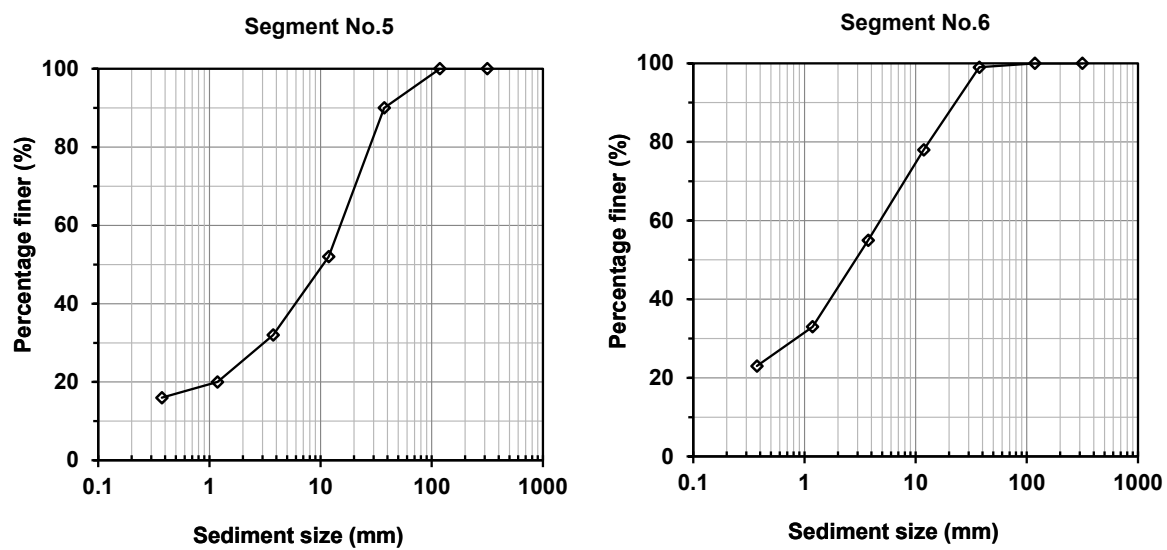


Figure D.1 Sediment size distribution extracted using onsite bed samples in the selected segments of Unazuki reservoir shown in Figure C.1.