Sediment Control In Alluvial Rivers Using Simulation-Based Optimization

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SEDIMENT CONTROL IN ALLUVIAL RIVERS USING SIMULATION-BASED OPTIMIZATION

A Thesis
presented in partial fulfillment of requirements
for the degree of Master in Engineering Science
in the National Center for Computational Hydroscience and Engineering, School of Engineering
The University of Mississippi

Moustafa Abdelkader Elgohry

August 2013
ABSTRACT

During storm events in rivers and watersheds, the peak runoff may creates excess erosion and deposition in channels or river reaches and cause changes in flow characteristics and fluvial geomorphology. Severe sediment erosion such as in-stream sever bed and bank erosion or gullying can threaten the stability of in-stream hydraulic structures, river banks, levees, and underground utilities. On the other hand, large amounts of sediment deposition can significantly reduce the flow capacity of channel or reservoir and increases further possibility of flooding. Therefore, sediment control is generally required for rivers and watersheds conservation purposes to maintain stable flow systems in channels and river reaches.

This study proposes an innovative optimization procedure to control sediment in alluvial rivers during extreme events based on the integration of sediment transport model with optimization approach. The aim of this study is to develop a decision making method which to minimize morphological changes in alluvial networks due to extreme events (e.g. floods and dam removals) under operational constraints so that the optimal sediment control can be achieved.

The developed model combines an optimization module with a well-established one-dimensional model (CCHE1D) for simulating open channel flows and sediment transport in alluvial rivers. An adjoint sensitivity model for CCHE1D and an optimization algorithm are developed to search for the best solution of the optimal control action. The developed model will be applied to control morphological changes by diverting both sediment and water during flood
or dam removal. It is believed that the developed tool will facilitate planning and management of sediment control.
DEDICATION

This work is dedicated to my wife, Dr. Mona Haron and my advisor, Dr. Yan Ding, without whose tireless encouragement I would have given up long ago.
ACKNOWLEDGMENTS

First of all, I would like to express sincere appreciation to Dr. Yan Ding, Research Associate Professor of NCCHE, the chair of my committee and my advisor, for his tremendous advising and support. He spent incredible amount of his time to guide me through every stage of my research. There were countless meetings and discussions regarding my study and research. He inspired me not only in my academic work, but also in my personal life and career.

I would like to express my deep gratitude to Dr. Mustafa Altinakar, Director of NCCHE and Dr. Sam Wang, Barnard Distinguished Professor and Emeritus Director and Founder of NCCHE, for their tremendous support technically and emotionally. Without their support and help, I would not be able to finish this work.

I am very grateful to Dr. Yafei Jia, Research Professor and Associate Director of NCCHE, and Dr. Dr. Weiming Wu, Research Associate Professor of NCCHE and every member of NCCHE for their invaluable understanding, support and help.

Last but not least, I would like to thank my wife, Dr. Mona Haron, for tremendous encouragement and support to complete this work. Without her sacrifice, I would give up.

This thesis is a result of research sponsored by the USDA Agriculture Research Service under Specific Research Agreement No. 58-6408-7-236 (monitored by the USDA-ARS National Sedimentation Laboratory) and The University of Mississippi.
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CHAPTER I

INTRODUCTION

1.1 Background and Motivation

A watershed is usually defined the area of land where all of the water that is under it or drains off of it goes into the same place, or as a series of ecosystems linked spatially and temporally by the downward flow of water. During storm events in rivers and watersheds, the peak runoff may creates excess erosion and deposition in channels or river reaches and cause changes in flow characteristics and fluvial geomorphology. Severe sediment erosion such as in-stream sever bed and bank erosion or gullying can threaten the stability of in-stream hydraulic structures, river banks, levees, and underground utilities. On the other hand, large amounts of sediment deposition can significantly reduce the flow capacity of channel or reservoir and increases further possibility of flooding. Therefore, sediment control is generally required for rivers and watersheds conservation purposes to maintain stable flow systems in channels and river reaches.

At present, there is a real need for new methodologies that can optimize the selection, design and operation of so-called Best Management Practices (BMPs) channels and rivers to control sediment transport and fate based on simulations of flow stream response during extreme events. BMPs are structural, operational or cultural methods by which sediment transport and
fate is controlled or optimized sufficiently to minimize the morphological changes and meet sediment concentration criteria of water quality. BMPs for sediment control include diverting flow and sediment gates, in-stream dams or weirs, sediment traps, vegetation, and riparian buffers.

1.2 Research Objectives
The major objectives for this research are:

- Development of a simulation-based optimization modeling system for optimal sediment control in alluvial rivers.
- Sediment control for dam-removal management practices.

1.3 Research Significance

This thesis proposes decision making software to control sediment in alluvial rivers during extreme events based on the integration of sediment transport with optimization approach. This research aims at developing a flexible tool for sediment control that can be easily modified and widely applied to different BMPs. It is believed that the developed tool will aid decision maker in planning and management of sediment control BMPs during optimized extreme events.

1.4 Organization of the Thesis

This thesis is composed of eight chapters: the first two chapters are the introduction and literature review. The introduction chapter contains this research background, motivation, objectives and significance, while the literature review chapter reviews the current research on
the topics of watershed management, flow control studies, sediment control studies, simulation models, and optimization methods. Chapter three illustrates the detailed of the proposed methodologies for developing an integrated optimization model to control sediment transport in watersheds. Chapter four shows developed model validation using an experimental case from the literature. Chapter five presents model applications using different cases of morphological change problems. Chapter six represents the application of the developed model to control excess deposition after a dam removal case. Chapter seven introduces the application of the developed model to watersheds through deriving internal boundary conditions at confluences. Chapter eight provides summary, conclusions and recommended future research.
CHAPTER II

LITERATURE REVIEW

2.1 Watershed Management

A watershed is usually defined the area of land where all of the water that is under it or drains off of it goes into the same place, or as a series of ecosystems linked spatially and temporally by the downward flow of water. Watershed Management may be defined as the study of the relevant characteristics of a watershed and the process of creating and implementing plans, programs, and projects to sustain and enhance watershed functions. During storm events in rivers and watersheds, the peak runoff may creates excess erosion and deposition in channels or river reaches and cause changes in flow characteristics and fluvial geomorphology. Severe sediment erosion such as in-stream sever bed and bank erosion or gullying can threaten the stability of in-stream hydraulic structures, river banks, levees, and underground utilities. On the other hand, large amounts of sediment deposition can significantly reduce the flow capacity of channel or reservoir and increases further possibility of flooding. Therefore, sediment control is generally required for rivers and watersheds conservation purposes to maintain stable flow systems in channels and river reaches.

At present, there is a real need for new methodologies that can optimize the selection, design and operation of so-called Best Management Practices (BMPs) at the watershed scale to

4
control sediment transport and fate based on simulations of watershed response during extreme events. BMPs are structural, operational or cultural methods by which sediment transport and fate in controlled or optimized sufficiently to minimize the morphological changes and meet sediment concentration criteria of water quality. BMPs for sediment control include diverting flow and sediment gates, in-stream dams or weirs, sediment traps, vegetation, and riparian buffers.

2.2 Flow Control Studies

Natural rivers morphological changes can be attenuated by applying better flow and sediment management policies at in-stream flow and sediment control structures. Nicklow and Mays (2000) reported that reservoir management release policies can be optimized by minimizing the in-stream deposition heights (e.g., in Yazoo River basin, MS). Diversion works and intakes can also be used in channels with low sediment transport to withdraw water/sediment from the channel flow to return the sediment charge to the channel (Lysne et al. 1995). Nicklow et al. (2003) coupled the U.S. Army Corps of Engineer’s HEC-6 sediment transport simulation model with a genetic algorithm procedure to minimize bed elevation changes in rivers and reservoirs of a large-scale network. Carriaga and Mays (1995) used Differential Dynamic Programming (DDP) procedure limiting their focus to minimizing the sum of aggradation and degradation depths in a single downstream river reach. Ding and Wang (2010) used sensitivity analysis method to control flow during extreme events in alluvial dendritic channels.
2.3 Sediment Control Studies

Natural river morphology is usually in a physical equilibrium state under existing steady flow and sediment conditions. However, excess deposition which may result from a dam removal upstream may reduce the effective channel cross-section downstream and increase the possibility of flooding. On the other hand, lack of sufficient sediment supply from upstream may cause excess erosion downstream. These changes threatens stability of hydraulic structures and underground utilities, may result in navigation difficulties, or increased possibility of flooding. Therefore, in order to effectively mitigate erosion and deposition impacts, an optimal sediment control approaches at structures need to be developed.

2.4 Simulation Models

Generally numerical simulation models are divided into three categories based on the dimensions of the models, i.e. one-dimensional (1-D), two-dimensional (2-D), and three-dimensional (3-D). Apparently, all the process-based models include hydrodynamic models and sediment transport models to compute hydrodynamic variables such as discharges, flow velocities, and water stages, and sediment transport rate, sediment mixing, and morphological changes. 1-D models generally enable to quickly predict cross-sectional discharges, water stages, sediment transport rates through river cross-sections, and cross-sectional area changes over a multiple-year long period and a large-scale watershed. Even though 1-D models can only give cross-sectional averaged predictions, because of computational efficiency and reasonable accuracy, they have been commonly applied to widely to simulate rivers/watersheds morphodynamics. (e.g. Wu et al. 2005, Elgohry et al., 2010).
Some 1-D models can only simulate steady flows, but most 1-D hydrodynamic models enable to compute unsteady flows through multiple flow regimes such as subcritical, supercritical, and transcritical. 2-D models generally are capable of computing temporal and spatial variations of hydrodynamic variables and morphodynamic processes over the entire river reach upstream and downstream. In comparison with 1-D model, 2-D models are computationally expensive and need more data preparation efforts. Lai et al. (2006) have applied a 2-D hydrodynamic and morphodynamic model to assess the response of river morphology such as bridge scours due to the removal of the Sandy River Delta Dam, Oregon. For 3-D models, one may refer to Papanicolaou et al. (2008). In applications of the models, 2-D and 3-D models are best applied to solve local problems associated with morphological changes over a relatively short period (e.g. storm duration).

CCHE1D (Center for Computational Hydrosience and Engineering One Dimensional (1D) model) is a general one-dimensional model, which is effective in the simulation of long term simulation and capable of handling the mixed-regime flows. The model simulates the non-equilibrium transport of uniform, nonuniform, cohesive and noncohesive sediment load under unsteady flow conditions for a single open channel or channel network with complex geometries. The hiding and exposure mechanism in bed material is considered and the non-equilibrium adaptation length \( L_s \), which characterizes the distance for sediment to adjust from a non-equilibrium state to an equilibrium state, is a very important parameter in the non-equilibrium transport model is incorporated.
The process-based model integrates two sub-models: hydrodynamic model, and sediment transport model which are solved using a decoupling procedure, and can be summarized as follows:

The governing equations for 1-D Dynamic Wave model for open-channel flows (The St. Venant Equations)

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} - q_t = 0$$

(1)

$$\frac{\partial}{\partial t} \left( \frac{Q}{A} \right) + \frac{\partial}{\partial x} \left( \frac{\beta Q^2}{2A^2} \right) + g \frac{\partial Z}{\partial x} + gS_f = 0$$

(2)

where $x$ and $t=$spatial and temporal axes; $Q=$flow discharge; $Z=$water stage; $A=$flow Area; $q_t=$lateral outflow per unit channel length; $\beta=$correction factor due to the non-uniformity of velocity distribution over the cross section; $g=$gravitational acceleration; $S_f =$friction slope, defined as $S_f = Q|Q|/K^2$, with $K$ being the conveyance.

The governing equation for the non-equilibrium transport of sediment used in the CCHE1D model is

$$\frac{\partial (AC_i)}{\partial t} + \frac{\partial Q}{\partial x} + \frac{1}{L_s}(Q - Q_s) - q = 0$$

(3)

where $A=$Cross-sectional area of flow; $C_i=$section-averaged sediment concentration; $Q_s=$actual sediment transport rate; $Q_s=$sediment transport capacity or the so-called equilibrium transport rate; $L_s=$non-equilibrium adaptation length of sediment transport; and $q=$side inflow or outflow sediment discharge from bank boundaries or tributary stream per unit channel length. The section-average concentration and the sediment transport rate can be expressed as $C_i = Q_i / (\beta_s AU)$ in which $\beta_s$ is a coefficient accounting for the difference between flow and sediment velocities that may produce time lags, and is assumed to be one here; $U$ is the section-
average velocity. Therefore, the governing Eq. (3) will become

$$L = \frac{\partial}{\partial t} \left[ \frac{Q}{U} \right] + \frac{\partial Q}{\partial x} + \frac{1}{L_s} (Q_t - Q_s) - q = 0$$

(4)

The sediment transport capacity can be written as a general form as $Q_s = Q^*$, where $Q^*$ = potential sediment transport capacity, which can be determined with the help of existing empirical relations, e.g. SEDTRA module (Garbrecht 1995) and Wu et al.’s (2004) formula. However, the results shown in this study are base on applying the latter formula. The bed deformation is determined with

$$\left(1 - p'\right) \frac{\partial A_b}{\partial t} = \frac{1}{L_s} (Q_t - Q_s)$$

(5)

where $p'$=bed-material porosity; and $\left(\partial A_b/\partial t\right)$=bed area deformation rate. For more details, one may refer to Wu et al. (2004).

2.5 Optimization Methods

Due to the nonlinearity of sediment control problems, a numerical optimization methodology needs to be applied. In general, there are three methods to solve optimization problem: (1) conjugate gradient methods (e.g., Fletcher-Reeves method); (2) line search methods (e.g., Limited Memory Quasi-Newton); and (3) trust region methods (e.g., Sakawa-Shindo method) (Nocedal and Wright 1999).

2.5.1 Conjugate gradient methods

The basis for a nonlinear conjugate gradient method is to effectively apply the linear conjugate gradient method, where the residual computed from previous iteration is replaced by
the gradient. The advantage of conjugate gradient methods is that they use relatively little memory for large-scale problems and require no numerical linear algebra, so each step is quite fast. The disadvantage is that they typically converge much more slowly than Newton or quasi-Newton methods. Also, steps are typically poorly scaled for length, so the line search algorithm may require more iterations each time to find an acceptable step.

2.5.2 Line search method

In the line search strategy, the algorithm chooses a search direction $d_k$ and tries to solve the following one-dimensional minimization problem

$$\min_{\alpha > 0} f(x_k + \alpha_k d_k)$$

where the scalar $\alpha_k$ is called the step length. In theory we would like optimal step lengths, but in practice it is more efficient to test trial step lengths until we find one that gives a good enough point.

Quasi-Newton methods are algorithms for finding local maxima and minima of functions. Quasi-Newton methods are based on Newton's method to find the stationary point of a function, where the gradient is 0. Newton's method assumes that the function can be locally approximated as a quadratic in the region around the optimum, and uses the first and second derivatives to find the stationary point. In higher dimensions, Newton's method uses the gradient and the Hessian matrix of second derivatives of the function to be minimized. In quasi-Newton methods the Hessian matrix does not need to be computed. The Hessian is updated by analyzing successive gradient vectors instead.

$$x_{k+1} = x_k - [Hf(x_k)]^{-1}\nabla f(x_k)$$
where $H$ is Hessian matrix of function $f$.

Among Quasi-Newton methods, Limited Memory Quasi-Newton (LMQN) methods have fast converging, numerical stability and modest storage requirements (Ding and Wang 2006). Further, Ding et al. (2004) have concluded that LMQN algorithms can effectively capture the objective parameters with high accuracy in the nonlinear open channel problems. Among LMQN algorithms, the Limited-memory Broyden, Fletcher, Goldfarb, and Shanno Bounded (L-BFGSB) algorithm is capable of optimization of large-scale problem because of its modest storage capacity requirements by using a sparse approximation to the inverse Hessian matrix of the objective function.

2.5.3 Trust region methods

Essentially these methods approximate only a certain region of the objective function with a simpler model function, $m_k$. When an adequate model of the objective function is found within the trust region then the region is expanded. Conversely, if the approximation is poor then the region is contracted. In the line search strategy, the direction is chosen first, followed by the distance, while in the trust-region strategy, the maximum distance is chosen first, followed by the direction.

The model function $m_k$ is usually defined to be a quadratic function of the form

$$m_k(x_k + p) = f_k + p^T \nabla f(x_k) + \frac{1}{2} p^T B_k p$$

where $B_k$ is a matrix, usually a positive definite approximation of the hessian matrix.

The minimization procedure can be summarized as follows:

- pick step $p_k$ to reduce “model” of $f(x_k + p)$
• accept $x_{k+1} = x_k + p_k$ if the decrease promised by the model is inherited by $f(x_k + p_k)$,
• otherwise set $x_{k+1} = x_k$ and improve the model.
CHAPTER III

DEVELOPMENT OF INTEGRATED OPTIMIZATION MODEL TO CONTROL SEDIMENT TRANSPORT IN ALLUVIAL RIVERS

The developed model is coupling an adjoint sensitivity model with a sediment transport simulation model (CCHE1D). Different optimization algorithms have been used to estimate the value of the diverted or imposed sediment along river reach to minimize the morphological changes under different practices and applications.

3.1 Mathematical Formulations

3.1.1 Governing Equations

Referring to section 2.4, the governing equations of CCHE1D flow model are

\[
\begin{align*}
\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} - q_l &= 0 \\
\frac{\partial}{\partial t}\left(\frac{Q}{A}\right) + \frac{\partial}{\partial x}\left(\frac{\beta Q^2}{2A^2}\right) + g \frac{\partial Z}{\partial x} + gS_f &= 0
\end{align*}
\]

where \(x\) and \(t\)=spatial and temporal axes; \(Q\)=flow discharge; \(Z\)=water stage; \(A\)=flow Area; \(q_l\)=lateral outflow per unit channel length; \(\beta\)=correction factor due to the non-uniformity of velocity distribution over the cross section; \(g\)=gravitational acceleration; \(S_f\)=friction slope, defined as \(S_f = Q[Q]/K^2\), with \(K\) being the conveyance.
The governing equation for the non-equilibrium transport of sediment used in the CCHE1D model is

$$\frac{\partial (AC_t)}{\partial t} + \frac{\partial Q_t}{\partial x} + \frac{1}{L_s}(Q_t - Q_{r*}) - q = 0$$  \hspace{1cm} (8)

where $A$=Cross-sectional area of flow; $C_t$=section-averaged sediment concentration; $Q_t$=actual sediment transport rate; $Q_{r*}$=sediment transport capacity or the so-called equilibrium transport rate; $L_s$=non-equilibrium adaptation length of sediment transport; and $q$=side inflow or outflow sediment discharge from bank boundaries or tributary stream per unit channel length. The section-average concentration and the sediment transport rate can be expressed as $C_t = Q_t / (\beta_s AU)$ in which $\beta_s$ is a coefficient accounting for the difference between flow and sediment velocities that may produce time lags, and is assumed to be one here; $U$ is the section-average velocity. Therefore, the governing Eq. (3) will become

$$N = \frac{\partial}{\partial t}\left[\frac{Q_t}{U}\right] + \frac{\partial Q_t}{\partial x} + \frac{1}{L_s}(Q_t - Q_{r*}) - q = 0$$  \hspace{1cm} (9)

The sediment transport capacity can be written as a general form as $Q_{r*} = Q^*_t$, where $Q^*_t$=potential sediment transport capacity, which can be determined with the help of existing empirical relations, e.g. SEDTRA module (Garbrecht 1995) and Wu et al.’s (2004) formula. However, the results shown in this study are base on applying the latter formula. The bed deformation is determined with

$$\left(1 - p^\prime\right) \left[\frac{\partial A_b}{\partial t}\right] = \frac{1}{L_s}(Q_t - Q_{r*})$$  \hspace{1cm} (10)

where $p^\prime$=bed-material porosity; and $(\partial A_b/\partial t)$=bed area deformation rate.
3.1.2 Performance Function

The optimization procedure for finding the optimal solution of a control variable \( q \) in physical system is to minimize the objective function \( J \) which is generally defined as the integration of a general measured function \( f \) over the solution domain, i.e.

\[
J = \int_{0}^{T} \int_{L}^{L} f(A_{b}, q, x, t)dxdt
\]

In this study the measuring function \( f \) is defined as the change rate of bed area, i.e.

\[
f = \frac{W}{LT} \left[ \frac{\partial A_{b}}{\partial t} \right]^{2} \delta(x-x_{0})
\]

where \( W \)=weighting factor; \( L \) and \( T \)=spatial and temporal simulation domain respectively; \( x_{0} \)=the target reach for mitigating the bed area change. By using the relation between the area of bed change and sediment transport rate (Eq. 5), the objective and measuring functions can be rewritten as follows:

\[
J = \int_{0}^{T} \int_{L}^{L} f(Q_{s}, q, x, t)dxdt
\]

\[
f = \frac{W}{LT} \frac{1}{(1-p)^{2}L_{s}^{2}} [Q_{s}(x,t)-Q_{s}^{obj}(x,t)]^{2} \delta(x-x_{0})
\]

3.1.3 Adjoint Sensitivity Analysis

In order to minimize the objective function (Eq. 13) and ensure that the sediment model variable satisfy the governing equation (Eq. 9), an augmented objective function \( J^{*} \) is formed using Lagrangian multiplier \( \lambda_{Q} \) as follows,

\[
J^{*} = J + \int_{0}^{T} \int_{L}^{L} \lambda_{Q} Ldxdt = \int_{0}^{T} \int_{L}^{L} f(Q_{s}, q, x, t)dxdt + \int_{0}^{T} \lambda_{Q}^{N} dxdt
\]
Taking the first variation of the augmented objective function, i.e.

$$\delta J^* = \int_0^T \left( \delta f + \lambda \delta N \right) dx dt$$  \hspace{1cm} (16)$$

where,

$$df = \frac{\partial f}{\partial Q} \delta Q + \frac{\partial f}{\partial q} \delta q$$  \hspace{1cm} (17)$$

$$\delta N = \frac{1}{U} \frac{\partial \delta Q}{\partial t} + \frac{\partial \delta Q}{\partial x} + \frac{1}{L_s} (\delta Q_s - Q_s) - \delta q = 0$$  \hspace{1cm} (18)$$

By using Green’s theorem and the variation operator $$\delta$$ in time-space domain shown in Figure (3.1), the first variation of the augmented function can be obtained

$$\delta J^* = \int_0^T \left[ \frac{\partial f}{\partial Q} \delta Q_i + \frac{\partial f}{\partial q} \delta q \right] dx dt - \int_0^T \left[ \frac{\partial \lambda}{\partial t} + \frac{\partial \lambda}{\partial x} - \frac{\lambda}{L_s} \right] \delta Q_i dx dt - \int_0^T \lambda \delta q dx dt$$

$$+ \int \left[ - \frac{\lambda}{U} \delta Q_i dx + \lambda \delta Q_i dt \right]$$  \hspace{1cm} (19)$$

![Figure 3.1 Solution domain of contour integral.](image)

For minimizing $$J^*$$, $$\delta J^*$$ must be equal zero which means all terms multiplied by $$\delta Q_i$$ must be set to zero which leads to the following equation,

$$\frac{\partial \lambda}{\partial t} + \frac{\partial \lambda}{\partial x} - \frac{\lambda}{L_s} = \frac{2W}{LT} \frac{1}{(1-p)^2 L_s} \left[ Q_i(x,t) - Q_i^{obj}(x) \right] \delta (x-x_0)$$  \hspace{1cm} (20)$$
which is the adjoint equation containing the Lagrangian multiplier $\lambda_q$.

The contour integral in Eq. (19) needs to be zero so as to satisfy the minimum condition of the performance function $J^*$, namely,

$$\oint \left[ -\frac{\lambda_q \delta Q_t}{U} dx + \lambda_q \delta Q_t dt \right] = \int_B (-\cdot) + \int_C (\cdot) + \int_D (\cdot) + \int_A (\cdot)$$

$$= \left[ -\frac{\lambda_q \delta Q_t}{U} \right]_0^L dx + \left[ \lambda_q \delta Q_t \right]_0^T dt - \left[ -\frac{\lambda_q \delta Q_t}{U} \right]_0^L dx + \left[ \lambda_q \delta Q_t \right]_0^T dt = 0$$

(21)

Since inflow sediment discharge must be known at upstream inlet node at all times, namely $Q|_{t=0} = Q_t(0,t), \ t \in [0,T]$ and by taking the first variation at upstream boundary condition, i.e. $\delta Q_t(0,t)=0$ and the integral $\int_{DA} (\cdot)$ vanishes which means $\lambda_q(0,t)$ cannot be determined.

Similarly, initial condition, $Q|_{x=0} = Q(x,0), x \in [0,L]$ and by taking the first variation, $\delta Q_t(x,0)=0$ and the integral $\int_{AB} (\cdot)$ vanishes and $\lambda_q(x,0)$ cannot be determined. Since the values of $Q_t$ at $x=L, \ t=T$ cannot be specified, i.e. $\delta Q_t(L,t) \neq 0$, and $\delta Q_t(x,T) \neq 0$, $t \in [0,T], x \in [0,L]; \lambda_q(L,t) = 0$ and $\lambda_q(x,T) = 0$ to make the integrals of $\int_{BC} (\cdot)$ and $\int_{CD} (\cdot)$ equal zero respectively, i.e. the Lagrangian multiplier at a downstream outlet is $\lambda_q(L,t) = 0, \ t \in [0,T]$ and transversality condition of the Lagrangian multiplier is $\lambda_q(x,T) = 0, x \in [0,L]$. Due to these boundary conditions, the adjoint Eq. (20) must be solved backward in both time and space.

3.1.4 Calculation of Sensitivity of Performance Function

The sensitivities of the control variables for sediment control can be obtained from the
variation of objective function in Eq. (16). The variation of the objective function with respect to the control variable \( q \) at a location \( x_n \) is

\[
\delta J = \int_0^T \left\{ \frac{\partial f}{\partial q_L} \bigg|_{x=x_n} - \dot{\lambda}_Q(x_n,t) \right\} \delta q(x_n,t) dt
\]

(22)

but \( f \) is not a function in \( q \), then the sensitivity at a time \( t_n \) is

\[
\frac{\delta J}{\delta q} \bigg|_{x=x_n}^{t=t_n} = -\dot{\lambda}_Q(x_n,t_n)
\]

(23)

The variable \( \dot{\lambda}_Q \) determines precisely the gradient of the objective function, \( \nabla J(q) \). In case upstream or downstream sediment transport control variables, Eq. (18) will be

\[
\frac{\delta J}{\delta q} \bigg|_{x=0}^{t=t_n} = -\dot{\lambda}_Q(0,t_n), \quad \text{and} \quad \frac{\delta J}{\delta q} \bigg|_{x=L}^{t=t_n} = -\dot{\lambda}_Q(L,t_n)
\]

(24)

respectively.

3.2 Numerical Approaches

The hydrodynamic and non-equilibrium sediment transport equations in CCHE1D model are discretized by using Preissmann scheme (Preissmann 1961) which is an implicit four-point finite difference scheme in time and space. This scheme replaces a continuous function \( f \) and its temporal and special derivatives by

\[
f = \theta[\psi f_{i+1}^{n+1} + (1-\psi)f_i^{n+1}] + (1-\theta)[\psi f_{i-1}^n + (1-\psi)f_i^n]
\]

\[
\frac{\partial f}{\partial t} = \psi \frac{f_{i+1}^{n+1} - f_i^n}{\Delta t} + (1-\psi) \frac{f_i^{n+1} - f_i^n}{\Delta t}
\]

\[
\frac{\partial f}{\partial x} = \theta \frac{f_{i+1}^{n+1} - f_i^{n+1}}{\Delta x} + (1-\theta) \frac{f_{i+1}^n - f_i^n}{\Delta x}
\]

(25)
where $\theta$, $\psi$ are the temporal and spatial weighting coefficients in the Preissmann’s scheme; $n$ is the time step; $i$ is the spatial step number; $\Delta t$ and $\Delta x$ are the step lengths in the time and space.

The same scheme is used to discretize the derived nonlinear adjoint equation (Eq. 15) as follows,

$$\frac{\psi}{U\Delta t}\left[\lambda_{Q,i+1}^{n+1}-\lambda_{Q,i}^{n+1}\right]+\frac{1-\psi}{U\Delta t}\left[\lambda_{Q,i}^{n+1}-\lambda_{Q,i}^{n}\right]+\frac{\theta}{\Delta x}\left[\lambda_{Q,i+1}^{n+1}-\lambda_{Q,i}^{n+1}\right]+\frac{1-\theta}{\Delta x}\left[\lambda_{Q,i+1}^{n}-\lambda_{Q,i}^{n}\right]
-\theta\left[\psi\lambda_{Q,i+1}^{n+1}+(1-\psi)\lambda_{Q,i}^{n+1}\right]-\frac{(1+\theta)}{L_s}\left[\psi\lambda_{Q,i}^{n}+(1-\psi)\lambda_{Q,i}^{n}\right]=\frac{2W_Q}{LT}\frac{1}{(1-p)^2}\left[Q(x,t)-Q_{obj}(x)\right]\delta(x-x_0)$$

(Eq. 26)

Eq. (26) can be written in the general form of a system of algebraic equations as follows,

$$c_1\lambda_{Q,i}^{n}=c_2\lambda_{Q,i}^{n+1}+c_3\lambda_{Q,i}^{n+1}+c_4\lambda_{Q,i}^{n+1}+c$$

(Eq. 27)

where

$$c_1=\frac{(1-\psi)}{U_i}\frac{(1-\theta)}{\Delta x}\frac{(1-\theta)(1-\psi)}{L_s}$$

$$c_2=\frac{\psi}{U_i}\frac{(1-\theta)}{\Delta x}\frac{(1-\theta)\psi}{L_s}$$

$$c_3=-\frac{(1-\psi)}{U_i}\frac{\theta}{\Delta x}\frac{\theta(1-\psi)}{L_s}$$

$$c_4=-\frac{\psi}{U_i}\frac{\theta}{\Delta x}\frac{\theta\psi}{L_s}$$

$$c=\frac{2W_Q}{LT}\frac{1}{(1-p)^2}\left[Q(x,t)-Q_{obj}(x)\right]$$

(Eq. 28)

3.3 Optimization Procedure

Figure 3.2 shows the flow chart of the optimization procedure of proposed optimal sediment control model for finding optimal control variable $q$ by line search algorithm, in which $\alpha_k$ is the step length of line search, and $d_k$ is the search direction (descent direction) and is equal $-\nabla J_k(q)$. 

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Figure 3.2 Flow chart for finding optimal control variable $q$ by LMQN method
CHAPTER IV

MODEL VALIDATION

The developed model has been verified by applying to a channel aggradation experiment performed by Seal et al. (1995) and the results compared with calculated sediment transport rate at specified location.

4.1 Simulation of an Aggradation Problem Due To Upstream Sediment Overloading in an Experimental Case

The channel aggradation experiment performed by Seal et al. (1995) was investigated using the developed model. The flume was 45 m long and 0.305 m wide, with an initial bed slope of 0.002. The tailgate was kept at constant height and input sediment was a mixture comprising a wide range from 0.125 to 64 mm. In simulated experimental run 2, the water discharge was 0.049 m$^3$/s, the sediment feed rate was 5.65 kg/min, and the tailgate water stage was 0.45 m. Figure 4.1 compares the measured and predicted bed profiles at various times, and the water surface profile at the final stage. The bed and water profiles were reproduced well. The results show that there is an aggradational wedge developed and its front gradually moves downstream while the upstream bed elevation continues to rise.
4.2 Sediment Control using Non-optimal Control Approach

To mitigate the excess deposition, a non-optimal control approach has been applied. A sediment diversion gate has been assumed to be at 10 m downstream where the optimal sediment diversion rate \( q(t) \) was identified in order to minimize bed change downstream that location. The non-optimal control approach is based on using the actual sediment transport rate at the gate location as the identified diverted sediment. The results are compared with those of the developed model in Figures 4.2 and 4.3.

Figure 4.1 Measured and simulated water and bed profiles
Figure 4.2 Diverted sediment and bed elevation change results from applying non-optimal control approach and developed model.

Figure 4.3 Iterations of the objective function and the norm of its gradient.
CHAPTER V

MODEL APPLICATION

To demonstrate the applicability and effectiveness of the proposed model, two hypothetical cases have been investigated. The two cases represent an aggradation problem after dam removal and a degradation problem downstream a reservoir due to lack of upstream sediment load.

5.1 A Hypothetical Aggradation Problem after Dam Removal

Elghory et al. (2010) used successfully CCHE1D model to assess the morphological changes after the removal of Marmot Dam, Oregon, USA. In this study, a hypothetical single channel with a downstream aggradation problem after dam removal was investigated by the developed model. The channel had a typical trapezoidal cross section of 10 m bed width and 1:2 side slope, and 0.005 bed slope. The inflow and sediment discharge were assumed 50 m$^3$/s and 10 kg/s respectively. The input and bed sediment was assumed uniform and 20 mm. Figure 5.1b shows that the fate of accumulated sediment in reservoir after one week which has caused aggradation problem downstream. To mitigate the aggradation problem, a diversion gate was set at the removed dam location and the developed model was applied to identify the optimal sediment diversion rate $q(t)$ at the diversion gate. The model results have been compared with a
non-optimal control approach results. Figure 5.1a shows there is a difference in the required diverted sediment rate between the non-optimal control approach and developed model. Figure 5.1b shows the effectiveness of the developed model to eliminate the deposition downstream compared to the case of no-control.

5.2 A Hypothetical Degradation Problem Downstream a Reservoir Due To Lack of Upstream Sediment Supply

A hypothetical single channel with a downstream degradation problem in front of an existing dam due to lack of upstream sediment load. The channel reach and sediment properties are similar to Case 2. The dam release discharge was assumed to be 10 m$^3$/s. Due to the existence of the dam, there is a lack of sediment supply from upstream to downstream the reservoir which cause degradation problem downstream as shown Figure 5.2b. To mitigate the degradation problem, the developed model was applied to identify the optimal release rate $q(t)$ of reservoir sediment which is shown in Figure 5.2a. Figure 5.2b shows the effectiveness of the developed model to eliminate the erosion downstream compared to the case of no-control (only clear water release from reservoir).

The same case has been again tested under stage dam release flow discharge which is shown in Figure 5.3. Due to the existence of the dam, there is a lack of sediment supply from upstream to downstream the reservoir which cause degradation problem downstream as shown Figure 5.5. To mitigate the degradation problem, the developed model was applied to identify the optimal release rate $q(t)$ of reservoir sediment which is shown in Figure 5.4. Figure 5.5 shows the effectiveness of the developed model to eliminate the erosion downstream compared to the case
of no-control (only clear water release from reservoir).

Further, the same case has been again tested under storm dam release flow discharge which is shown in Figure 5.6. A constraint for reservoir sediment release has been used by limiting it to 15 (kg/s). To mitigate the degradation problem, the developed model was applied to identify the optimal release rate $q(t)$ of reservoir sediment which is shown in Figure 5.7. The results showed the effectiveness of the developed model to consider reservoir sediment release limited capacity. Figure 5.8 shows the effectiveness of the developed model to eliminate the erosion downstream compared to the case of no-control (only clear water release from reservoir).

Figure 5.1 Diverted sediment and bed elevation change results from applying non-optimal control approach and developed model
Figure 5.2 Upstream sediment discharge and bed elevation change from applying non-optimal control approach and developed model

Figure 5.3 Stage reservoir water release
Figure 5.4 Upstream sediment discharge from applying the developed model

Figure 5.5 Bed elevation change from applying the developed model
Figure 5.6 Storm reservoir water release

Figure 5.7 Upstream sediment discharge from applying the developed model
Figure 5.8 Bed elevation change from applying the developed model
6.1 Background Information on the Study Reach

The Sandy River basin extends approximately 89 km from its headwater to its confluence with Columbia River (see Figure 6.1). The Bull River is the largest tributary to the Sandy River. The river basin is mountainous resulting from volcanic and glaciated events. Figure (6.2) shows the Sandy River longitudinal profile which incorporate approximately 11.3 m sudden drop at dam location. Approximately 750,000 cubic meter of sediment are stored behind the dam. Figure (6.3) shows the accumulated deposited sediment in the reservoir. The deposited sediment is mainly composed of a surface gravel layer (Unit 1) and underneath sand layer (Unit 2). Figure (6.4) provides the average composition of the reservoir sediment for each prescribed layers. The USGS operates stream flow gauges on the Sandy River. Daily discharge data from the Sandy River near Marmot gauge (station number 14137000) and the Sandy River below Bull Run gauge (station number 14142500) were used in sediment transport modeling. The dam has been removed on in summer 2007 and the coffer dam was breached on October 19, 2007.
Figure 6.1 Map of Sandy River basin (Source: Stillwater Sciences 2000)

Figure 6.2 Sandy River longitudinal profile (from Stillwater Sciences 2000)
6.2 Simulation of Marmot Dam Reach

One year simulation after dam removal from October 19, 2007 (coffer dam breaching date) to September 30, 2008 has been used to calibrate the model parameters. A discharge series spanning the period of model runs was required as input hydrographs to the model. Daily
discharge data used as input for the modeling are from the USGS Sandy River near Marmot
gauge (station number 1413700), which is located 0.5 km above Marmot Dam and has been in
operation since 1911, and the Sandy River below Bull Run gauge (station number 14142500),
which is located 0.2 km downstream of the Bull Run River confluence (RM 18.4) and has
several periods of records from 1910 to 1914; 1929 to 1966; and 1984 to present. The years
following the first year were selected randomly from all of the water years in the period of
Record Water years in Table (6.1). The Bull Run River is the largest tributary that enters the
Sandy River downstream of Marmot Dam and its flow discharge has been assigned as a lateral
discharge. Other tributaries create small incremental increases in drainage area, and therefore
likely create only small increases in water discharge and sediment load in the Sandy River and
their contribution have been neglected. The water stage downstream has been calculated using
Manning flow equation with the observed water discharge and normal depth assumption. To
attain an initial condition of simulation, a base flow of 5 m$^3$/s has been assumed. The sediment
transport rate was varying from about 250,000 metric tons per year at the Marmot Dam, of which
the majority is fine sediment.

Table 6.1 Water year series selected for the use in simulation

<table>
<thead>
<tr>
<th>Year in model</th>
<th>Water year</th>
<th>Year in model</th>
<th>Water year</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2008</td>
<td>6</td>
<td>1949</td>
</tr>
<tr>
<td>2</td>
<td>1932</td>
<td>7</td>
<td>1997</td>
</tr>
<tr>
<td>3</td>
<td>1951</td>
<td>8</td>
<td>1992</td>
</tr>
<tr>
<td>4</td>
<td>1991</td>
<td>9</td>
<td>1932</td>
</tr>
<tr>
<td>5</td>
<td>1988</td>
<td>10</td>
<td>1948</td>
</tr>
</tbody>
</table>
The field observations by PGE, Summer 2008 (after one year from dam removal) and reported in 0have been used to calibrate the model parameters. Different model parameters have been used in the calibration stage. Table (6.2) lists all the required model parameters and their values which have been calibrated. The roughness coefficient has been calibrated under three different scenarios: (1) The roughness coefficient is the same for upstream and downstream reaches, (2) The roughness coefficient has two separate values for upstream and downstream reaches, and (3) The roughness coefficient changes linearly in reservoir reach from a higher value upstream to a lower value at dam location while it is constant in downstream reach.

Table 6.2 Simulation parameters and associated values

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roughness Coefficient (Manning’s n)</td>
<td>0.03-0.06 upstream and 0.04-0.06 downstream</td>
</tr>
<tr>
<td>Sediment transport equation</td>
<td>Wu-Wang-Jia’s formula (Wu et al. 2000), SEDTRA module (Garbrecht et al. 1995), Modified Ackers-White formula (Proffit and Sutherland 1983), and Engelund and Hansen’s formula (Engelund and Hansen 1967)</td>
</tr>
<tr>
<td>Bed load adaptation length</td>
<td>250, 350, 500 and 1000m</td>
</tr>
<tr>
<td>Suspended load adaptation coefficient</td>
<td>0.25, 0.5 and 1.0</td>
</tr>
<tr>
<td>Mixing-layer thickness</td>
<td>0.05, 0.1 and 0.2m</td>
</tr>
<tr>
<td>Porosity</td>
<td>0.25</td>
</tr>
<tr>
<td>Simulation time step, $\Delta t$</td>
<td>0.5, 1, 3 and 6 minutes</td>
</tr>
<tr>
<td>Cross sections spacing,</td>
<td>Varying (12m-325m)</td>
</tr>
</tbody>
</table>

In order to model the in-field bathymetry conditions, surveyed cross sections data
provided by PGE (Stillwater Sciences 2000) have been used. The provided cross sections are
eleven cross sections from former dam location to approximately 1.5 miles (2.4 kilometers)
meters upstream, five cross sections immediately downstream of the dam to approximately 680
meters, and five cross sections downstream between RM 29.3 and RM 29.6 approximately 480
meters. To improve the simulation results, a number of cross sections have been linearly
interpolated or extrapolated in the chosen simulation reach.

6.2.1 Model Calibration

One year simulation after dam removal from October 19, 2007 (coffer dam breaching
date) to September 30, 2008 has been used to calibrate the model parameters. The simulation
results for different model parameters have been compared with the observed thalweg elevations
to determine the model parameters values which give the best agreement and can be used for the
long term simulation. The sensitivity of different model parameters has been investigated. Fig.
(6.5) shows the sensitivity of bed change with different scenarios of roughness coefficient. Many
different values of roughness coefficient for each scenario have been investigated; however, only
few cases are shown in Fig. (6.5) to demonstrate the significant effect of roughness coefficient
on the bed change results specially downstream (Fig. 6.5b,c) and to show the best scenario.

Four sediment transport capacity formulas have been implemented in CCH1D model:
SEDTRA module (Garbrecht et al., 1995), Wu et al.’s (2000) formula, the modified Ackers and
White’s (1973) formula (Proffit and Sutherland, 1983), and Engelund and Hansen’s formula. The
sensitivity of the CCH1D model has been tested using different formulas and the results are
shown in Fig. (6.6).
Non-equilibrium adaptation length characterizes the distance for sediment to adjust from Non-equilibrium state to an equilibrium state. Unfortunately its value has to be prescribed empirically and considerable uncertainty exists as different values have been adopted by different researchers. In Natural rivers where alternate bars are the dominate bed form, adaptation length may take the length of alternate bars, which is about 6.3 times the average channel width (Yalin, 1972). According to this approach, a value of 350 m for the average adaptation length has been considered in this study. However and to test the sensitivity of the deployed model, additional values of adaptation length have been tested and are summarized in Table (6.3) and the results are shown in Fig. (6.7). Table (6.3) which shows the values of model parameter which gives good results compared with the field observation. These values have been used in long term simulation under water flow series stated before. Fig. (6.8) shows the bed changes using the calibrated model parameters. In the first year following dam removal, sediment would form an erosion wave travelling upstream of the dam (Fig. 6.8a). Meanwhile, eroded sediment from upstream would form a deposition fan immediately downstream the dam (Fig. 6.8b), with very small amount for bed change predicted further downstream (Fig. 6.8c). The total flushed sediment during the first year is 620,620 cubic meters while the previously accumulated sediment in the reservoir is about 730,000 cubic meters and upstream sediment transport rate is 250,000 cubic meter per year. The simulation results are in good agreement with the observations which promote the applicability of the applied model for simulating dam removal cases.

6.2.2 Long-Term Simulation

To predict the flow and morphological changes due the Marmot Dam removal, a long-
term simulation has been carried out using the calibrated model parameters. A ten years simulation has been selected because it was believed that after ten years, the change in flow and bed elevation may be indistinguishable from the natural scour and fill processes. A discharge hydrograph spanning the simulation period was required as input to the model. The discharge hydrographs for first year after dam removal and previous recorded hydrographs downloaded from USGS (2009). The recorded first year discharge hydrograph was used as input for first year of simulation. The years following the first year were selected randomly from all the previous years records to represent the normal and extreme flow events. In this study, the selected input water hydrographs for years 2 through 10 are similar to that used by Stillwater Sciences (2000) which have been selected using a numerical random generator.

Figs. (6.9) and (6.10) show the model simulation results of water elevation and bed change evolutions respectively. The water elevation changing pattern is the same like the bed change. Generally the water elevation upstream the dam location is reducing due to the increased erosion with elapsed time (Fig. 6.9a). However, that behavior is reversed just upstream the dam because the sediment wedge at the dam location is flushed out and thus the bed slope becomes more gentle with time. In a similar way the water elevation downstream the dam location is increased due to the increased erosion with elapsed time (Fig. 6.9b). Further downstream, the water elevation in mainly changing in a natural behavior due the changing river bathymetry (Fig. 6.9c).

The bed change results plotted in Fig. (6.10) show that while there is increasing erosion upstream (Fig. 6.10a); there is increasing deposition downstream with time (Fig. 6.10b, c) as was expected. The flow discharge increases at dam location after dam removal and thus its sediment-
carrying capacity which result in increasing erosion upstream with elapsed time. Meanwhile, the
downstream gentle slope will reduce the flow carrying capacity of sediment and thus deposition
occurs. Just before the dam location, the deposition rate is slightly reduced with time which may
be due to the flushing out of the sediment wedge at the dam location with time as more gentle
slopes are developed which reduces the flow capacity to erode bed.

The model was stable and the simulation results are free from oscillations. The CPU
simulation time is mainly based on the simulation time step and the used computer configuration,
e.g. the CPU simulation time was 76,368s on a PC with AMD Athlon™ 64 FX-74 processor.

Table 6.3 Simulation parameters and associated values

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roughness Coefficient (Manning’s $n$)</td>
<td>0.04 upstream and 0.06 downstream</td>
</tr>
<tr>
<td>Sediment transport equation</td>
<td>Wu-Wang-Jia’s formula (Wu et al. 2000)</td>
</tr>
<tr>
<td>Bed load adaptation length</td>
<td>350 m</td>
</tr>
<tr>
<td>Suspended load adaptation coefficient</td>
<td>0.5</td>
</tr>
<tr>
<td>Mixing-layer thickness</td>
<td>0.05 m</td>
</tr>
<tr>
<td>Simulation time step</td>
<td>0.5 minute</td>
</tr>
</tbody>
</table>
Figure 6.5 Bed change under different scenarios for roughness coefficient (1 year simulation)
Figure 6.6 Bed change under different scenarios of Adaptation Length (1 year simulation)
Distance from Marmot Dam (km)

Average Bed Elevation Change (m)

Observations
SEDTRA module
Wu et al. s formula
Modified Ackers-White formula
Engelund and Hansen formula

(b) Observations
SEDTRA module
Wu et al. s formula
Modified Ackers-White formula
Engelund and Hansen formula
Figure 6.7 Bed change under using different sediment transport formulas (1 year simulation)
Figure 6.8 Bed evolutions using calibrated simulation parameters (1 year simulation)
Figure 6.9 Long-term water stage evolution using calibrated simulation parameters
<table>
<thead>
<tr>
<th>Distance From Marmot Dam (km)</th>
<th>Average Bed Elevation Change (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>0.2</td>
<td>Observations after 1 year</td>
</tr>
<tr>
<td>0.4</td>
<td>1 year</td>
</tr>
<tr>
<td>0.6</td>
<td>3 years</td>
</tr>
<tr>
<td>0.8</td>
<td>5 years</td>
</tr>
<tr>
<td>1</td>
<td>1 year</td>
</tr>
</tbody>
</table>

Figure 6.10 Long-term bed change evolution using calibrated simulation parameters
6.3 Application of Developed Model

The developed model has been applied to Marmot Dam reach under three different scenarios of operations:

- Scenario (1): Calculate the required sediment release of Marmot dam before removing to mitigate the excess erosion downstream. Simulating period is chosen between October 1, 2005 and September 30, 2007 before dam removal.

- Scenario (2): A hypothetical case to calculate the required diverted sediment after Marmot dam removal at the location of the dam to mitigate the excess deposition downstream. This scenario has been investigated in the same time period of scenario (1).

- Scenario (3): Calculate the required diverted sediment after Marmot dam removal at the location of the dam to mitigate the excess deposition downstream. Simulating period is one year immediately after dam removal.

Model parameters were assumed similar to calibrated ones in Marmot Dam Removal study. Sediment composition has been assumed the same throughout the simulation time. The results are based on using L-BFGS-B algorithm since it has the most convergent rate among the different algorithms investigated in this study.

6.3.1 Application before Dam Removal

Cross sections data are available in summer 2005 (Stillwater, 2007). The dam has been removed in summer 2007. Simulating period is chosen between October 1, 2005 and September 30, 2007 before dam removal. In this simulation, the storage capacity of reservoir was neglected and only clear water release was assumed. Upstream sediment transport rate is 250,000 cubic
meter per year. Figures (6.11 and 6.12) show the upstream and downstream hydrographs respectively during the simulation periods. These hydrographs specify both upstream and downstream flow boundary conditions.

Figure 6.11 Upstream hydrograph at 0.5km upstream dam (Oct. 1, 2005 - Sep. 30, 2007)
Fig. (6.13) shows the variation of calculated sediment release after the dam which mitigate the excess erosion downstream. The results are based on 70 iterations of optimization procedure, beyond which there is no improvement in the value of objective function. There is a similarity between sediment release and upstream hydrograph shown in Fig. (6.11). During peak periods, the sediment capacity of flow increases which requires releasing more sediment to overcome the erosion effect of clear water. The corresponding average bed change is depicted in Fig. (6.14). The model was not able to eliminate the erosion completely. This may be due to the complexity of river bathometry and assumption that Sediment composition has been assumed the same throughout the simulation time. However, the model was able mitigate the erosion depths significantly along the river reach. The total flushed sediment during the simulation period is
504,576 cubic meters while the accumulated sediment in the reservoir is about 730,000 cubic meters and upstream sediment transport rate is 250,000 cubic meter per year.

Figure 6.13 Required upstream sediment feed from applying optimal control model
Figure 6.14 Bed elevation change without control and with control scenarios.

6.3.2 Application after Dam Removal (hypothetical case):

Cross sections data are available in summer 2005. Simulation period is from summer 2005 to summer 2007 (similar to the previous case). Sediment and hydraulic boundary conditions are similar to the previous case. Model parameters were assumed similar to calibrated ones in Marmot Removal study.

Fig. (6.15) shows the bed change under without- and with- control simulations. The model was able to successfully reduce the shows average bed change. Fig. (6.15) shows a comparison between natural sediment flushed from reservoir during without-control simulation and optimal diverted sediment. The difference between both is somehow representing the carrying capacity of flow. The total flushed sediment during the simulation period is 203,074
cubic meters while the accumulated sediment in the reservoir is about 730,000 cubic meters and upstream sediment transport rate is 250,000 cubic meter per year.

Figure 6.15 Bed elevation change without control and with control scenarios.
6.3.3 Application after Dam Removal

The dam has been removed in summer 2007. One year simulation after dam removal from October 19, 2007 (coffer dam breaching date) to September 30, 2008 has been simulated. Fig. (6.17) shows the bed change under without- and with- control simulations. The model was able to successfully reduce the shows average bed change. Fig. (6.18) shows a comparison between natural sediment flushed from reservoir during without-control simulation and optimal diverted sediment. The difference between both is somehow representing the carrying capacity of flow. The total flushed sediment during the simulation period is 504,576 cubic meters while the accumulated sediment in the reservoir is about 730,000 cubic meters and upstream sediment transport rate is 250,000 cubic meter per year. Although some oscillations in the optimization procedure and it took some time to converge to the optimal solution (Fig. 19), the overall
performance is good and the model convergences.

Figure 6.17 Bed elevation change without control and with control scenarios.
Figure 6.18 Comparison between natural sediment flushed and optimal diverted sediment

Figure 6.19 Iterations of the objective function
CHAPTER VII

MODEL APPLICATION TO CHANNEL NETWORK

The application of the developed can be extended to be applicable to control sediments in watersheds. At confluences, the internal boundary condition of the derived adjoint equation (Eq. 20) was derived, then the developed model has been applied a channel network. The preliminary results proved that the model can be effectively applied to mitigate morphological changes in watersheds.

7.1 Internal Conditions for Confluence in Channel Network

In order to solve the adjoint equations or optimal control in a channel network, it is indispensable to impose the internal boundary condition at every confluence in a channel network. This boundary condition at a confluence which comprises three channels, i.e., Channel 1 and Channel 2 at the upstream, and Channel 3 at the downstream, can be derived as follows:
The internal boundary conditions of sediment discharges are specified as follows:

\[ Q_i^3 = Q_i^1 + Q_i^2 \]  \hspace{1cm} (29)

Taking the variations of the internal boundary condition

\[ \delta Q_i|_{x_3} = \delta Q_i|_{x_1} + \delta Q_i|_{x_2} \]  \hspace{1cm} (30)

\[
\int_0^T \dot{\lambda}_q \delta Q_i|_{x_3} \, dt = \int_0^T \dot{\lambda}_q \delta Q_i|_{x_1} \, dt + \int_0^T \dot{\lambda}_q \delta Q_i|_{x_2} \, dt \]  \hspace{1cm} (31)

\[
\int_0^T \left( \dot{\lambda}_q \delta Q_i|_{x_1} + \dot{\lambda}_q \delta Q_i|_{x_2} - \dot{\lambda}_q \delta Q_i|_{x_3} \right) \, dt = 0 \]  \hspace{1cm} (32)

\[
\int_0^T \left( \dot{\lambda}_q \delta Q_i|_{x_1} + \dot{\lambda}_q \delta Q_i|_{x_2} - \dot{\lambda}_q |_{x_3} \delta Q_i|_{x_1} + \dot{\lambda}_q |_{x_3} \delta Q_i|_{x_2} \right) \, dt = 0 \]  \hspace{1cm} (33)

\[
\int_0^T \left( \dot{\lambda}_q |_{x_1} - \dot{\lambda}_q |_{x_3} \right) + \left( \dot{\lambda}_q |_{x_2} - \dot{\lambda}_q |_{x_3} \right) \, dt = 0 \]  \hspace{1cm} (34)

which result in the Lagrangian multiplier boundary condition at a confluence

\[ \dot{\lambda}_q |_{x_3} = \dot{\lambda}_q |_{x_1} \]  \hspace{1cm} (35)

\[ \dot{\lambda}_q |_{x_2} = \dot{\lambda}_q |_{x_3} \]  \hspace{1cm} (36)
By imposing all internal boundary conditions at every confluence in the channel network, the adjoint equation and its boundary conditions on the channel network are well defined and, therefore, are ready for numerical solution.

7.2 Optimal Control of Flood Diversions in Channel Network

The dendritic channel network with a main reach and two branches as shown in Figure (7.2) was taken into account in this case, in which the compound cross section shown in Figure (7.3) was assumed in all three channels. Three triangular hydrographs were imposed on the three inlets of the channels, the parameters of which are listed in Table (7.1) in which $Q_p=$ peak discharge; $Q_b=$ base flow discharge; $T_p=$ time to peak and $T_d=$ flood duration. The hydrograph at the inlet of Channel 2 was assumed to be the same as that of Channel 1. The hydrograph at the inlet of Channel 3 main reach had a higher peak discharge than those of the other two. In the simulation of flood propagation, the channels were divided into a total of 43 short reaches with equal spatial increments $x=500$ m. To test the capability of the sediment control model for the channel network under complex geometry, one upstream gate was assumed to be located at main stream inlet.
Figure 7.2 Channel network configuration

Figure 7.3 Compound channel cross section
Table 7.1 Hydrograph Parameters

<table>
<thead>
<tr>
<th>Channel No.</th>
<th>$Q_p$ (m$^3$/s)</th>
<th>$Q_b$ (m$^3$/s)</th>
<th>$T_p$ (hour)</th>
<th>$T_d$ (hour)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>50.0</td>
<td>2.0</td>
<td>16.0</td>
<td>48.0</td>
</tr>
<tr>
<td>2</td>
<td>50.0</td>
<td>2.0</td>
<td>16.0</td>
<td>48.0</td>
</tr>
<tr>
<td>3</td>
<td>60.0</td>
<td>6.0</td>
<td>16.0</td>
<td>48.0</td>
</tr>
</tbody>
</table>

Figure (7.4) shows that the main channel undergoes excess erosion during flood duration due to the increased flow capacity to erode bed and transport sediment downstream. To balance the erosion effect, sediment is fed upstream. After about 70 iterations of deploying the developed using L-BFGS-B algorithm, the required upstream sediment discharge was found and is shown in Figure (7.5). The corresponding bed change in the main channel has been reduced significantly compared to the case without control as shown in Figure (7.4). The model was stable and convergent as shown in Figure (7.6).

Figure 7.4 Bed elevation change from applying the developed model
Figure 7.5 Upstream sediment discharge from applying the developed model

Figure 7.6 Iterations of the objective function and the norm of its gradient
CHAPTER VIII

SUMMARY, CONCLUSIONS AND FUTURE RESEARCH

8.1 Summary

An optimal procedure to minimize bed changes in open-channels was developed. It is based on coupling adjoint sensitivity analysis with a one dimensional sediment transport model CCHE1D. The optimization module includes a numerical solver for the adjoint equation and an optimization procedure. The model has the flexibility to control the rate of bed deformation cross-sectional area under different control variables i.e. side inflow/outflow, upstream or downstream sediment discharge conditions. The model has been applied to sedimentation problems and the results demonstrated that the model is able to mitigate the morphological changes effectively. The developed approach for real world cases such as optimal sediment diversion after dam removal has been elaborated.

The current research proposes an innovative optimization approach procedure which can minimize morphological changes in alluvial networks due to extreme events (e.g. floods and dam removals) under operational constraints so that the optimal sediment control can be achieved. The developed approach will be applied first to control morphological changes by only diverting sediment from diversion structure. Further, the developed approach will be elaborated to a more practical approach by diverting both water and sediment during flood in watersheds.
8.2 Conclusions

The following points can be concluded from this research:

- Improved knowledge for optimal sediment control by non-linear optimization.
- Developing new approaches for sediment control in alluvial rivers at sediment control structures under different scenarios of normal operation or during extreme events.
- Developing integrated software coupling sediment transport model with developed optimization algorithms.
- Demonstrate the effective applicability of the developed software to solve or mitigate different sediment problems e.g. reservoir sediment control for dam removal practices.

8.3 Future Work

While the present study has proposed an effective tool which can be used to control sediment and mitigate morphological changes, there are still some issues may be investigated in future. Based on the experience gained during conducting this research, the following areas are recommended to future study:

- Elaborate the developed model to be applied at multiple control locations in rivers to minimize the morphological changes along the target reach.
- In this study, the location of the control location has been pre-determined before applying the developed model. It future work, the model can be improved by incorporating the capability of searching for the optimum location.
• The developed model can be further tested to control sediment in real cases of watersheds.

• The developed can be coupled with an optimal flow control model to best control both flow and sediment during extreme events.
BIBLIOGRAPHY


Conference (MAESC), May 3, Christian Brothers University, Memphis, TN, USA.


pp.241-252.


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