Development of hydrograph-based approach to modeling fate and transport of sediment-borne bacteria in lowland rivers

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DEVELOPMENT OF HYDROGRAPH-BASED APPROACH TO MODELING FATE AND TRANSPORT OF SEDIMENT-BORNE BACTERIA IN LOWLAND RIVERS

A Dissertation

Submitted to the Graduate Faculty of the
Louisiana State University and
Agricultural and Mechanical College
in partial fulfillment of the
requirements for the degree of
Doctor of Philosophy

in

The Department of Civil and Environmental Engineering

by

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B.S., Odessa State Academy of Civil Engineering and Architecture, 1992
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May, 2012
To my late mother who left for heaven before it was complete
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ABSTRACT

Fecal pollution is one of the major factors responsible for water quality impairments of rivers and streams, particularly organic-rich fine-grained lowland streams. While predicting fecal pollution is generally required in the development of water quality restoration plans like Total Maximum Daily Loads, no single model has been widely recognized as an efficient and effective tool for estimating fecal pollution.

This dissertation develops a simple yet effective modeling approach, called Hydrograph-based Approach, to bacterial fate and transport modeling in lowland rivers. The new hydrograph-based approach is simple and efficient in terms of its less data requirements as compared with other models. The new approach utilizes widely available hydrographs as the primary model input data. The new hydrograph-based approach is effective in terms of its capability in predicting bacterial concentrations for a wide range of flow conditions from low flow without sediment to flood events carrying high concentrations of sediment. The development of this new approach is based on the following major works: 1) a hydrograph-based method for determining bed shear velocity and other flow parameters was developed and tested using measured experimental data as well as simulated results from HEC-RAS for two river flood events; 2) a relatively simple hydrograph-based method for estimating sediment transport during unsteady flows was developed and tested using sediment concentration data collected during several flood events in two US rivers; 3) the solute transport process in rivers, in particular, the effect of channel size on residence time distribution, was investigated using a variable residence time model; and 4) a hydrograph-based approach for modeling bacterial fate and transport was developed, utilizing the variable residence time model for mass transport and hydrograph-based methods for flow and sediment transport, and tested through case studies using data observed in
three rivers with distinct flow and sediment transport characteristics. This hydrograph-based approach includes most of the important bacterial transport and fate processes such as advection, dispersion, transient storage exchange, resuspension/deposition, and bacterial growth/decay. The modeling results using this approach appear to be better or at least comparable with the results from other more complicated models.
CHAPTER 1. INTRODUCTION

1.1. Significance

Fecal pollution is one of the major factors responsible for water quality impairments of rivers and streams, particularly organic-rich fine-grained lowland streams. In fact, approximately 35 percent of impairments of rivers and streams are caused by fecal indicator bacteria (U. S. Environmental Protection Agency 2000). Since predicting fecal pollution is generally required in the development of water quality restoration plans like Total Maximum Daily Loads, extensive efforts have been made to develop efficient and effective modeling tools for estimating fecal pollution.

The contribution of flood events to the annual load of *Escherichia coli* (*E. coli*, fecal indicator bacteria) from the catchment has been reported to be as high as 98 percent in some rivers (Chu et al. 2011; McKergow and Davies-Colley 2010). Wilkinson et al. (1995) observed 25 times increase in fecal coliform concentrations during an artificial flood event where the contribution from the watershed was negligible. Previous studies indicated that a significant portion of the bacterial load during peak flows may actually come from the bed sediment (Cho et al. 2010; Wilkinson et al. 2011), which may work as a reservoir of *E. coli* during low flows (Smith et al. 2008). High flow events transport the major part of microbial contaminants in rivers, and yet no simple methods for estimating contaminant transport during such events are available.

Peak bacterial concentrations in streams are found to occur usually during the rising limb of the storm hydrograph (Davies-Colley et al. 1994; Jamieson et al. 2005) well ahead of the discharge peak and close to the line of maximum flow acceleration (McKergow and Davies-Colley 2010; Nagels et al. 2002). An early peak in suspended sediment concentration during
floods is also a very common phenomenon. It means that sediment and especially fine-grained organic-rich sediment are the primary vector of bacteria in streams. A reason for the early peak in sediment concentration is that the maximum bed shear stress occurs well before the discharge reaches its peak (De Sutter et al. 1999; Nezu and Nakagawa 1995; Tu and Graf 1993). An accurate prediction of bed shear stress, therefore, is critical to estimating the transport of sediment and associated bacteria during flood events. However, no simple methods for finding the bed shear stress during unsteady flows are available. Many existing methods for determining shear velocity such as logarithmic profile, drag, Reynolds shear stress and turbulent kinetic energy methods are either too difficult or unsuitable for unsteady flow conditions in natural channels (Biron et al. 2004). Detailed data necessary for hydrodynamic computations using full St. Venant equations are often not available. In most cases, the available data are only flow hydrographs interpreted from recorded stage data and base flow conditions. Consequently, steady uniform flow formula is still applied, which may often lead to large and unacceptable errors.

Previous bacterial modeling efforts either overly simplified bacterial transport by assuming fecal indicator bacteria to be like a solute, or did not fully include sediment transport processes (Bai and Lung 2005). Fecal indicator bacteria are commonly transported in two phases: dissolved phase (or free-living phase) and sediment-associated phase while the fraction of attached bacteria varies greatly from less than 0.1 (Liu et al. 2006) to over 0.8 (Hipsey et al. 2006) in natural rivers. In fact, when the sediment concentration is low and free-living bacteria are dominant, the bacterial transport is much like solute transport. This is why Shen et al. (2008) were able to successfully simulate bacteriophage tracer test in Grand River, Ohio, using the transient storage
model. On the other hand, when the sediment transport processes are dominant, ignoring contribution of sediment-borne bacteria may lead to unacceptable results.

Despite recent efforts in incorporating sediment-bacteria or sediment-water column interactions into modeling, there is a significant gap in knowledge, pertaining to actual river flood events. Most of the models have been tested using hypothetical scenarios and artificial flooding studies (Bai and Lung 2005; Gao et al. 2011). Insufficiency in measured data, inadequate knowledge of sediment-water interface dynamics, particularly in cohesive sediment environments, and lack of simple methods for dealing with unsteady flows are hindering our capability to predict the fate and transport of microbes in natural rivers.

1.2. Goals and Objectives

Overall goals of this work are to increase understanding of basic processes controlling contaminant transport in lowland rivers and to explore approaches to solving practical problems without resorting to complex numerical methods requiring extensive input data that are often difficult to obtain. The study is focused on two aspects: (1) modeling of early peak phenomenon in flow shear stress, sediment, and associated bacterial transport during unsteady high flows; and (2) extension of variable residence time based model to include bacterial transport in rivers with high sediment-bacteria interaction.

Objectives of the research are to: (1) develop an efficient method for bed shear velocity for unsteady flows using flow hydrograph and channel geometry as primary inputs; (2) develop a sediment transport model based on hydrograph for suspended sediment dominated lowland rivers; (3) characterize the solute transport processes in rivers using a variable residence time model; and (4) extend the variable residence time model by incorporating sediment
resuspension/deposition-induced bacterial transport, watershed inputs, and bacterial
growth/decay processes.

1.3. Organization of Dissertation

The dissertation is organized into six chapters following the journal style format
recommended by LSU Graduate School. The major part of this dissertation is made up of four
chapters (2–5) that are based on four peer-reviewed journal papers either already published,
submitted or in preparation. Since each major chapter is prepared as a stand-alone journal paper,
some information may be repeated in some of the chapters for clarity and completeness.

Chapter 2 develops a method for estimating the shear velocity of unsteady flow using flow
hydrograph and channel geometry data. Using equations for open channel flow, formulas for
shear velocity are derived in terms of discharge gradient, representing the friction slope of
unsteady flow in prismatic channels. The method is tested using published experimental datasets
and compared with existing methods.

Chapter 3 develops a sediment transport estimation method for unsteady flows based on
flow hydrograph-based bed shear stress and friction slope. The method is explored using a
hypothetical hydrograph and then applied to two lowland rivers. Finally, the results are
compared with those from the widely used HEC-RAS program.

Chapter 4 investigates the solute transport process in large and medium rivers using tracer
test data and a variable residence time model. In particular, the effect of channel size on
longitudinal transport of solutes in rivers is explored in terms of various shapes of residence time
distributions. The influence of shear dispersion, lateral inflows/outflows and hyporheic exchange
on solute concentration breakthrough curves is investigated.
Chapter 5 presents an approach for modeling bacterial transport and fate in natural streams. The variable residence time based model is extended to include bacterial transport and fate by considering unsteadiness of flow using hydrograph-based approach and resuspension/deposition processes of sediment and associated bacteria. The model is applied to simulate bacterial transport in natural streams with significantly different flow conditions ranging from low flow without sediment to flood events carrying high concentrations of sediment and bacteria.

Chapter 6 summarizes and discusses major findings of the dissertation.

Organization of the dissertation and interrelationship among chapters are shown in the following diagram.

![Organization of dissertation diagram]

**Figure 1-1 Organization of dissertation**
1.4. References


CHAPTER 2. HYDROGRAPH-BASED METHOD FOR BED SHEAR VELOCITY

2.1. Introduction

Assessment of shear velocity or bed shear stress during a flood event is important to estimating sediment transport in natural rivers. However, determining shear velocity change during unsteady flows is a relatively difficult task due to its non-linear relationship with flow parameters such as flow depth or discharge. Even in a prismatic channel of simple rectangular cross-section, the bed shear stress during the large floods is not symmetrical. The bed shear stress is higher during rising stages than corresponding falling stages, and it reaches its maximum before the peak flow (De Sutter et al. 1999; Graf and Qu 2004; Meirovich et al. 1998; Nezu et al. 1997; Song and Graf 1997; Tu and Graf 1993). The maximum shear stress values are sometimes as high as 3-4 times base flow values (Nezu and Nakagawa 1995). The steady uniform flow formula, with a constant friction slope equal to the bed slope, produces unacceptable results for highly unsteady flood flows (Bares et al. 2006; Rowinski et al. 2005; Tu and Graf 1993). It gives symmetrical bed shear values during both rising and falling stages and severely underestimates during rising periods.

No simple methods for estimating shear velocity during unsteady flows are currently available. Many existing methods for determining shear velocity (Rowinski et al. 2005) such as logarithmic profile, drag, Reynolds shear stress and turbulent kinetic energy methods are either too difficult or unsuitable in unsteady flow conditions in natural channels (Biron et al. 2004). During large flood events, when major sediment movement occurs, it is difficult to collect detailed data necessary for accurate hydrodynamic computations using full St. Venant equations.

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These equations require measurements of water surface slope at the same time increments as the mean velocities, which is a relatively difficult task to implement (Rowinski et al. 2000). In most cases, the available information is only flow hydrographs interpreted from recorded stage data and base flow conditions. Consequently, steady uniform flow formula is still applied, which may often lead to large and unacceptable errors.

The objective herein is to present a simple method for shear velocity estimation during flood events using easily available flow and channel geometry data. A formula based on the non-inertial wave approximation and kinematic wave profile is derived for practical applications in natural flood events of mild-sloped rivers. The method is tested using measured data from flume experiments and simulated results from the Hydrologic Engineering Centers River Analysis System (HEC-RAS) software. As the concept of friction slope has been extensively used in studies of stage-discharge relationship, the newly developed formula is also examined and compared with the established formula from the literature (Ghimire and Deng 2011).

### 2.2. Friction Slope during Unsteady Flows

#### 2.2.1. Friction Slope in Gradually Varied Flows

For unsteady open channel flows, the friction slope $S_f$ is essential to estimating the bulk shear velocity

$$U_* = \sqrt{g R_h S_f}$$

(2.1)

where $g$ = acceleration due to gravity, $R_h$ = hydraulic radius, and $S_f$ = friction slope. It accounts for shear resistance from the wetted perimeter and is time-dependent under unsteady flow conditions. For flows in wide open channels where the effect of side wall friction is negligible, $R_h$ is often replaced with flow depth, $h$. Under steady uniform flow conditions, $S_f$ is substituted
by channel bed slope $S_o$. The steady flow formula thus obtained is commonly applied in many river studies and is still widely used by researchers. However, under unsteady flows, $S_f$ is not constant. Solving one-dimensional St. Venant equations is the common approach to finding friction slopes (Rowinski et al. 2000).

For one-dimensional gradually-varied unsteady flow without lateral flows in a prismatic channel having constant bed width, the St. Venant equations for continuity and momentum are given by

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = \frac{\partial h}{\partial t} + U \frac{\partial h}{\partial x} + h \frac{\partial U}{\partial x} = 0 \tag{2.2}$$

$$S_f = S_o - \frac{\partial h}{\partial x} - \frac{U \partial U}{g} - \frac{1}{g} \frac{\partial U}{\partial x} \tag{2.3}$$

These equations, also called dynamic wave equations, define the propagation of flood waves in open channels. Four terms on the right side of the Eq. (2.3) represent different components of the friction slope: the first term is channel bed slope reflecting gravity force, the second term is pressure differential reflecting change in depth in longitudinal direction, the third term is convective acceleration reflecting change in velocity in longitudinal direction, and the fourth term is local acceleration reflecting unsteadiness of flow. Third and fourth terms are often called the inertial terms. Based on the significance of the slope terms, Eq.(2.3) can be simplified to represent a particular wave type such as kinematic wave, or noninertia wave (Yen and Tsai 2001). St. Venant equations and various methods for representing friction slopes are well explained in the literature (Moussa and Bocquillon 1996; Ponce 1989; Singh 1996).

Graf and Song (1995) combined the equation of continuity and motion to obtain

$$S_f = S_o + \frac{\partial h}{\partial x} (Fr^2 - 1) + \frac{1}{gh} \left( U \frac{\partial h}{\partial t} - h \frac{\partial U}{\partial t} \right) \tag{2.4}$$
where \( U \) = cross-sectionally averaged velocity, \( t \) = time, \( x \) = longitudinal distance along the channel, and \( Fr = \frac{U}{\sqrt{gh}} \) = Froude number. Tu and Graf (1993) used the wave velocity concept to transform the momentum equation into an expression without spatial variation of depth

\[
S_f = S_o + \frac{1}{C} \frac{\partial h}{\partial t} - \frac{1}{g} \frac{\partial U}{\partial t} \left( 1 - \frac{U}{C} \right) \tag{2.5}
\]

where \( C \) is celerity of the flood wave. Although much simpler than Eq. (2.4), this equation still contains an acceleration term and is expressed in terms of depth gradient instead of discharge gradient. Exclusion of non-significant slope terms for such flows is possible without introducing significant errors. It will be done in section 2.3.

### 2.2.2. Friction Slopes in Rating Curve Formulas

The friction slope term has been used in many formulas for unsteady flow discharge estimation. Starting from Jones (1916), various authors derived or modified the existing discharge estimation formulas using stage records at a single station (Fenton 1999; Fread 1975; Henderson 1963; Perumal and Rang Raju 1999; Perumal et al. 2004) and using simultaneous stage measurements at more than one station (Chow 1959; Dottori et al. 2009). These formulas were intended to account for the hysteresis in stage-discharge relationship, which is commonly observed in mild sloped rivers and is attributed to the secondary terms in the momentum equation (Ponce 1989). In these formulas, the discharge during unsteady flow is obtained from the reference discharge computed using a single stage-discharge relationship \( Q = Q_{ref} \sqrt{S_f/S_{ref}} \), where subscript \( ref \) stands for reference condition (Schmidt and Yen 2008). By assuming that the reference condition is the steady uniform flow condition, when \( S_{ref} = S_o = S_f = S_w \), the Jones formula is obtained as
\[
Q = Q_o \left( \frac{1}{S_o} \left( S_o + \frac{1}{C} \frac{\partial h}{\partial t} \right) \right) = Q_o \left( 1 + \frac{1}{S_o C} \frac{\partial h}{\partial t} \right)
\]

(2.6)

where \(Q_o\) is the discharge during base (reference) flow. In this equation \(S_f = S_o + \frac{\partial h}{(C \partial t)}\), is a result of the kinematic wave assumption. This basic assumption of non-attenuating waves in deriving unsteady flow formula has led many researchers (e.g., Henderson 1963) to doubt the logical correctness of Jones formula. Nevertheless, Jones and many other variants were used in many cases with varying degree of success, mostly depending upon the reference conditions. Schmidt and Yen (2008) note that the Jones formula produces good result if \(S_{ref} = S_o\), which means that the reference base flow condition is a steady uniform flow. The friction slopes used in rating curve formulas for unsteady flow are summarized in Table 2-1.

Table 2-1 Friction slope in Jones and other formulas for stage-discharge relationship

<table>
<thead>
<tr>
<th>Authors</th>
<th>Expressions for friction slopes</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Jones (1916)</td>
<td>(S_o + \frac{1}{C} \frac{\partial h}{\partial t})</td>
<td>Jones formula</td>
</tr>
<tr>
<td>2. Henderson (1963)</td>
<td>(S_o + \frac{1}{C} \frac{\partial h}{\partial t} + \frac{2S_o}{3r^2})</td>
<td>Accounted for subsidence</td>
</tr>
<tr>
<td>3. Fenton (1999)</td>
<td>(S_o + \frac{1}{C} \frac{\partial h}{\partial t} - \frac{Q_o}{2BS_oC^3} \frac{\partial^2 h}{\partial t^2})</td>
<td>Included a diffusive term</td>
</tr>
<tr>
<td>4. Perumal and Rang Raju (1998)</td>
<td>(S_o + \frac{1}{C} \frac{\partial h}{\partial t} \left[ 1 - m^2F r^2 p^2 \left( \frac{\partial R_k}{\partial h} \right)^2 \right] )</td>
<td>Included inertial forces</td>
</tr>
<tr>
<td>5. Perumal et al. (2004)</td>
<td>(\frac{S_o}{2} + \frac{1}{2C} \frac{\partial h}{\partial t} + \frac{S_o}{2} \left( \frac{1}{\frac{S_o}{C} \frac{\partial h}{\partial t}} \right) - \frac{2Q}{BS_o^2C^3} \frac{\partial^2 h}{\partial t^2})</td>
<td>Refined estimate of (\partial h/\partial x)</td>
</tr>
</tbody>
</table>

Note: \(r = \frac{S_o}{(\partial h/\partial x)}\) is the ratio of channel bottom slope to entering wave slope. It can be approximated by the ratio of flood wave height to its half-length.
2.3. Event Flow Hydrograph-Based Formula for Shear Velocity

In case of most natural flood waves it is often not necessary to have all slope terms in equation of motion. For such flows the inertial terms can be neglected as they are small in comparison with the bed slope (Henderson 1963). In fact, this non-inertial wave approximation is widely used for flood routing and is more generic than kinematic by including pressure force term in the momentum equation (Moussa and Bocquillon 1996). Weinmann and Laurenson (1979) note that in flat channels, the pressure term may even approach bed slope. This approximation is considered as the simplest among approximations that consider backwater effects and yield good results (Yen and Tsai 2001). The omission of convective and local acceleration terms does not result in much error, not only because they are much smaller than the pressure term, but also because they have opposite signs. With this consideration, the Eq. (2.3) may be simplified as

\[ S_f = S_o - \frac{\partial h}{\partial x} \]  

(2.7)

where the right-hand side is simply the local water surface slope and the second term on the right side is responsible for diffusion or subsidence of flood waves. Since measuring the water surface slope during flood events is a difficult task (Rowinski et al. 2000), it is preferable to find an alternative expression for \( \partial h/\partial x \). Following Henderson (1963) and assuming that the bulk of flood wave moves approximately as a kinematic non-subsiding monoclinal wave, discharge \( Q(x, t) \) can be treated as a constant relative to an observer moving with the wave velocity. Then, its total derivative is expressed as

\[ dQ = \frac{\partial Q}{\partial t} dt + \frac{\partial Q}{\partial x} dx = 0 \]  

(2.8)
Assuming that the channel width $B$ remains constant and there is no lateral in- or outflow, the mass conservation principle requires

$$\frac{\partial Q}{\partial x} = -B \frac{\partial h}{\partial t} \tag{2.9}$$

The wave celerity may then be expressed as

$$C = \frac{dx}{dt} = -\frac{\partial Q/\partial t}{\partial Q/\partial x} \tag{2.10}$$

Using Eq. (2.10) and Eq. (2.9), $\partial h/\partial t$ can be expressed in term of $\partial Q/\partial t$ as

$$\frac{\partial h}{\partial t} = \frac{1}{BC} \frac{\partial Q}{\partial t} \tag{2.11}$$

Following the same logic as used in Eq. (2.8) for water depth $h(x,t)$ we get

$$\frac{dh}{dt} = \frac{\partial h}{\partial t} + \frac{dx \partial h}{dt \partial x} = 0 \tag{2.12}$$

$$\frac{\partial h}{\partial x} = -\frac{1}{C} \frac{\partial h}{\partial t} \tag{2.13}$$

By substituting the Eq. (2.11) into Eq. (2.13), an alternative expression for $\partial h/\partial x$ is obtained as

$$\frac{\partial h}{\partial x} = -\frac{1}{BC^2} \frac{\partial Q}{\partial t} \tag{2.14}$$

where $\partial Q/\partial t$ is discharge gradient, which can be obtained from the flow hydrographs.

Substituting $\partial h/\partial x$ from Eq. (2.14) into Eq. (2.7) and then combining with Eq. (2.1), the shear velocity is obtained as

$$U_* = \sqrt{gR_h \left(S_o + \frac{1}{BC^2} \frac{\partial Q}{\partial t} \right)} \tag{2.15}$$

As the flood wave is nearly kinematic for most natural floods, the celerity may be obtained using the area-discharge relationship at a given station from Seddon’s law as $C = \partial Q/\partial A$, where $A$ is cross-sectional area. Celerity $C$ can be obtained for various channel shapes by relating it
with the average velocity, i.e. \( C = \beta U \), where \( \beta \) is a kinematic wave parameter. For a wide rectangular channel \((B \gg h)\), the Manning law gives the highest value of \( \beta = 1.67 \). If the celerity at a cross-section is approximately constant, Eq. (2.15) further simplifies to

\[
U_* = \sqrt{gh \left( S_o + \alpha \frac{\partial Q}{\partial t} \right)}
\]  

(2.16)

where \( \alpha \) is a parameter relating the slope of the flood hydrograph to the shear velocity of unsteady flows. It may be determined either as a modeling parameter using observed shear velocities, or by following the method explained in the next section.

### 2.4. Verification of the Hydrograph-Based Method

The proposed Eqs. (2.15) and (2.16) were tested using published experimental datasets and HEC-RAS simulations. Data from flume experiment S-15-936 (Graf and Song 1995), S30-931 (Graf and Song 1995) and NS1(1) (Tu and Graf 1993) were used in this study. All experiments were carried out in a rectangular flume (16.8 m long, 0.6 m wide and 0.8 m high) with gravel bed and glass walls designed to study the flows with different unsteadiness using modern measuring instruments (Graf and Qu 2004). The shear velocity in all of these experiments was measured using Clauser’s method (Clauser 1956). The flow parameters are given in Table 2-2.

Table 2-2 Summary of flow parameters and data sources

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>( S_o ) (%)</td>
<td>–0.15</td>
<td>0.3</td>
<td>0.2</td>
</tr>
<tr>
<td>( T ) (s)</td>
<td>50</td>
<td>102</td>
<td>110</td>
</tr>
<tr>
<td>( h ) (cm)</td>
<td>14.0 to 18.3</td>
<td>11 to 13.7</td>
<td>9 to 21.2</td>
</tr>
<tr>
<td>( U ) (cm/s)</td>
<td>60.2 to 84.9</td>
<td>88.3 to 108.2</td>
<td>40.8 to 94.9</td>
</tr>
<tr>
<td>( Q ) (l/s)</td>
<td>50.6 to 92.3</td>
<td>58.5 to 89.1</td>
<td>22.8 to 119.3</td>
</tr>
<tr>
<td>( \Gamma ) (-)</td>
<td>0.018</td>
<td>0.004</td>
<td>0.013</td>
</tr>
<tr>
<td>( F ) (-)</td>
<td>0.50 to 0.60</td>
<td>0.80 to 0.90</td>
<td>0.43 to 0.66</td>
</tr>
<tr>
<td>( U_* ) (cm/s)</td>
<td>4.17 to 6.02</td>
<td>6.52 to 8.06</td>
<td>3.29 to 8.50</td>
</tr>
</tbody>
</table>

Note: \( T = \) duration of hydrograph, \( \Gamma = \) unsteadiness (Graf and Song 1995), \( F = U/(gh)^{1/2} \)
Observations of experimental data indicate that Eq. (2.16) gives the best result if $C*$ is the celerity of the velocity wave, i.e. the wave with maximum average flow velocity. The best fitted

![Graph](image_url)

Figure 2-1 Comparison of shear velocities using Exp S-15-936 data (Graf and Song 1995). $\alpha$ values were practically identical to $1/BC^2$. However, finding $C*$ using Seddon’s method was difficult in flume experiments due to a large scatter of $C$ near the peak. As the flood waves were not kinematic, the friction law gave less accurate, albeit smooth, results. Therefore $\beta$ was replaced with the 'equivalent kinematic wave parameter' $\beta_e$, producing $C = \beta_e U$ close to that given by Seddon’s law. The celerity of the velocity wave was then determined as $\beta_e U_{\text{max}}$. When the flood wave is nearly kinematic, the celerity is equal to $\beta U$, where $\beta$ is kinematic flood wave parameter determined using friction laws for given cross-section. For example, for flood waves
in rectangular channels, the Manning's law gives $\beta = 5U/3$ and $C^*$ is simply $5/3U_{max}$, where $U_{max}$ is the speed of the velocity wave. In case of natural floods in prismatic channels $\beta_e \approx \beta$. So $\alpha$ was directly obtained as $\beta U_{max}$.

Figure 2-2 Comparison of shear velocities using Exp S30-931 data (Graf and Song 1995)

The calculated and measured shear velocities of three experiments are shown in Figures 2-1–2-3. The values of $\beta_e$ were 3.07, 1.2, and 1.8 with corresponding $\alpha$ values of $2.47 \times 10^{-07}$, $9.8 \times 10^{-07}$ and $5.7 \times 10^{-07}$ s²/cm³, respectively. To account for sidewall effects, the Vanoni-Brooks method (Julien 1998) was applied.

The accuracy of all six formulas was also evaluated and compared with measured shear velocities using a numerical measure called Root-Mean-Squared Error (RMSE). RMSE is a
measure of difference between values predicted by an estimator and the observed data, and is expressed in the unit of shear velocity.

\[ \text{RMSE} = \sqrt{\frac{1}{k} \sum_{i=1}^{k} (U_{\text{cal},i} - U_{\text{obs},i})^2} \]  

(2.17)

where, \( U_{\text{cal}} \) = calculated shear velocity, \( U_{\text{obs}} \) = measured shear velocity using Clauser’s method, \( k \) = number of observations during a hydrograph event and \( i \) = observation number. RMSE and indicate how close are the observed shear velocities to the predicted values using the given equations. Lower RMSE values mean better prediction by the given formula and vice versa. The results are shown in Table 2-3.
Figure 2-4 Comparison of shear velocities using Eqs. (2.15) and (2.16) with HEC-RAS results for March 2002 flood at McConnelsville of Muskingum River.

Table 2-3 Root-Mean Squared Errors (RMSE)

<table>
<thead>
<tr>
<th>Experiment</th>
<th>Eq. (2.15)</th>
<th>Eq. (2.16)</th>
<th>(Jones 1916)</th>
<th>(Henderson 1966)</th>
<th>Constant $S_f$</th>
</tr>
</thead>
<tbody>
<tr>
<td>S-15-936</td>
<td>0.29</td>
<td>0.25</td>
<td>0.51</td>
<td>0.52</td>
<td>0.64</td>
</tr>
<tr>
<td>S30-931</td>
<td>0.35</td>
<td>0.30</td>
<td>0.26</td>
<td>0.24</td>
<td>0.42</td>
</tr>
<tr>
<td>S1(S1)</td>
<td>0.75</td>
<td>0.53</td>
<td>0.60</td>
<td>0.93</td>
<td>1.59</td>
</tr>
<tr>
<td>Average</td>
<td>0.46</td>
<td>0.36</td>
<td>0.45</td>
<td>0.56</td>
<td>0.88</td>
</tr>
</tbody>
</table>

The proposed formulas were also tested for two river flood events at McConnelsville of Muskingum River which occurred during March-April 2002 and January 2004 (Figures 2-4 and 2-5). The Eqs. (2.15) and (2.16) were compared with the simulated result using standard software HEC-RAS 4.0.
Figure 2-5 Comparison of shear velocities using Eqs. (2.15) and (2.16) with HEC-RAS results for January 2004 flood at McConnelsville of Muskingum River.

The selected river reach is relatively straight with bed slope of about $0.7 \times 10^{-4}$. Manning’s roughness $n$ was estimated as $0.03 \text{ m}^{-1/3} \text{s}$ for both flood events and $\beta$ was taken as $5/3$. The flow width, peak discharge, rising time, and unsteadiness of March-April 2002 flood are 120 m, 634.4 m$^3$/s, 84 h, and $2.37 \times 10^{-4}$ and for January 2004 flood are 150 m, 1486.6 m$^3$/s, 102 h, and $1.24 \times 10^{-4}$ respectively. The value of $\alpha$ in Eq. (2.16) was estimated as $3.9 \times 10^{-3} \text{ m}^{-3} \text{s}^2$ for 2002 flood and $1.85 \times 10^{-3} \text{ m}^{-3} \text{s}^2$ for 2004 flood respectively.

2.5. Discussion

Figures 2-1–2-3 and Table 2-3 indicate that Eqs. (2.15) and (2.16) compare well with the other formulas for predicting the shear velocity. Equation (2.16) predicts $U_*$ more accurately, especially in the rising hydrograph limb including its peak and in the later part of the falling
limb. It also has the lowest RMSE among all formulas. However, both Eqs. (2.15) and (2.16) also underestimate \( U^* \) after the peak. This agrees well with Bares et al. (2006), who found that the spatial derivatives of the flow depth assessed using the kinematic wave velocity concept introduces an underestimation in the falling branch of the unsteady flow hydrograph. This effect is more pronounced in Exp. S30-931 because of inertial effects ignored. The dimensionless wave periods in these flume tests were much smaller than 171, a threshold for the kinematic wave (Singh 1996).

In case of river floods both Eqs. (2.15) and (2.16) predicted well (Figures 2-4 and 2-5) when compared with HEC-RAS results. The kinematic wave celerity given by Manning’s law appears to be acceptable for natural river floods in relatively straight reaches of mild-sloping rivers. Most gauge sites in lowland rivers, therefore, should be appropriate for this method.

Relatively better result from Eq. (2.16) shows that the inaccuracy resulting from adopting a constant celerity for the whole hydrograph was smaller or comparable with the effect of noise in the measured data for other equations. As some errors in measurements of flow parameters are unavoidable in real situations, the advantage in using Eq. (2.16) is obvious for practical applications. Moreover, relatively better result during rising period makes it more suitable for applications where the sediment resuspension is of primary concern.

The proposed method has limitations. It assumes that the channel reach is prismatic. Therefore, this method is not applicable to highly irregular channel reaches with marked change in channel width and bottom elevation, or flow over flood plains. Just like Jones’ formula, Eqs. (2.15) and (2.16) assume that the flow is uniform and steady prior to a flood wave so that \( S_o = S_w = S_f \). If this assumption is not met, the bed slope needs to be changed with an appropriate friction slope, as suggested by Schmidt and Yen (2008).
Finally, because the method uses the kinematic wave approximation, it cannot consider the phase lag between depth and discharge peaks. By replacing $\partial h/\partial t$ term in the slope formula with a term containing $\partial Q/\partial t$, it was implied that the discharge arrives at its peak at the same time as the depth, which is not true. The peak discharge arrives before peak depth during the passage of a hydrograph (Graf and Qu 2004; Henderson 1963). Fortunately, this drawback has limited practical implications as most natural flood waves in mild-sloping rivers are non-inertial with very close instantaneous and local wave crests (Henderson 1966). Moreover, the peak shear velocity, which always occurs ahead of discharge peaks, is least affected by this assumption.

2.6. Conclusion

A simple event flow hydrograph-based method was developed for determining shear velocity during river floods. The method is based on St. Venant equations, simplified for floods in mild sloped rivers of constant channel width. Of the two proposed formulas, Eq. (2.16) generally gives more accurate results, especially in the rising limb of the hydrographs, whereas Eq. (2.15) is simpler to use and produces acceptable results in natural rivers. Both of the equations slightly underestimate in the falling limb which is a limitation shared with most of the formulas that use velocity wave concept for unsteady flows.

While the proposed method is comparable to the steady flow method in terms of data requirements, the results are comparable to more complex methods. This method is most appropriate when the river channel is more or less prismatic and the flow conditions are similar to those in typical lowland rivers.

2.7. References


Fenton, J. D. "Calculating hydrographs from stage records." *Proc., Proceedings of 28th IAHR Congress*.


CHAPTER 3. MODELING SEDIMENT TRANSPORT USING EVENT FLOW HYDROGRAPH-BASED METHOD

3.1. Introduction

The prediction of sediment concentrations during flood events is essential to modeling of sediment and sediment-associated contaminant transport in rivers. Extensive efforts have been made in developing various modeling tools for sediment transport, ranging from simple empirical formulas to high dimensional numerical models (García 2006). A review of existing sediment transport models (e.g., Papanicolaou et al. 2008) suggests that many of the existing numerical models either use quasi-steady flow approach for sediment transport, and therefore, do not perform well for highly unsteady flood flows, or are often too complex in terms of input data requirements, making them unsuitable for practical use. A method that can efficiently predict sediment transport by considering unsteadiness of flow would provide a needed bridging tool for studying sediment and sediment-associated contaminant transport.

The average sediment concentration at a station is often found to be higher during rising than corresponding falling stages of a flood hydrograph (Asselman 1999; Langlois et al. 2005). One of the reasons for a positive or clockwise hysteresis of sediment concentration is that the shear stress is greater during rising stages (De Sutter et al. 2001; Nezu et al. 1997; Rowinski et al. 2000; Tu and Graf 1993). Although the suspended sediment behavior is only partly a transport phenomenon due to energy conditions and other factors such as supply and depletion of the sediment also play important roles (Asselman 1999), the resuspension of sediment mainly occurs during the rising stage when the shear stress is greater. Summer and Zhang (1998) argue that the availability of more stream power during rising flows is one of the main reasons for hysteresis effect in sediment discharge rating curve, whereas the hydrologic factors, such as rainfall intensity and areal distribution, and runoff amount and rate, usually have a second major
impact for such an effect. Although the relative role of different factors is important for the ultimate sediment behavior with respect to hysteresis, the prediction of sediment transport during unsteady flows greatly depends upon the accuracy with which the flow parameters such as shear stress, flow velocity and friction slope are determined.

While many studies stress on the accuracy of shear stress and friction slope for estimating sediment transport under unsteady flows, relatively fewer studies, mostly experiment-based (e.g., De Sutter et al. 1999; Lee et al. 2004; Song and Graf 1997), examined the effect of unsteadiness on sediment transport. Investigating the effect of flow unsteadiness on sediment transport in rivers remains a difficult task, partly because of a lack of a practical method for estimating bed shear stress or friction slope for unsteady flows. Solving Saint Venant’s equations for friction slope requires input data that are difficult to measure (Rowinski et al. 2000) and the friction slopes used in the existing formulas for stage-discharge relationships for unsteady flows (e.g., Henderson 1963; Jones 1916; Perumal and Rang Raju 1998; Perumal et al. 2004) use stage instead of flow hydrographs. In this context, a recently developed flow hydrograph-based shear velocity method by Ghimire and Deng (2011) promises a simple yet efficient approach appropriate for flows in lowland rivers. The estimates of shear velocity using this method seem to be better or at least comparable with other established methods and Hydrologic Engineering Center’s River Analysis System (HEC-RAS) simulation results. However, applicability and usefulness of this method for sediment transport modeling in rivers has not been explored yet.

The primary objective of this study is to provide a simple yet effective method for hydrologic engineers to estimate sediment transport during flood events. The unique features of this new method include (1) this method uses a flow hydrograph as primary input data. More specifically, the shear stress, shear velocity, and friction slope are computed using a flood
hydrograph and the shear velocity equation developed by Ghimire and Deng (2011); and (2) conventional simple sediment transport formulas, originally proposed for steady uniform flows, are now extended in this study to unsteady flows or flood events by means of the flow parameters calculated using the hydrograph. To achieve the objective, a complete procedure for computing sediment transport in rivers using the new hydrograph-based method is presented. The complete procedure is further demonstrated through applications to two rivers: Muskingum River, OH and Brazos River, TX for multiple flood events. The performance of the hydrograph-based method is evaluated by comparing it with the HEC-RAS model.

3.2. Methods

3.2.1. Flow Hydrograph-Based Method for Shear Stress

Bed shear stress \( \tau_b \) is one of the most important hydraulic parameters governing sediment transport (Tayfur 2002) and its efficient estimation is crucial to estimating sediment transport in rivers. Many non-cohesive sediment transport functions and most of the cohesive sediment entrainment relations are based on shear stress approach and include \( \tau_b \) or bed shear velocity \( U^* = \sqrt{\tau_b/\rho} \) in their formulations. In fact, most of the bed-load transport formulas are based on shear stress approach. The commonly used bed-material load formulas (e.g., Ackers and White 1973; Engelund and Hansen 1967), derived based on the stream power concept of Bagnold (1966), also use \( \tau_b \) or \( U^* \). The stream power, which is a product of \( \tau_b \) and average velocity \( U \), is considered to be the main driving factor for sediment transport. Note that for natural river flows, which are rarely steady and uniform, \( \tau_b = \rho g R_h S_f \), where \( \rho \) = water density, \( g \) = gravitational acceleration, \( R_h \) = hydraulic radius, and \( S_f \) = friction slope. For sufficiently wide rivers, \( R_h \) may be approximated by the flow depth \( (h) \). Since in gradually varied flows \( S_f \) is different from the bed
slope $S$ and is time dependent, the task of finding $\tau_b$ for a given geometry basically reduces to finding $S_f$, which is the primary variable in almost all sediment transport formulas.

Finding bed shear stress during flood events is a relatively difficult task and it normally involves numerical modeling requiring detailed flow and geometry data. For this reason, many researchers still use methods based on steady uniform flow ($\tau_b = \rho g R h S$), which is not suitable for unsteady flows (Rowinski et al. 2000). Here we use the flow hydrograph-based shear velocity method by Ghimire and Deng (2011) to estimate shear stress and friction slope during unsteady flows. This method is based on Saint-Venant equations, simplified for floods in relatively straight reaches of lowland rivers using the wave velocity concept as discussed in Henderson (1963). The major advantage of this method is that it is simple to use while it requires less field data, mainly flow hydrograph and bed geometry, and gives good results when applied to natural floods in mild-sloped rivers. The bed shear stress, $\tau_b$, is given by:

$$\tau_b = \rho g R h \left( S_o + \alpha \frac{\partial Q}{\partial t} \right)$$  \hspace{1cm} (3.1)

where $S_o$ = friction slope during base flow, $Q$ = flow discharge, $t$ = time, and $\alpha$ is a parameter relating the slope of the flood hydrograph with $\tau_b$ during unsteady flow. In case of natural flood events $\alpha = 1/(BC^2)$, where $B$ = channel width and $C$ = average wave celerity. The parameter $C$ is normally determined using friction law: $C = \beta U$, where $\beta$ is kinematic wave parameter. For a wide rectangular channel ($B >> h$), Manning’s law gives a maximum value of $\beta = 1.67$ (Singh 1996). In mild-sloped rivers, $S_o$ can be approximated with the water surface slope during base flow. For most practical applications where the base flow is nearly uniform and steady, $S_o$ is simply taken as $S$.

Eq. (3.1) indicates that at any instant, $S_f$ is different from $S_o$ by a dynamic component of friction slope i.e., $\alpha(\partial Q/\partial t)$. For increasing flow, this component adds to the friction slope.
whereas for decreasing flow it has the opposite effect. It follows that depending on whether the discharge is increasing or decreasing, the friction slope or the bed shear stress at any time is either greater or smaller than under normal flow conditions for the same discharge. The extent to which $\partial Q/\partial t$ affects the bed shear stress depends on $\alpha$, which is a function of channel width and wave celerity. As the channel width may be considered a constant at any station, small flood wave celerity results in a higher value of $\alpha$, and vice versa. The discharge gradient $\partial Q/\partial t$, therefore, is an important factor for bed shear stress variation in mild-sloped rivers where the celerity of natural river flood waves is relatively low. As the determination of $\partial Q/\partial t$, and thereby the bed shear stress $\tau_b$, requires an event flow hydrograph describing the variation of discharge $Q$ with time $t$, Eq. (3.1) is called the hydrograph-based formula for estimation of the bed shear stress during flood events (Ghimire and Deng 2011).

The variation of $\tau_b$ and other parameters affected during a flood event can be analyzed using Eq. (3.1). For this purpose, we use a hypothetical inflow hydrograph given by a log-Pearson III distribution taken from Weinmann (1977), and analyzed by Chang et al. (1983):

$$Q(0, t) = Q_o + (Q_p - Q_o) \left(\frac{t}{t_p}\right)^{\frac{1}{m-1}} \exp \left[\left(\frac{1}{m-1}\right)\left(1 - \frac{t}{t_p}\right)\right]$$

where $m = 1.15$, $Q_o = 100$ m$^3$/s, and $Q_p = 1000$ m$^3$/s, subscripts $o$ and $p$ denoting initial and peak flow conditions, respectively. The time to peak flow $t_p$ was changed from 10 hours to 48 hours to make the hydrograph more realistic for lowland rivers. Other parameters used for calculation are: $S_o = 0.0001$, $\beta = 1.67$, $B = 200$ m and $n = 0.03$, where $n$ is Manning’s roughness coefficient. The unsteadiness parameter $\Gamma$ (Graf and Song 1995) of the hydrograph was calculated as 0.00017.

For the hypothetical hydrograph given by Eq. (3.2), $\tau_b$ and $S_f$ were determined using the hydrograph-based shear velocity method. Parameter $\alpha$ was found using the friction law approach. The plot of $\tau_b$ during the flood period using both the hydrograph-based formula as well as the
Figure 3-1 Hydraulic parameters during a hypothetical flow event: (a) $\partial Q/\partial T$ and $\tau_b$, (b) $Q$ and $S_f$. 

- $\tau_b$ (Hydrograph-based)
- $\tau_b$ (Steady uniform flow-based)
- $dQ/dt$
steady uniform flow formula is shown in Figure 3-1(a), where $\partial Q/\partial t$ is also plotted. The variation in $S_f$ with respect to $Q$ is shown in Figure 3-1(b). As shown in Figures 3-1(a) and 3-1(b), the peak of $S_f$ precedes the peak of $\tau_b$ and both occur well ahead of peak of $Q$. This is in well agreement with Graf and Qu (2004) and Song and Graf (1996), who investigated the sequence of arrival of peak hydraulic parameters of a hydrograph using the Saint-Venant equations as well as laboratory experiments. They have demonstrated that the peak of friction slope occurs first followed by the peaks of bed shear velocity, average flow velocity, flow discharge, and flow depth. This phenomenon, i.e., the delay in peaks, primarily results from changes in the energy slope caused by the flood wave. The slope of the water surface is greater on the rising limb than on the falling limb of the hydrograph, thus the flow accelerates on the rising and decelerates on the falling limb. Therefore the rising limb of the hydrograph passes at a lower stage than the falling limb for a particular discharge, exhibiting a looped relationship between stage and discharge. Figure 3-1 shows that the peak of $S_f$ occurs slightly ahead of $\partial Q/\partial t$. Note, however, that both $S_f$ and $\partial Q/\partial t$ would reach their peaks simultaneously if the parameter $\alpha$ is considered a constant at a cross-section. The steady uniform flow formula, on the other hand, assumes that all the hydraulic parameters reach peak at the same time, which is obviously not correct. The parameter $\tau_b$ is usually underpredicted in the rising limb of the hydrograph and overpredicted in the falling limb if steady uniform flow formula is used.

### 3.2.2. Flow Hydrograph-Based Sediment Transport Modeling

The hydrograph-based method for shear velocity estimation (Ghimire and Deng 2011), which provides a simple way for calculating hydrodynamic parameters such as bed shear stress and friction slope during an unsteady flow, is extended in this study to include sediment transport processes for unsteady flow. Using this approach, applying common sediment
transport functions for unsteady flows is straightforward. With some simplifying assumptions, for instance, of uniform sediment and an equilibrium transport, estimation of sediment transport may be performed very quickly using a spreadsheet without complex numerical computations.

The steps involved in the hydrograph-based sediment transport method can be summarized as follows:

1. Find $\frac{\partial Q}{\partial t}$ using daily flow records and backward difference approach;
2. Find $h$ from Manning’s resistance formula using $Q$, $S_o$, $B$ and $n$. Then determine $R_h$ and $U$;
3. Find $\alpha (= B^{-1} C^{-2})$ and $S_f (= S_o + \alpha \frac{\partial Q}{\partial t})$ using friction law method for kinematic celerity $C$;
4. Find new values of $h$, $R_h$ and $U$ using calculated $S_f$, instead of $S_o$ in step (2);
5. Repeat steps (2) to (4) until the successive values of $S_f$ do not differ by more than a threshold value, $10^{-6}$. Three to four iterations are usually sufficient;
6. Compute $\tau_b$ and $U*$ using Eq. (3.1);
7. Using computed $\tau_b$, $U*$, and $S_f$ and other parameters at any time instant, apply a selected sediment transport formula to predict the sediment transport capacity of the flow.

Comparative studies of commonly used sediment transport formulas (Chih and Schenggan 1991; Wu et al. 2008; Yang and Huang 2001) indicate that no single formula can reliably predict sediment transport in natural rivers under diverse flow and sediment conditions. These formulas are mostly derived under steady uniform flow conditions using measurements from laboratory flumes or rivers of modest discharges for calibration. Even under normal flow conditions the estimates based on them often spread over an order of magnitude, their extrapolation for floods is at best questionable (Komar 1988). Any confidence in the chosen formula, therefore, should come from the verification of results with the measured data although the assumptions and range of data used in the derivation of the formula are also important. In this study, the total sediment
transport formula of Ackers and White (1973) with modified coefficients by Wallingford (1990) is used. This commonly used total sediment transport formula postulates that only the part of shear stress on the channel bed is effective in causing movement of coarse sediments while for the fine sediments the total shear stress is effective (Yang 1996). The original Ackers and White formula is known to give excessively high results with fine sediments (van den Berg and van Gelder 1993; Wu et al. 2008).

Based on Bagnold’s stream power concept, Ackers and White (1973) used dimensional analysis to derive a total sediment transport function

\[ q_t = cUd \left( \frac{U}{U_*} \right)^{n'} \left( \frac{F_{gr}}{A} - 1 \right)^M \]  

(3.3)

where \( q_t \) = total volumetric sediment transport per unit channel width, \( d \) = sediment particle size, \( F_{gr} \) = a mobility number defined as a ratio of the shear stress on a unit area of the bed to the immersed weight of a layer of grains, given by

\[ F_{gr} = \left( \frac{U^{n'}}{(s-1)gd} \right)^{0.5} \left( \frac{U}{\sqrt{32\log \frac{10h}{d}}} \right)^{1-n'} \]  

(3.4)

Parameters \( A, n', M \) and \( c \) were determined based on flume experiments. All of them are related with the dimensionless grain diameter \( D_* \) given by

\[ D_* = d_{50} \left[ \frac{g(s-1)}{v^2} \right]^3 \]  

(3.5)

where \( d_{50} \) = median diameter of the sediment particles, \( g \) = gravitational constant, \( s \) = specific gravity of the sediment, and \( v \) = kinematic viscosity of water. For a flow with Froude number \( (Fr) < 0.8 \), \( d_{50} > 0.04 \) mm, and \( 1 \leq D_* \leq 60 \), the parameters are calculated as \( n' = 1.0 - 0.56 \log D_* \); \( M = 6.83D_*^{-1} + 1.67 \); \( A = 0.23D_*^{-0.5} + 0.14 \); and \( \log c = 2.79 \log D_* - 0.98 (\log D_*)^2 - 3.46 \).
Figure 3-2 (a) Equilibrium average sediment concentration, and (b) clockwise hysteresis relative to flow, during a hypothetical flow event ($d_{50} = 0.1$ mm)
The coefficients $M$ and $c$ are the revised versions proposed by Wallingford (1990). The representative particle size is taken as $d_{50}$ instead of $d_{35}$ originally proposed by Ackers and White (1973). The detailed description of this method is found in the literature (e.g., van Rijn 1993; Yang 1996).

Using the hypothetical hydrograph and flow parameters used in the previous section, the hydrograph-based sediment transport method was applied to study the SSC variation during an unsteady flow. Ackers and White’s (1973) total sediment transport formula with modified coefficients from Wallingford (1990) was used. For simplicity, uniform sediment of 0.1 mm diameter was used and the bed load contribution was neglected. The average sediment concentration during the entire hydrograph was determined following Van Rijn (1993) procedure. The same procedure was also used to calculate the sediment concentration using the steady uniform flow formula. As shown in Figure 3-2(a), the hydrograph-based sediment transport method predicted higher sediment concentration during rising and lower sediment concentration during falling phase of the hydrograph in comparison with the steady uniform flow formula method. The peak sediment concentration from the hydrograph-based method also occurs well ahead of that from the steady uniform formula. The clockwise hysteresis of sediment concentration produced with the proposed hydrograph-based sediment transport method is shown in Figure 3-2(b).

### 3.3. Application of Hydrograph-Based Method to Flood Events

#### 3.3.1. Overall Approach

The new method was applied for simulating sediment transport at two USGS gauge sites: i) McConnelsville, Ohio on Muskingum River, and ii) Richmond, Texas on Brazos River. These
sites are located in mild-sloped reaches of lowland rivers with fine sand bed where the suspended sediment is dominant.

In order to test the applicability of the hydrograph-based sediment transport method, the predicted suspended sediment concentrations by the hydrograph-based sediment transport method are compared with the HEC-RAS model results. HEC-RAS is a public domain 1-D numerical model available from Hydrologic Engineering Centre. It allows users to perform 1-D steady flow, unsteady flow calculations, movable boundary sediment transport computations and water temperature analysis (Brunner 2010). The sediment transport module incorporates most of the features of its predecessor model HEC-6, which is a 1-D quasi unsteady mobile bed model widely used in many river sediment studies (Bhowmik et al. 2008). HEC-RAS is suitable for simulation of sediment transport resulting from scour and deposition over a moderate time period although the application to single events is also possible. The model uses quasi-unsteady flow approach utilizing three time durations of decreasing order: flow duration, computation increment and the mixing time step (Brunner 2010). In this study, Ackers and White (1973) function with modified coefficients from Wallingford (1990) are used in both hydrograph-based sediment transport method and HEC-RAS models.

As the measured suspended sediment data include wash load and no measured bed material data are available, two important assumptions are made to evaluate the applicability of the new method. First, the formula for bed-material load is considered to be suitable for predicting the total sediment load including wash load. Second, the bed load is assumed to be negligible in comparison with the suspended load. In other words, the total sediment concentration is considered to be equal to the measured sediment concentrations.
The evaluation of the goodness-of-fit between measured and computed concentrations using hydrograph-based sediment transport method is performed using statistical parameters: discrepancy ratio \((R)\), mean discrepancy ratio \((R_m)\), mean normalized error \((MNE)\), root mean square error \((RMSE)\) and normalized root mean square error \((NRMSE)\).

\[
R = \frac{C_{ci}}{C_{mi}}; R_m = \frac{1}{N} \sum_{i=1}^{N} R_i
\]

\[
MNE \, (\%) = \frac{100}{N} \sum_{i=1}^{N} \frac{C_{ci} - C_{mi}}{C_{mi}}
\]

\[
RMSE = \left( \sum_{i=1}^{N} \frac{1}{N} \left( \frac{C_{ci} - C_{mi}}{C_{mi}} \right)^2 \right)^{1/2}; NRMSE = \frac{RMSE}{\sum_{i=1}^{N} \frac{1}{N} C_{mi}}
\]

where \(C_{ci}\) and \(C_{mi}\) are computed and measured sediment concentrations in mg/l, respectively, and \(N\) is the number of data sets. For a perfect fit \(R\) should be close to one, whereas smaller values of \(MNE, RMSE,\) and \(NRMSE\) indicate a better fit. HEC-RAS model results are also compared using the same statistical measures listed in Eqs. (3.6)–(3.8).

### 3.3.2. Muskingum River at McConnelsville, OH

This study site is in the lower part of the Muskingum River at McConnelsville, Ohio. This is a U.S. Geological Survey (USGS) stream flow-gaging station 03150000 located in a relatively straight reach of the river. It has a very mild bed slope of about \(7 \times 10^{-5}\) and a width of 130 m. The average daily flow and drainage area are 220 m\(^3\)/s and 19220 km\(^2\), respectively. The Manning’s roughness coefficient \((n)\) is estimated to be 0.03 m\(^{-1/3}\)/s.

The observed flow and suspended sediment data from the Water Quality Laboratory (WQL) of Heidelberg College during high flow seasons of 2001, 2002, and 2003 were used in this study. As a part of its Ohio Tributary Monitoring Program, the WQL has been monitoring the water...
quality and has records of instantaneous daily suspended sediment and flow data since 1994. The measured total suspended solids (TSS) were used for performance evaluation of the proposed method, although they are usually negatively biased when comparing with suspended sediment concentration (SSC). No adjustment was done due to lack of paired data at this site. Although it is not an ideal option, TSS is commonly used (e.g., Ziegler et al. 2000; Ziegler and Nisbet 1994) for model calibration in lowland rivers with fine sediments.

The proposed hydrograph-based sediment transport method was evaluated by comparing with the HEC-RAS model results. First, the sediment concentration was determined using hydrograph-based sediment transport method involving seven steps described previously. The following parameters were used for this site: 1) $d_{50} = 120 \mu m$, 2) sediment settling velocity $w_s = 0.01 m/s$, 3) Shields critical shear stress parameter $\theta_{cr} = 3.04$, 4) dynamic viscosity of water $\nu = 1 \times 10^{-6} m^2/s$, and 5) particle Reynold’s number $Re_p = 5.29$. The $d_{50}$ size was chosen as an average value from the grain size distribution of suspended sediment after excluding fines smaller than 62μm size. The Ackers and White’s (1973) total sediment transport formula with modified coefficients from Wallingford (1990) was used. The parameters in Ackers and White formula were computed as $A = 0.72$, $c = 0.0045$, and $M = 3.92$.

Next, the HEC-RAS model was applied to simulate the total sediment concentration using the same geometric ($S_o, B$), hydraulic ($Q$), and sediment ($d_{50}$) parameters as those used in the hydrograph-based method. Ackers and White transport function with the same calibration parameters ($A, c, \text{and } M$) was used for sediment transport. Van Rijn method for fall velocity was used and no cohesive sediment option was selected. The downstream boundary was kept sufficiently far away from the considered cross-section to have negligible backwater effect. Equilibrium load and normal depth are set as the upstream sediment and downstream flow
boundary conditions, respectively. Bed slope was specified for calculating the normal depth using Manning’s equation in the downstream boundary.

Finally, the total sediment concentration by hydrograph-based sediment transport method as well as HEC-RAS model at McConnelsville, OH during high flow seasons of 2001, 2002, and 2003 are compared with the measured SSC as shown in Figures 3-3 (a), 3-3 (b), and 3-3 (a) respectively. The computed flow depths using hydrograph-based shear velocity method and HEC-RAS are also shown in these figures along with the flow discharge. The scatter plot of measured versus computed sediment concentration using hydrograph-based sediment transport method at McConnelsville, OH is shown in Figure 3-4. The proposed method was also compared

![Figure 3-3](image_url)

**Figure 3-3** Measured and computed sediment concentrations using hydrograph-based method and HEC-RAS model at McConnelsville, OH during high flow seasons: (a) 2001, (b) 2002, and (c) 2003 (top); measured discharge Q and computed flow depths h (bottom).
with the HEC-RAS model using several statistical measures (Table 3-1), which include percentage of data in $0.5R–2R$ range denoted by $R_{0.5-2.0}$ (%).

![Graph showing measured and computed sediment concentration using hydrograph-based method at McConnelsville, OH.](image)

**Figure 3-4** Comparison of measured and computed sediment concentration using hydrograph-based method at McConnelsville, OH.

Clockwise hysteresis of sediment concentration was observed during most of the single and multi-peak flood events at McConnelsville, OH. The measured sediment concentration during two typical single flood events of 2001 is plotted and compared with computed results using hydrograph-based method, as shown in Figure 3-5.
Figure 3-5 Hysteresis of sediment concentrations at McConnelsville during two single peak flood events: (a) 1/27/2001–2/10/2001, (b) 12/15/2001–12/27/2001.
Table 3-1  Summary of comparison between computed and measured sediment concentration at McConnelsville, OH

<table>
<thead>
<tr>
<th>Method</th>
<th>$R_m$</th>
<th>$R_{0.5-2}$ (%)</th>
<th>MNE (%)</th>
<th>RMSE</th>
<th>NRMSE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hydrograph-based</td>
<td>1.35</td>
<td>82.6</td>
<td>52.4</td>
<td>31.6</td>
<td>0.57</td>
</tr>
<tr>
<td>HEC-RAS</td>
<td>1.36</td>
<td>79.4</td>
<td>54.3</td>
<td>37.4</td>
<td>0.67</td>
</tr>
</tbody>
</table>

3.3.3. Brazos River at Richmond, TX

This study site is the USGS stream flow-gaging station 08114000 on the Brazos River at Richmond, TX. The channel has a bed slope of $1.2 \times 10^{-4}$ and an average width of 75 m at this station. The average daily discharge and drainage area are 214.9 m$^3$/s and 92050 km$^2$, respectively. Manning’s roughness ($n$) is estimated to be $0.03$ m$^{-1/3}$/s. The $d_{50}$ for the total width

![Figure 3-6](https://example.com/figure36.png)

Figure 3-6  Measured and computed sediment concentrations using hydrograph-based method and HEC-RAS model at Richmond, TX during high flow seasons of 1974-1975 (top); measured discharge Q and computed flow depth h (bottom).
sediment load is estimated as 100 μm. Van Rijn formula gives $w_s = 0.008 \text{ m/s}$, $\theta_{cr} = 0.078$, $v = 1 \times 10^{-6} \text{ m}^2/\text{s}$, and $Re_p = 4.08$.

Following the same procedure as in the McConnelsville site, the total sediment concentration at Richmond, TX was first computed using the hydrograph-based sediment transport method. Ackers and White formula with parameters $A = 0.284$, $c = 0.0033$, and $M = 4.434$ was used. The HEC-RAS model was then run using the same calibration parameters ($A$, $c$, and $M$). The computed sediment concentrations using both the hydrograph-based method and the HEC-RAS model at Richmond, TX during Sept 1974 to June 1975 along with the measured

![Figure 3-7 Comparison of measured and computed sediment concentrations using hydrograph-based method at Richmond, TX.](image)
daily SSC data (http://co.water.usgs.gov/sediment/seddatabase.cfm) are shown in Figure 3-6. The computed flow depths using hydrograph-based shear velocity method and HEC-RAS are also shown along with the flow discharge inputs. The scatter plot of measured versus computed sediment concentration using hydrograph-based sediment transport method at Richmond, TX is shown in Figure 3-7. A comparison of the measured sediment concentration values and computed results using several statistical measures is given in Table 3-2.

Table 3-2 Summary of comparison between computed and measured sediment concentration at Richmond, TX.

<table>
<thead>
<tr>
<th>Method</th>
<th>$R_m$</th>
<th>$R_{0.5-2}$ (%)</th>
<th>$MNE$ (%)</th>
<th>$RMSE$</th>
<th>$NRMSE$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hydrograph-based</td>
<td>1.04</td>
<td>96.2</td>
<td>25.0</td>
<td>477.4</td>
<td>0.38</td>
</tr>
<tr>
<td>HEC-RAS</td>
<td>1.05</td>
<td>93.1</td>
<td>39.1</td>
<td>574.1</td>
<td>0.46</td>
</tr>
</tbody>
</table>

3.4. Discussions

3.4.1. Performance of Hydrograph-Based Method

3.4.1.1. Comparison with HEC-RAS Model

The visual inspection of Figure 3-3 through 3-7, and Tables 3-1 and 3-2 shows that the proposed hydrograph-based method gives total sediment concentration profiles comparable with those from the advanced numerical model HEC-RAS when it is applied to flood events in channels with simple geometry and uniform sediment. The results for both study sites indicate that the hydrograph-based method consistently gives better results than the HEC-RAS model during rising as well as falling phases of the hydrographs. For the same flow, geometry, and sediment input data, the new method produces slightly higher concentrations during rising periods of the hydrographs including their peaks, and lower concentrations during falling periods. This is probably due to better accounting for unsteadiness of flow by the hydrograph-
based method in comparison with the HEC-RAS sediment module. The HEC-RAS sediment module works under quasi-unsteady flow principle by approximating a hydrograph into discrete steady flow profiles. Better correlation of the new method with observed data, expressed by all statistical parameters $R$, $R_m$, $MNE$ ($\%$), $RME$, and $NRMSE$ may also be explained by the same reason.

As expected, the flow depths predicted by the HEC-RAS sediment transport module are generally larger than those predicted by the hydrograph-based method during rising flows and flood peaks (Figures 3-3 and 3-6). The maximum differences in flow depths at McConnelsville during 2001, 2002, and 2003 were found out as 11.3%, 12.7%, and 12.1% respectively, whereas the difference at Richmond was 13.5%. During the falling flows, the depths predicted by the hydrograph-based method are slightly larger for all floods at McConnelsville, whereas the difference is very small at Richmond. It should be noted that this comparison of depths has a more qualitative character, since the HEC-RAS sediment module uses steady flow profiles.

Ghimire and Deng (2011) reported that the bed shear velocities from hydrograph-based method in general are close to those from unsteady flow module of HEC-RAS, except near tail part of the falling limb where the hydrograph-based method tends to underestimate the shear velocity. It is expected that a similar discrepancy also exist in depths; however for sediment concentration simulation, its effect is practically insignificant.

### 3.4.1.2. Comparison with Measured Data

In case of McConnelsville of the Muskingum River, the plots of data during three consecutive high flow seasons from 2001 to 2003 generally show a good agreement between computed and measured sediment concentrations. The hydrograph-based method has predicted well during low and medium flows. The prediction appears to be slightly better when the flood
events are spaced apart. But when two or more flood events occur within a short period, such as during flood events in May-June 2002, as shown in Figure 3-3b, the predicted concentrations during falling phases are much higher than the measured ones. This is probably due to depletion of fine sediments or by armoring effects. None of these effects are considered in this study. In general, the predicted sediment concentrations during rising phases of the hydrograph better correlate with the measured concentrations. The prediction is better during most periods of the rising phase, but both the hydrograph-based method and HEC-RAS model generally underpredict during peak flows.

In general, the performance of the hydrograph-based method in Richmond of Brazos River is similar to that in the McConnelsville site. The prediction is relatively better during rising phases of the flood hydrograph than during falling phases, when the predicted sediment concentrations are higher than the measured concentrations (Figure 3-6). The predicted concentrations are smaller during most of the peak floods but are slightly higher during receding flows. Based on the percentage of data in range of 0.5 to 2.0 $R; R_m; MNE$ and $NRMSE$ (Figure 3-7, Table 3-1 and 3-2), the goodness-of-fit results appear to be slightly better in Richmond than in McConnelsville. Relatively higher $RMSE$ in Richmond is due to much higher average sediment concentration (1252 mg/L) than in McConnelsville (56 mg/L).

3.4.1.3. Hysteresis Effect

The hydrograph-based method is able to reproduce a clockwise hysteresis of sediment concentration observed at study sites, particularly at McConnelsville, OH (Figure 3-5). A slight overestimation during rising and underestimation during falling limb by the hydrograph-based method is probably due to ignoring of some of the terms in St. Venant’s equation while deriving the formula for bed shear velocity (Ghimire and Deng 2011). Although the difference is small for
natural floods, the bed shear velocity determined using hydrograph-based approach tends to be higher during rising and lower during falling phases of the hydrograph. Note that due to the complexity of processes governing sediment dynamics in a natural river system, patterns of the discharge-sediment concentration relationships are highly variable (Asselman 1999; Klein 1984; Williams 1989) and no general methods can be found appropriate for all cases. While an anti-clockwise hysteresis is observed when the sediment origins from a distant field (Asselman 1999; Klein 1984; Williams 1989) or due to temporal lag effect caused by inability of alluvial system to respond to immediately changing flow conditions (Phillips and Sutherland 1990), the clockwise hysteresis is mainly a result of either unsteady effect or sediment depletion during falling phases of the hydrograph. The hydrograph-based method is appropriate when the trends in shear stress explain the observed trend in sediment transport during unsteady flows. This is the case at both study sites where the higher sediment concentration during rising stage of the flood could be explained by the increased bed shear stress due to unsteadiness of flow and vice versa. The other reasons for hysteresis, such as the timing and amount of sediment arriving at measured site or the proximity of its sources (Williams 1989), should be handled separately.

3.4.2. Limitations

The major limitations of the present study come from the lack of measured bed material data. The sediment transport functions are developed and tested for bed material loads which do not include the wash load. By using the SSC or TSS data instead of bed material data, an additional uncertainty due to the wash load is introduced. The wash load refers to that size fraction of the total sediment which is not present in the streambed in significant amounts (Einstein 1950). Once introduced into the channel, it is kept in suspension by flow turbulence and passes through the channel with negligible interaction with the streambed. As the wash load
transport rate is generally dependent on the supply from the upstream, the sediment transport functions are less effective in quantifying its transport rate. There is no simple method for finding the contribution of wash load to the suspended load without additional information on the size distribution. Most of the SSC data available at USGS sites includes wash load and the information on wash load is rarely available. The bed material data during flow events are almost non-existent. Due to this reason, the suspended load or bed material load formulas are often used without considering the contribution of the wash load (e.g., Wu et al. 2008; Ziegler et al. 2000).

The median particle size $d_{50}$ is often estimated by including the wash load or determined as a calibration parameter, which is theoretically incorrect.

It is assumed that a sediment transport formula for bed material concentration is also applicable for computing total sediment transport concentration. Although such an approach is theoretically questionable, it worked well for the sediment laden Yellow River (Wu et al. 2008). The results from both hydrograph-based method and the HEC-RAS model (Figure 3-3 to 3-7) also show that the improved Ackers and White formula predicts the total sediment concentration fairly well. The concentration seems to be a function of local hydrodynamics in these sites and the upstream supply of sediment does not alter a trend in sediment transport concentrations.

By using measured TSS or SSC data as a surrogate for total bed material concentration, the bed load is assumed to be either a small fraction of the total sediment load or practically equal to the wash load. In general, the bed load constitutes a small portion (5-15%) of the total load in lowland rivers (U.S. Army Corps of Engineers 1995). During floods, however, the bed load transport may be more important, especially in rivers with small catchments (Turowski et al. 2010). Note that the bed load data are rarely available, and there is no widely accepted method for partitioning bed load from total sediment load. It is not surprising why many authors either
ignore the bed load or use a fixed percentage of the total sediment load. This percentage varies greatly from site to site and flood to flood, the most common numbers being between 10% and 20% (Simons and Sentürk 1992; Summerfield and Hulton 1994). Turowski et al. (2010) provides an excellent review on this subject. In the present study, we assume that the bed load and the wash load are either not significant or approximately equal, and therefore, allow comparing the computed total sediment load directly with the measured SSC or TSS data. Compared to the uncertainties normally involved in the sediment transport prediction, this assumption should not be a problem for evaluating the hydrograph-based sediment transport method.

The other limitations such as the assumption of equilibrium transport and uniform sediment are used for the simplicity of computation. The selected reaches are relatively stable and no severe sediment erosion or deposition is expected. The assumption of equilibrium sediment inflow and uniform channel geometry ensures that no changes in bed profile occur during the study period. As the sizes of the sediment for different flows are not available, it is also kept constant for all flows, which may not represent the real situation.

The hydrograph-based method is suitable for prismatic reaches in mild sloped rivers. This method is not appropriate for highly irregular reaches with marked changes in channel geometry or flow over flood plains. Since this method uses the kinematic wave approximation, it cannot consider the phase lag between depth and discharge peaks. However, it has little practical implication for sediment transport as most natural flood waves in mild-sloped rivers are non-inertial with close instantaneous and local wave crests. Obviously, this method is difficult to apply for non-equilibrium sediment transport or with non-uniform sediments where numerical modeling is most desirable.
3.5. Conclusions

A new method, called the hydrograph-based method, for modeling sediment transport during flood events in mild-sloped rivers is proposed. A complete procedure for using the new method is outlined and demonstrated through applications to two U.S. rivers. The following conclusions can be drawn based on the application findings:

- Under the assumptions of equilibrium transport, uniform sediment and prismatic channel, the application of the proposed method indicates that this method is comparable with more advanced numerical models such as HEC-RAS in terms of overall accuracy and gives relatively better results during rising as well as falling phases of large flood events. This is an encouraging result, especially as the proposed method is much simpler, less data intensive and does not require numerical computation.

- This method is able to reproduce clockwise hysteresis of sediment concentration frequently observed in natural rivers. Obviously, this method assumes that the trends in shear stress explain the trends in sediment transport. When the sediment supply is limited, or the sediment transport is largely independent of channel hydraulics, this method is not directly applicable.

- The hydrograph-based sediment transport method can provide a more realistic value of sediment concentrations during flood events by considering the tendency of bed shear stress to be greater during rising stages of the hydrographs than corresponding falling stages.

- With this simple method to consider the unsteadiness of the flow, the applicability range of conventional sediment transport formulas is practically extended to unsteady flows. This method, therefore, may be useful for evaluating performance of conventional sediment transport formulas for flood flows.
The proposed method appears to be a more practical alternative to advanced numerical models that are included in the contaminant transport modeling framework but are costly in terms of input data. With this simple yet effective tool, the need to compromise with the accuracy by resorting to the steady uniform flow formula for simplicity during unsteady flow events is largely eliminated.

3.6. References


CHAPTER 4. CHARACTERIZING SOLUTE TRANSPORT IN LARGE AND MEDIUM SIZE RIVERS USING VARIABLE RESIDENCE TIME MODEL²

4.1. Introduction

Hyporheic exchange is the process through which surface stream water and subsurface ground water exchange solute (nutrients, contaminants, and dissolved oxygen) and energy across the sediment-water interface. The exchange controls nutrient uptake and retention in streams by increasing both residence time and the contact of nutrients with biogeochemically active surfaces (Ensign and Doyle 2005; Hinkle et al. 2001). The exchange can attenuate pollutants in contaminated streams (Gandy et al. 2007; Smith 2005). The exchange also determines the thermal regime of channel bed sediments (Cardenas and Wilson 2007; Smith 2005) and the abundance of microbial and invertebrate communities in hyporheic zones (Boulton et al. 1998; Jones and Mulholland 2000). Over the past decades, extensive investigations have been conducted to understand and simulate processes underlying hyporheic exchange-induced long-tailed residence time distributions (RTDs) of solute in streams and rivers (Boano et al. 2007; Briggs et al. 2009; Cardenas et al. 2004; Elliott and Brooks 1997; Gooseff et al. 2003; Haggerty et al. 2002; Kasahara and Hill 2006; Packman and Bencala 2000; Runkel 1998; Soulsby et al. 2001; Wörman et al. 2002). The investigations have found that solute concentration breakthrough curves (BTCs) observed in streams can be described using exponential or power-law RTDs (Gooseff et al. 2003) or lognormal RTDs (Cardenas et al. 2004). The reported RTDs are generally based on tracer experiments performed in small streams. Alexander et al. (2000) found that nitrogen-loss rates in streams decline rapidly with increasing stream depth. This means that channel size has a significant effect on mass losses in streams. However, it is not

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clear whether channel size and mass losses affect RTDs. In fact, the influence of river channel size on RTDs is rarely studied.

The primary goal of this study is to analyze the effect of channel size on solute transport processes in streams characterized by the shapes of RTDs. This goal can be achieved by using field tracer test data collected from rivers and the Variable Residence Time (VART) model developed by Deng and Jung (2009). The US Geological Survey conducted nationwide dye tests in 1960s and the early 1970s on fifty-one river reaches (Nordin and Sabol 1974), ranging from about 300 m–300 km and delivering flows from about 0.85–6820 m$^3$/s including hyporheic flow and associated mass losses/gains. The field tracer experiments resulted in excellent data for evaluating effects of channel size and hyporheic exchange on longitudinal transport of solute in rivers. The effect of channel size can be analyzed by comparing simulated RTDs for rivers with distinct flow depths as discussed in Deng et al. (2010).

### 4.2. Variable Residence Time (VART) Model

#### 4.2.1. Basics of VART Model

The VART model (Deng and Jung 2009) is based on the concept of varying residence time for simulating transient storage process.

\[
\frac{\partial C}{\partial t} + U \frac{\partial C}{\partial x} = K_s \frac{\partial^2 C}{\partial x^2} + \frac{A_{adv}}{A} + \frac{A_{dif}}{A} \frac{1}{T_v} (\lambda C_s - C) + \frac{q_h}{A} (C_L - C) \quad (4.1)
\]

\[
\frac{\partial C_s}{\partial t} = \frac{1}{T_v} (C - C_s) + \frac{q_h}{A_{adv} + A_{dif}} (C_h - C_s) \quad (4.2)
\]

\[
A_{dif} = 4\pi D_s t_s \quad (4.3)
\]

\[
T_v = \begin{cases} T_{min} & \text{for } t \leq T_{min} \\ t & \text{for } t \geq T_{min} \end{cases} \quad (T_{min} > 0) \quad (4.4)
\]
where $C, C_s, C_h, C_L = $ solute concentrations in main channel, storage zones, hyporheic exchange induced water gains, and lateral inflows respectively; $U = $ cross-sectionally averaged flow velocity; $t = $ time; $T_V = $ actual varying residence time of solute; $t_S = $ time since the solute release from storage zones to the main stream; $T_{min} = $ minimum mean residence time for solute to travel through the advection-dominated storage zone; $A, A_S = $ cross-sectional flow areas of main channel and transient storage zones; $A_{adv}, A_{diff} = $ areas of advection-dominated transient storage zone and effective diffusion-dominated transient storage zone; $q_h, q_L = $ subsurface hyporheic exchange-induced water gain/loss and surface lateral inflow rates per unit channel length; $K_S = $ longitudinal Fickian dispersion coefficient excluding the transient storage effect; $D_S = $ dispersion or effective diffusion coefficient in hyporheic zone; $\lambda = $ ratio of hyporheic inflow rate to outflow rate ($\lambda > 1 $ for gain and $\lambda < 1 $ for loss of hyporheic exchange-induced water). If significant mass gains or losses are caused by lateral inflows or outflows, according to the mass conservation principle the concentration differential $(C_L - C)$ in Eq. (4.1) can be replaced with $(M - 1)C$, where $M$ is the recovery ratio of tracer/mass (Nordin and Sabol 1974), and the last term in Eq. (4.2) can be dropped. Likewise, if significant mass gains or losses are caused by hyporheic inflows or outflows, the concentration differential $(C_h - C_S)$ in Eq. (4.2) can be replaced with $(M - 1)C_S$ and the last term in Eq. (4.1) can be removed.

The flow velocity $U$, channel cross-section area $A$, and lateral inflow/outflow rate $q_L$ are commonly known or calculable for a given stream reach. If there are no observed hyporheic flow data available, the parameter $\lambda$ can be roughly estimated using the relationship $\lambda \approx Q_D/Q_U$ where $Q_D$ and $Q_U$ are stream flow rates at downstream and upstream sampling stations in a stream reach, respectively. Excluding the lateral inflow/outflow rate $q_L$, the parameter $q_h$ can be
estimated using the relationship \( q_h = (Q_D - Q_U)/L \) where \( L \) is the length of the stream reach. If a water loss \((\lambda < 1)\) occurs, there is no need to determine \( q_h \) because the second term on the right-hand side of Eq. (1b) disappears \((C_h = C_S)\). Therefore, there are at most four parameters \((K_S, D_S, A_{adv}, \text{and } T_{min})\) to estimate in the VART model. Specifically, the four parameters \((K_S, D_S/A, A_{adv}/A, \text{and } T_{min})\) can be determined using tracer test data and the fractional Laplace transform-based parameter estimation method proposed by Deng et al. (2006) or other parameter optimization methods (Scott et al. 2003). There are only three parameters \((K_S, D_S/A, \text{and } A_{adv}/A)\) to estimate for small and moderate-sized streams and some large rivers due to the new finding of a simple method for calculation of the parameter \( T_{min} \).

4.2.2. Modification of Eq. (4.1)\(^3\)

While the Eq. (4.1) was derived based on mass conservation principle, a constant discharge \( Q = U A \) was assumed in the control volume (equation 10a, Deng and Jung 2009). As a result, the variation of discharge from net gains of water from hyporheic exchange was ignored. This may lead to an error, especially when the hyporheic exchange is significant. More accurate alternative to Eq. (4.1) is derived herein following the conceptual model and control volume approach by Deng and Jung (2009).

The rate of mass accumulation in the control volume of the mainstream is equal to the net mass flux into the control volume through all control surfaces,

\[
(Adx) \frac{\partial C}{\partial t} = \sum_{cs} \dot{m}_i - \sum_{cs} \dot{m}_o
\]

where, \( \dot{m}_i = \) mass influx, \( \dot{m}_o = \) mass efflux across control surfaces \((cs)\). The net mass flux through all the control surfaces may be written as

\[^3\] This modification is based on the comments of a reviewer
\[ \sum_{cs} \dot{m}_i - \sum_{cs} \dot{m}_0 = -\frac{\partial (CQ)}{\partial x} dx + \frac{\partial}{\partial x} \left(K_s A \frac{\partial C}{\partial x} \right) dx \]
\[ + q(\lambda C_s - C) + Q_L^{in} C_L - Q_L^{out} C \]

(4.7)

where, \(q\) = hyporheic exchange-induced flow rate from the mainstream to the storage, \(Q_L^{in}\) = lateral inflow rate and \(Q_L^{out}\) = lateral outflow rate. The first and second terms on the right side of the equation (4.6) are net advective and dispersive mass fluxes through upstream and downstream control surfaces in the main channel. The third term is net mass flux across sediment-water interface due to hyporheic exchange, and the fourth and fifth terms are mass fluxes across stream banks due to lateral inflow and outflow respectively.

Combining Eqs. (4.6) and (4.7), and rearranging gives,
\[ (Adx) \frac{\partial C}{\partial t} + Q \frac{\partial C}{\partial x} dx + C \frac{\partial Q}{\partial x} dx = \frac{\partial}{\partial x} \left(K_s A \frac{\partial C}{\partial x} \right) dx \]
\[ + q(\lambda C_s - C) + Q_L^{in} C_L - Q_L^{out} C \]

(4.8)

The water mass balance over a stream reach of length \(dx\) in a steady condition may be written as
\[ \frac{\partial Q}{\partial x} dx = Q_L^{in} - Q_L^{out} + q(\lambda - 1) \]

(4.9)

where \(q\lambda\) is the hyporheic exchange-induced flow rate from storage to the stream. The first two terms give the net lateral inflow to the main stream whereas the third term is the net gain in the stream flow rate caused by the hyporheic exchange.

Substitution of Eq. (4.9) into Eq. (4.8) and dividing both sides by \(Adx\) yields
\[ \frac{\partial C}{\partial t} + \frac{Q}{A} \frac{\partial C}{\partial x} = \frac{1}{A} \frac{\partial}{\partial x} \left( K_s A \frac{\partial C}{\partial x} \right) + \frac{q\lambda}{Adx} (C_s - C) + \frac{Q_L^{in}}{Adx} (C_L - C) \]

(4.10)

Finally, substitution of \(U = Q/A, \ q_L = Q_L^{in}/dx, \ T_v = A_s dx/q = (A_{adv} + A_{dif}) dx/q\), and assuming that \(A\) and \(K_s\) are constant along the stream reach results in
\[
\frac{\partial C}{\partial t} + U \frac{\partial C}{\partial x} = K_s \frac{\partial^2 C}{\partial x^2} + \frac{A_{adv} + A_{dif}}{A} \frac{\lambda}{T_v} (C_s - C) + \frac{q_L}{A} (C_L - C)
\]

which is the same as Eq. (4.1) except for the second term on the right side which is strongly dependent on \( \lambda \). The Eq. (4.11) together with Eqs. (4.2)–(4.5) will constitute the VART model applied in this study.

A split-operator method is utilized to split Eq. (4.11) into a pure advection sub-equation and a dispersion sub-equation with the transient storage and lateral inflow terms. The pure advection part is solved using Semi-Lagrangian approach (Deng et al. 2006). The dispersion sub-equation with transient storage and lateral inflow terms in conjunction with Eqs. (4.2)–(4.5) is solved using an implicit finite-difference method.

**4.3. Residence Time Distributions in Large Rivers**

Most existing models for stream solute transport are tested using tracer experiment data collected from small streams. To understand the effect of channel size on RTDs and to evaluate the performance of VART model in simulating solute transport in large rivers, data of tracer injection experiments conducted in the Mississippi River, Red River, and Bayou Bartholomew are obtained from the USGS report by Nordin and Sabol (1974).

The Mississippi River is one of the largest rivers in US and in the world. A dye test was conducted in the Mississippi River on August 7, 1968. Four sampling sites were located at 54.72 km (Crystal City, Missouri), 96.56 km (Genevieve, Missouri), 117.48 km (Chester, Illinois), and 294.51 km (Cairo, Illinois) downstream of the dye injection site. The reported flow discharge along the four river reaches was 6824.4 m\(^3\)/s, meaning that \( \lambda = 1 \), \( q_L = 0 \), and \( q_h = 0 \). Flow depths at the four sites were 9.24 m, not available, 8.90 m, and 7.34 m. Recovery ratios of tracer at the four sites were 0.972, 0.843, 1.113, and 0.589, respectively. It should be noted that the recovery
ratio values are directly taken from the USGS report (Nordin and Sabol 1974) in which the recovery ratios were determined based on the mass conservation principle. The ratios are employed to represent solute losses ($M < 1$) or gains ($M > 1$) in the VART model. In order to identify the effect of water and solute losses/gains on RTDs, numerical simulations are performed under two cases where water and solute losses/gains are omitted in Case O and included in Case I. Estimates of VART parameters, used in producing the lognormal RTDs for the four reaches shown in Figure 4-1, are listed in Table 4-1 under the two (O and I) cases. Figure 4-1 shows that the same type of lognormal RTD was maintained along the 294.51 km long river reach. Table 4-1 indicates that the inclusion of solute losses in reaches 1, 2, and 4, and the solute gain in reach 3 have no effect on root mean square errors (RMSEs) because $q_L = 0$ and $q_h = 0$ make the last terms in Eqs. (1a) and (1b) disappear. It means that the solute losses/gains in the Mississippi River were caused by some other mechanisms that are not included in the VART model. The RMSE is commonly employed as a metric for evaluating the goodness of fit of model simulations to measured data (Bard 1974).

The Red River is a tributary of the Mississippi River. A dye test in the Red River was conducted on April 7, 1971. Four sampling sites were located at 5.74 km (Grand Ecore, Louisiana (LA)), 75.64 km (Colfax, LA), 132.77 km (Alexandria, LA), and 193.12 km (St. HWY 115, Moncla, LA) downstream of the dye injection site, respectively. Flow discharges at the four sites were 230.2 m$^3$/s, 245.2 m$^3$/s, 249.5 m$^3$/s, and 249.5 m$^3$/s, respectively. Hyporheic exchange-induced water gains in reaches 1–4 were 0, $2.15 \times 10^{-4}$ m$^3$/s-m, $7.44 \times 10^{-5}$ m$^3$/s-m, and 0, respectively. No lateral inflows/outflows were reported. Flow depths at the four sites were 4.82 m, not available, not available, and 1.62 m, respectively. Recovery ratios of tracer at the four sites were 0.741, 0.740, 0.695, and 0.587, respectively. Values of parameters used in
producing the lognormal (VART–L) RTDs for the four reaches shown in Figure 4-2 are listed in Table 4-2 for the two cases. Figure 4-2 shows that the same type of lognormal RTD was maintained along the 193.12 km long river reach. Table 4-2 indicates that the incorporation of solute losses in the reaches and hyporheic flow-induced water gains in reaches 2 and 3 into the VART model reduces RMSEs (especially for the last reach) and thus improves the fitting of simulated RTDs to the observed BTCs.

Figure 4-1 RWT (Rhodamine WT) concentration BTCs observed (circles) on August 7, 1968 in four sampling stations in the Mississippi River and simulated (lines) using the VART model for an instantaneous dye addition. The curves VART-LO and VART-LI are produced with the parameter values in Case O and Case I shown in Table 4-1, respectively.
Table 4-1 Parameter values used in Figure 4-1 for the Mississippi River.

<table>
<thead>
<tr>
<th>Case</th>
<th>Reach</th>
<th>U (m/s)</th>
<th>$K_s$ (m$^2$/s)</th>
<th>$A_{adv}/A$</th>
<th>$D_v/A$ (1/s)</th>
<th>$T_{min}$ (hours)</th>
<th>$\lambda$</th>
<th>$M$</th>
<th>$q_h$ (m$^2$/s)</th>
<th>$q_L$ (m$^2$/s)</th>
<th>RMSE</th>
</tr>
</thead>
<tbody>
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<tr>
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<td>0.0</td>
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<td>0.589</td>
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</tr>
</tbody>
</table>

Figure 4-2 RWT concentration BTCs observed (circles) on April 7, 1971 in four sampling reaches in series along the Red River and simulated (lines) using the VART model for an instantaneous dye addition.
Bayous are commonly found in the south and southeastern United States and characterized by fine-grained substrates, low gradient, low flow velocity, high TDS (total dissolved solids) and turbidity. Bayou Bartholomew is the largest bayou in the world and home to majestic cypress trees and extensive amounts of wildlife and fish species, making it one of the most species-rich streams in North America. Substratum is predominantly silt and clay. The principal source of coarse substratum is large woody debris and gravel or cobble-sized particles from road and bridge construction. Bayou Bartholomew, flanked by wet bottomland forest, meanders through extensive croplands. An instantaneous RWT injection was performed on June 25, 1971 in the Bayou Bartholomew. Four sampling sites were established at Jones, Green Grove, Beekman, and Mouth, Louisiana, USA. The four sampling sites were located at 3.22 km, 25.75 km, 59.54 km, and 117.48 km downstream of the dye injection site, respectively. Flow discharges at the four sites were 4.1 m$^3$/s, 4.8 m$^3$/s, 6.5 m$^3$/s, and 8.1 m$^3$/s, respectively. Groundwater discharges into reaches 1–4 were 0, 3.14 × 10$^{-5}$ m$^3$/s-m, 5.03 × 10$^{-5}$ m$^3$/s-m, and 2.74 × 10$^{-5}$ m$^3$/s-m, respectively. No lateral inflows/outflows were reported. Flow depths at the four sites were 1.18 m, not available, 0.73 m, and 2.07 m, respectively. Recovery ratios of tracer at the four sites were 0.811, 0.842, 0.844, and 1.404, respectively. The flow velocity in each river reach is calculated using the distance from the injection site and the time to peak concentration at each sampling.

Table 4-2 Parameters values used in Figure 4-2 for the Red River.

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<th>Case</th>
<th>Reach</th>
<th>U (m/s)</th>
<th>K_s (m$^2$/s)</th>
<th>A_ad/A</th>
<th>D_f/A (1/s)</th>
<th>T_min (hours)</th>
<th>λ</th>
<th>M</th>
<th>q_l (m$^3$/s)</th>
<th>q_D (m$^3$/s)</th>
<th>RMSE</th>
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</thead>
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site. Other four parameters ($K_s$, $D_s/A$, $A_{adv}/A$, and $T_{\text{min}}$) are determined using the method presented by Deng et al. (2006). Estimated parameter values and RMSEs of the simulated RTDs shown in Figure 4-3 are listed in Table 4-3 for the two cases. The table indicates that the incorporation of solute losses in reaches 1–3 and the solute gain in reach 4 as well as hyporheic flow-induced water gains in reaches 2–4 into the VART model significantly reduces RMSEs (especially in the last reach) and thus improves the fitting of simulated RTDs to the observed BTCs.

![Graphs showing RWT concentration BTCs](image)

Figure 4-3 RWT concentration BTCs observed (circles) on June 25, 1971 in four sampling reaches in series along the Bayou Bartholomew and simulated (lines) using the VART model for an instantaneous dye addition.
Silty and clayey streams like the Bayou Bartholomew are generally excluded from studies on transient storage effect including hyporheic exchange that is commonly assumed to occur in sandy and gravel streams. Hulbert et al. (2002) studied micrographs of fine-grained sediments from Louisiana bayous and found that the sediment-water interface is characterized by great porosity and long (deep) pore-fluid pathways. It was also suggested that exchange between the pore fluid and the overlying water column would be relatively unhindered and the permeability of the undisturbed interface would be relatively high. However, little is actually known about the transient storage effect of solute in bayous.

Table 4-3 Parameter values used in Figure 4-3 for the Bayou Bartholomew.

<table>
<thead>
<tr>
<th>Case</th>
<th>Reach</th>
<th>U (m/s)</th>
<th>K_s (m²/s)</th>
<th>A_adv/A</th>
<th>D_i/A (l/s)</th>
<th>T_min (hours)</th>
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<th>M</th>
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</table>

Figures 4-1–4-3 clearly indicate that all the large rivers exhibit the same type of lognormal RTDs although the bed sediment (primarily silt and clay) in the Bayou Bartholomew is significantly different from those (primarily sand and gravel) in the Red and Mississippi Rivers. It means that the shape of the BTC in large rivers is not controlled by sediment properties and hyporheic exchange. Figures 4-1–4-3 and Tables 4-1–4-3 also demonstrate that the VART model is able to reproduce the BTCs observed in large rivers with a reasonable accuracy (RMSE). Simulated VART–L distributions fit observed tracer concentration BTCs very well for all stations in the three large rivers except the last station in the Mississippi River and the Bayou Bartholomew. It can be seen from Figure 4-3 and Table 4-3 that a significant mass gain (\( M = \)
1.404) occurred in the last reach of the Bayou Bartholomew, causing a significant underestimation of tracer concentration in the falling limb of the BTC. It is not clear what caused the abnormal mass gain. It appears that large rivers are capable of maintaining the same type of RTDs over a long distance.

4.4. Residence Time Distributions in Medium-Sized Rivers

In order to understand solute transport dynamics in moderate-sized rivers and the performance of VART model in such rivers, data of tracer experiments conducted in the Tickfaw River and Tangipahoa River, Louisiana (LA), USA, were gathered from the USGS report by Nordin and Sabol (1974). The two rivers are located in the Lake Pontchartrain River Basin.

The Tangipahoa River begins as an upland stream in Mississippi and flows southeastward for 127 km from the Mississippi-Louisiana state line through Tangipahoa Parish into Lake Pontchartrain. As it makes its way southward, it flows through rolling hills where it has a sand and gravel substrate. South of Highway 190, characteristics of the river change to those of a lowland stream where flat land levels off, substratum is silt and clay, and the water becomes sluggish, curved (meandering), and often muddy. A dye test in the Tangipahoa River was conducted on September 15, 1969. Four of seven sampling sites were located at 8.21 km (Kentwood, LA), 41.52 km (Amite, LA), 70.97 km (Natalbany, LA), and 93.98 km (Ponchatula, LA) downstream of the dye injection site, respectively. Flow discharges at the four sites were 3.5 m$^3$/s, 6.9 m$^3$/s, 8.6 m$^3$/s, and 10.8 m$^3$/s, respectively. Flow depths at the four sites were 0.49 m, not available, 0.46 m, and 0.76 m, respectively. Average flow depth in the reaches was 0.52 m. Recovery ratios of tracer at the four sites were 1.023, 0.802, 0.741, and 0.696, respectively. The remaining three sites are not included here because they were very close to the last three sites and were thus not representative. Lateral (tributary) inflow rates of reaches 1–4 were 0,
1.05 \times 10^{-4} \text{ m}^3/\text{s-m}, 0, \text{ and } 9.72 \times 10^{-5} \text{ m}^3/\text{s-m}, \text{ respectively. The discharge gain in stream reach 3 was attributed to the hyporheic flow-induced water gain at the rate of } 5.67 \times 10^{-5} \text{ m}^3/\text{s-m} \text{ because no tributaries joined the reach. Estimated parameter values and RMSEs of the RTDs shown in Figure 4-4 are listed in Table 4-4 for the two cases. Table 4-4 indicates that the incorporation of solute losses in reaches 2–4 and the solute gain in reach 1 as well as water gains in reaches 2 and 4 due to lateral inflows and in reach 3 due to hyporheic exchange into the VART model markedly reduces RMSEs (especially in reaches 2 and 4) and thus improves the fitting of simulated RTDs to observed BTCs.

The Tickfaw River originates in Southern Mississippi, USA and flows southeastward from the Mississippi-Louisiana state line through St. Helena and Livingston Parishes, Louisiana and eventually empties into Lake Maurepas. The scenic portion of the stream, approximately 110 kilometers long, flows southward through flat, alluvial bottomland with seepage (hyporheic flow) from ground water aquifers sustaining the stream flow. Substratum variation is similar to that of the Tangipahoa River. A dye test in the Tickfaw River was conducted on October 8, 1968 within the scenic portion. Four sampling sites were located at 6.44 km (Montpellier at Highway 16, LA), 22.53 km (Camp above Starns Bridge, LA), 38.62 km (Holden, LA), and 49.89 km (Springville, LA) downstream of the dye injection site, respectively. Flow discharges at the four sites were 2.0 m$^3$/s, 2.2 m$^3$/s, 1.9 m$^3$/s, and 2.9 m$^3$/s, respectively. Groundwater discharges into reaches 1–4 were 0, 1.23 \times 10^{-5} \text{ m}^3/\text{s-m}, -2.29 \times 10^{-5} \text{ m}^3/\text{s-m}, \text{ and } 9.3 \times 10^{-5} \text{ m}^3/\text{s-m}, \text{ respectively. No lateral inflows/outflows were reported. Flow depths at the four sites were 0.43 m, 0.80 m, 0.54 m, and 1.04 m, respectively. Average flow depth in the reaches was 0.70 m that was greater than that in the Tangipahoa River (0.52 m). Recovery ratios of tracer at the four sites were 0.829, 0.764, 0.560, and 0.781, respectively. Estimates of parameters and RMSEs of the
RTDs shown in Figure 4-5 are listed in Table 4-5 for the two cases. The table indicates that the incorporation of solute losses in the four reaches and hyporheic flow-induced water gains in reaches 2 and 4 as well as the water loss in reach 3 into the VART model significantly reduces RMSEs and thus improves the fitting of simulated RTDs to observed BTCs.

Figure 4-4 RWT concentration BTCs observed (circles) on September 15, 1969 in four sampling reaches in series along the Tangipahoa River and simulated (lines) using the VART model for an instantaneous dye addition.

<table>
<thead>
<tr>
<th>Case</th>
<th>Reach</th>
<th>U (m/s)</th>
<th>Ks (m²/s)</th>
<th>Aadv/ A</th>
<th>Dv/A (1/s)</th>
<th>Tmin (hours)</th>
<th>λ</th>
<th>M</th>
<th>qₘ (m²/s)</th>
<th>qₖ (m²/s)</th>
<th>RMS E</th>
</tr>
</thead>
<tbody>
<tr>
<td>O</td>
<td>1</td>
<td>0.165</td>
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<td>0.2</td>
<td>-4.29E-7</td>
<td>2.44</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>0.44</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.29</td>
<td>20</td>
<td>0.2</td>
<td>-2.43E-7</td>
<td>7.4</td>
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<tr>
<td></td>
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<td>8</td>
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<td>-1.46E-5</td>
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<td>1</td>
<td>0</td>
<td>0</td>
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<tr>
<td></td>
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<td>0.29</td>
<td>8</td>
<td>0.3</td>
<td>-1.72E-5</td>
<td>11.5</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>0.29</td>
</tr>
<tr>
<td>I</td>
<td>1</td>
<td>0.165</td>
<td>3</td>
<td>0.2</td>
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<td>2.44</td>
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<td>1</td>
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<td>0.44</td>
</tr>
<tr>
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<td>20</td>
<td>0.2</td>
<td>-2.43E-7</td>
<td>7.4</td>
<td>1</td>
<td>1</td>
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<td>8</td>
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<td>9.4</td>
<td>1.24</td>
<td>0.741</td>
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<tr>
<td></td>
<td>4</td>
<td>0.29</td>
<td>8</td>
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<td>-1.72E-5</td>
<td>11.5</td>
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<td>0.696</td>
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<td>9.72E-5</td>
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Figure 4-5 RWT concentration BTCs observed (circles) on October 8, 1968 in four sampling reaches in series along the Tickfaw River and VART-L RTDs (lines) simulated using the VART model for an instantaneous dye addition.

Figure 4-6 RWT concentration BTC observed (circles) on October 8, 1968 in the upper reach of the Tickfaw River and the simulated VART-P (power-law) and VART0E (exponential) distributions (lines).
Table 4-5 Parameter values used in Figure 4-5 and Figure 4-6 for the Tickfaw River.

<table>
<thead>
<tr>
<th>Case</th>
<th>Reach</th>
<th>U (m/s)</th>
<th>Ks (m²/s)</th>
<th>A_adv/A</th>
<th>Ds/A (1/s)</th>
<th>T_min (hours)</th>
<th>λ</th>
<th>M</th>
<th>Qh (m³/s)</th>
<th>QL (m³/s)</th>
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<td>1</td>
<td>0.0</td>
<td>0.0</td>
<td>0.32</td>
</tr>
<tr>
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<td>10</td>
<td>0.2</td>
<td>-5.36E-7</td>
<td>10.2</td>
<td>1</td>
<td>1</td>
<td>0.0</td>
<td>0.0</td>
<td>0.26</td>
</tr>
<tr>
<td>I</td>
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<td>0.1</td>
<td>-4.07E-7</td>
<td>3</td>
<td>1</td>
<td>0.829</td>
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<tr>
<td></td>
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<td>0.104</td>
<td>3</td>
<td>0.1</td>
<td>-9.17E-8</td>
<td>9.1</td>
<td>1.1</td>
<td>0.764</td>
<td>1.23E-5</td>
<td>0.0</td>
<td>0.23</td>
</tr>
<tr>
<td></td>
<td>3</td>
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<td>2</td>
<td>0.15</td>
<td>-1.45E-7</td>
<td>16.1</td>
<td>0.84</td>
<td>0.560</td>
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<td>1.56</td>
<td>0.781</td>
<td>9.3E-5</td>
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</tbody>
</table>

Figure 4-4, Figure 4-5 and Figure 4-6 show that moderate-sized rivers may display either lognormal or power-law RTDs. Most reaches of the Tickfaw River exhibit typical lognormal (VART-L) RTDs, as seen in Figure 4-5. The first (upper) reach of the river can also be fitted using a power-law (VART-P) distribution with RMSE = 0.37, as shown in Figure 4-6. Likewise, the four reaches of the Tangipahoa River may also be fitted using power-law (VART-P) RTDs although the simulated RTDs shown in Figure 4-4 and Table 4-4 belong to VART-L distributions. In fact, the BTCs of the Tangipahoa River (with a mean water depth of 0.52 m) are closer to power-law distributions than those of the Tickfaw River (with a mean water depth of 0.70 m).

Residence time distributions in small streams were analyzed and reported by Deng and Jung (2009). The study showed that small streams may exhibit a wide variety of RTD types from upward curving patterns to a straight line (power-law distributions) and further to downward curving lognormal distributions when plotted in log-log coordinates.

4.5. Discussions

A comparison among the parameter values listed in Tables 4-1–4-5 show that the three VART parameters $A_{adv}/A$, $Ds/A$, and $T_{min}$ commonly vary in the ranges of 0.1–0.5, $1.0 \times 10^{-7}$ – $9.0 \times 10^{-7}$ s⁻¹, and 0.3–16.0 hours, respectively, in large and moderate-sized rivers and also in
small streams (Deng and Jung 2009). It appears that the parameter values/ranges are independent of channel size because there are no significant variation trends in the values of the three parameters that govern solute exchange between surface stream water and subsurface sediment pore water. Among the four VART parameters $K_S$, $A_{adv}/A$, $D_S/A$, and $T_{min}$, the parameter that is most sensitive to changes in channel size is the longitudinal dispersion coefficient $K_S$. The parameter $K_S$ varies over several orders of magnitude in the ranges of $0.1–1.0 \text{ m}^2/\text{s}$ in small streams (Deng and Jung 2009), $1.0–10.0 \text{ m}^2/\text{s}$ in moderate-sized rivers, and $10.0–250.0 \text{ m}^2/\text{s}$ in large rivers, as shown in Tables 4-1–4-5 and in Deng et al. (2001). The formula for estimation of the longitudinal dispersion coefficient presented by Deng et al. (2001) shows that parameter $K_S$ is proportional to flow depth and $U^2$. Obviously, river size significantly affects longitudinal dispersion coefficient $K_S$. In order to better understand the effect of the variation in parameter $K_S$ on RTDs, two extreme but commonly-used scenarios are discussed as follows.

In the first scenario it is assumed that the transient storage term including lateral inflows/outflows is negligible as compared to the longitudinal dispersion term in Eq. (4.11). This
is a fundamental assumption made in the classical advection-dispersion theory for streams (Deng et al. 2001; Fischer et al. 1979). Then, Eqs. (4.11), (4.2)–(4.5) reduce to the conventional advection and dispersion equation that has the following analytical solution for an instantaneous slug injection of tracer:

$$C = \frac{W}{A\sqrt{4\pi K_s t}} \exp\left(-\frac{(x - Ut)^2}{4K_s t}\right)$$  \hspace{1cm} (4.12)

where $W$ is the mass of tracer injected. Figure 4-7 shows the solute concentration BTCs obtained from Eq. (4.12) by fixing values of parameters $W$, $A$, $x$, and $U$ and changing the dispersion coefficient $K_S$. The BTCs shown in Figure 4-7 are typical lognormal distributions. It means that a lognormal BTC can be generated by the conventional advection and dispersion processes in streams without transient storage zones. This scenario may occur in large rivers where the longitudinal dispersion coefficient becomes large and advection and dispersion process dominates solute transport.

In the second scenario it is assumed that the dispersion term is negligible as compared to the transient storage term in Eq. (4.11). This assumption is often used for small streams (Schmid
2003; Scott et al. 2003). Figure 4-8 shows the five types of VART series RTDs proposed by Deng and Jung (2009) and simulated using Eq. (4.11) without the longitudinal dispersion term ($K_S = 0$) by changing the parameter $D_S/A$ and fixing other parameters. The VART series distributions, when plotted in log-log coordinates, switch from the upward curving VART+U and VART0U to a straight line (power-law: VART–P) and further to the downward curving VART–L (lognormal) distributions when the parameter $D_S/A$ decreases from positive to zero and further to negative values, as shown in Table 4-6 and Figure 4-8. The VART+U RTDs commonly occur in small streams (Deng and Jung 2009). Figure 4-8 clearly indicates that the transient storage term in the VART model, namely the way in which transient storage is parameterized in the model, can generate different tail behaviors of the RTDs due to the advection and effective diffusion (differential advection) processes in the two types of hyporheic zone: (1) an advection-dominated upper transient storage zone and (2) an effective diffusion-dominated lower transient storage zone. It means that in theory (VART model) small streams with a negligible longitudinal dispersion coefficient are capable to produce various RTDs due to the hyporheic exchange. It should be noted that the lognormal, power-law, and exponential RTDs are not specific to the VART model. Instead, these RTDs can also be generated as output of other models such as STAMMT-L (Gooseff et al. 2003) or CTRW (Boano et al. 2007).

Table 4-6 Parameter values used in the VART series distributions shown in Figure 4-8.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>$U$ (m/sec)</th>
<th>$K_S$ (m$^2$/s)</th>
<th>$A_{adv}/A$</th>
<th>$D_S/A$ (1/s)</th>
<th>$T_{min}$ (hrs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>VART+U</td>
<td>0.37</td>
<td>0.0</td>
<td>0.12</td>
<td>+2.46E-8</td>
<td>4</td>
</tr>
<tr>
<td>VART0U</td>
<td>0.37</td>
<td>0.0</td>
<td>0.12</td>
<td>0</td>
<td>4</td>
</tr>
<tr>
<td>VART0E</td>
<td>0.37</td>
<td>0.0</td>
<td>0.12</td>
<td>0</td>
<td>4</td>
</tr>
<tr>
<td>VART-P</td>
<td>0.37</td>
<td>0.0</td>
<td>0.12</td>
<td>-4.91E-8</td>
<td>4</td>
</tr>
<tr>
<td>VART-L</td>
<td>0.37</td>
<td>0.0</td>
<td>0.12</td>
<td>-3.69E-7</td>
<td>4</td>
</tr>
</tbody>
</table>

The simulation results show that solute RTDs are dispersion-dependent and thereby scale-dependent. RTDs in large rivers are controlled primarily by instream advection-dispersion
processes and thus exhibit lognormal (VART−L) distributions. The influence of hyporheic exchange on RTDs increases with decreasing channel size and thus decreasing longitudinal dispersion coefficient. Small streams with negligible dispersion coefficient may display various types of the VART series distributions, as shown in Figure 4-8. Moderate-sized rivers are transitional in terms of RTDs. Thus, moderate-sized rivers commonly exhibit lognormal (VART−L) and power-law (VART−P) RTDs, as shown in Figure 4-4 and Figure 4-5. More investigations are definitely needed to provide quantitative criteria for describing the transition from one type of VART series RTDs to another one. Anyway, there is no single scale-independent type of RTDs that can be universally applied to all rivers and streams. In that sense, RTDs are channel size or spatial scale dependent. Eqs. (4.11), (4.2)–(4.5) clearly shows that VART model is also a temporal scale-dependent model (Deng and Jung 2008) because an actual variable residence time is employed in Eqs. (4.11) and (4.2).

Tables 4-1–4-5 and Figures 4-1–4-6 demonstrate that losses/gains of water and solute are an important factor affecting RTDs. The incorporation of losses/gains of water and solute in the VART model can make parameter values more meaningful, improving simulation results. It is interesting to find that the ratio of the time to peak concentration ($T_{peak}$) to the minimum mean residence time ($T_{min}$), $T_{peak}/T_{min}$, is very close to the recovery ratio of tracer, $M_{rec}/M_{inj}$, i.e., $T_{peak}/T_{min} \approx M_{rec}/M_{inj}$, where $M_{rec}$ is the tracer mass recovered at the end of a tracer experiment and $M_{inj}$ is the mass injected.

Figure 4-9 includes both measured data and simulated results of tested river reaches including the small streams simulated by Deng and Jung (2009). The field data of $T_{peak}$ and recovery ratios ($M_{rec}/M_{inj}$) are taken from the USGS report by Nordin and Sabol (1974). The
The minimum residence time \( T_{min} \) shown in Figure 4-9 is estimated using the fractional Laplace transform-based parameter estimation method proposed by Deng et al. (2006). Figure 4-9 illustrates that all small and moderate-sized rivers have an almost perfect correlation between the recovery ratio, \( M_{rec}/M_{inj} \), and the ratio of the time to peak to the minimum residence time, \( T_{peak}/T_{min} \). It means that the ratio, \( T_{peak}/T_{min} \), represents mass loss (\( T_{peak}/T_{min} < 1 \)) or gain (\( T_{peak}/T_{min} > 1 \)) or balance (\( T_{min}/T_{peak} = 1 \)) during solute transport in a stream. The significance of this finding is that the minimum mean residence time \( T_{min} \) involved in the VART model can be simply calculated using the relation \( T_{peak}/T_{min} = M_{rec}/M_{inj} \) if the data of \( T_{peak} \) and \( M_{rec}/M_{inj} \) are available, providing a simple method for determination of the parameter \( T_{min} \). Actually, the finally adopted \( T_{min} \) values, shown in Figures 4-2, 4-3 (first two reaches), 4-4, 4-5 and 4-6, and in Deng and Jung (2009) were determined using this method as the \( T_{min} \) values initially estimated using the optimization procedure (Deng et al. 2006) are very close to those determined from the relation \( T_{peak}/T_{min} = M_{rec}/M_{inj} \). The \( T_{min} \) values shown in Figure 4-9 are determined using the
optimization procedure. The values of parameter $T_{\text{min}}$ are determined using the relation $T_{\text{peak}}/T_{\text{min}} = M_{\text{rec}}/M_{\text{inj}}$ for both the case O and the case I in Tables 4-1–4-5. Therefore, $T_{\text{min}}$ values do not change when solute gains or losses are included. Consequently, the VART model contains only three fitting parameters $A_{\text{adv}}/A$, $D_s/A$, and $K_S$ for small and moderate-sized streams and some large rivers like the Red River, greatly simplifying the application of VART model. It is not very clear why the five reaches of two large rivers do not follow the relation $T_{\text{peak}}/T_{\text{min}} = M_{\text{rec}}/M_{\text{inj}}$. A possible explanation for the deviation is that the solute losses/gains in the five reaches were not caused by the hyporheic exchange (controlling the relation $T_{\text{peak}}/T_{\text{min}} = M_{\text{rec}}/M_{\text{inj}}$). In fact, $q_h = 0$ in the four reaches of the Mississippi River. It appears from the Mississippi River case that the relation $T_{\text{peak}}/T_{\text{min}} = M_{\text{rec}}/M_{\text{inj}}$ may also be utilized to determine the contribution of hyporheic exchange to the attenuation of nutrients and contaminants in streams. More field observation efforts are definitely needed to confirm the findings. It should be pointed out that mass losses and gains generally occur in rivers. Solute losses or gains may reduce or increase peak concentration, time to the peak concentration, and the minimum mean residence time (Nordin and Sabol 1974; Runkel 2002; Scott et al. 2003) and thereby affect RTDs.

4.6. Conclusions

The major findings can be summarized as follows:

(1) The VART model is able to reproduce essentially any type of solute BTCs observed in rivers and streams with 3–4 fitting parameters while no prescribed RTD functions are needed.

(2) Instream advection and dispersion processes can produce only lognormal RTDs.

(3) The hyporheic exchange process described by the VART model without the dispersion and lateral inflow/outflow terms is able to generate a wide variety of RTD types from the upward curving VART+U to a straight line (VART−P) and further to the downward curving
VART–L distributions when plotted in log-log coordinates.

(4) RTDs depend on both temporal and spatial scales. In terms of temporal scale, RTDs depend on an actual variable residence time. In terms of spatial scale, RTDs are channel-size dependent. Stream channel size affects the pattern of RTDs of solute through changing the relative contributions of the dispersion and hyporheic exchange terms to solute transport described in the VART model. Large rivers are dominated by instream advection and dispersion processes due to large longitudinal dispersion coefficient and tend to exhibit lognormal (VART–L) distributions. The influence of hyporheic exchange on RTDs increases with decreasing channel size. RTDs in small streams are affected more significantly by hyporheic exchange. Therefore, small streams may display various types of the VART series distributions. Moderate-sized rivers are transitional in terms of RTDs and can exhibit both lognormal and power-law RTDs. There is no single scale-independent type of RTDs that can be generally applied to any rivers and streams.

(5) The effect of water and solute losses/gains on RTDs is significant. Mathematically, the incorporation of water and solute losses/gains into the VART model is to maintain the mass balance of tracer and improve simulation results, making parameter values more reasonable. Physically, solute losses or gains can change peak concentration, time to the peak concentration, and the minimum mean residence time and thereby affect RTDs.

(6) The ratio of the time to peak to the minimum mean residence time in the VART model is equal to the recovery ratio of tracer. The relation provides a simple method for determination of the VART parameter $T_{\text{min}}$. Consequently, the VART model contains only three fitting parameters ($A_{\text{adv}}/A, D_S/A$, and $K_S$) for small and moderate-sized streams and some large rivers, greatly simplifying and enhancing the application of VART model.
4.7. References


CHAPTER 5. HYDROGRAPH-BASED APPROACH TO MODELING BACTERIAL FATE AND TRANSPORT IN RIVERS

5.1. Introduction

Bacterial concentrations in rivers have been observed to be often much higher during storm events than during low flows. In fact, storm events export major part of the annual load of *Escherichia coli* (*E. coli*, fecal indicator bacteria) reaching as high as 98% (Chu et al. 2011; McKergow and Davies-Colley 2010) in some rivers. Wilkinson et al. (1995) observed an increase in fecal coliform concentrations by 25 times during an artificial flood. Peak concentrations of bacteria have been found to occur usually during rising limb of a storm hydrograph (Davies-Colley et al. 1994; Jamieson et al. 2005b) well ahead of the discharge peak and close to the line of maximum flow acceleration (McKergow and Davies-Colley 2010; Nagels et al. 2002). Artificial flood experiments without any watershed input of bacterial loads also showed a significant increase in *E. coli* levels during rising hydrographs (Muirhead et al. 2004), indicating that riverbed sediment may serve as an important bacterial source in streams. It is often found that during flows with high bed shear stress the entrainment of fecal bacteria takes place (Jamieson et al. 2005b) and the release of bacteria takes place when the bed stress reaches a certain critical value. As fecal coliforms are often concentrated near the sediment-water interface (SWI) and are mostly associated with fine particulates of low settling velocity (Wilkinson et al. 1995), accurate assessment of the entrainment of fine sediment from the channel bed is important to modeling bacterial transport, especially during high flow events.

Various numerical models have been developed to simulate bacterial transport and fate in rivers by considering the sediment and water column interaction. Jamieson et al. (2005a) studied the controlling processes for fate and transport of enteric bacteria in alluvial streams by combining field experiments and mathematical modeling. A strain of *E. coli* was mixed with
stream water and bed sediment, and loaded in streams to monitor the transport of sediment and *E. coli* at downstream locations. As the experiment was carried out during low and steady flow, no entrainment was included in their model. Bai and Lung (2005) added fecal bacterial transport component to the Environmental Fluid Dynamics Code model to study the impact of sediment transport processes on fecal transport in rivers. The flux of fecal bacteria was linked with sediment dynamics across SWI. Hipsey et al. (2006) developed a model within an aquatic ecology model Computational Aquatic Ecosystem Dynamics Model (CAEDYM) to include sedimentation and resuspension processes in addition to other processes such as growth, mortality and predation. Although more generic, this model is too complex for short term simulations such as for storm events. Rehmann and Soupir (2009) quantified the effect of interaction between sediment and water column for microbial concentration using one dimensional steady state model of transport in a river. Transport equations were derived for depth averaged microbial concentrations in the water column and sediment separately, and solved. The longitudinal dispersion process was ignored in their model. Cho et al. (2010a; 2010b) followed the approach by Steets and Holden (2003) for bacterial transport by incorporating the resuspension and sedimentation terms into a net resuspension term in their models. Both used a simple formula for bed shear stress calculated based on flow velocity using a constant friction coefficient. Unlike the other two, Cho et al. (2010b) did not use sediment storage model but determined the bacterial concentration in bed sediment from model calibration. Recently, Gao et al. (2011) developed a numerical model based on DIVAST (Depth Integrated Velocities and Solute Transport Model) with a focus on predicting the impact of sediment fluxes on fecal bacteria levels in water column. The model was applied to several idealized case studies and also to an artificial flood study. Finally, Wilkinson et al. (2011)
modeled *E. coli* pulses in Motueka River, New Zealand, using records of *E. coli* concentration during several storm events in 2003–2004. Their model domain consists of main river reach and sub-catchments with three layers: riparian land, river reach water column and river reach channel storage. The model includes sediment resuspension and deposition processes along with a bacterial die-off term but does not use advection-dispersion equation and is very much site specific.

Despite efforts to include all processes in bacterial transport modeling, the transient storage effect was mostly ignored. It is well observed that natural streams possess permeable banks and bed sediment which create transient storage zones and thereby generate significant mass exchange between surface and subsurface waters due to the hyporheic exchange (Deng and Jung 2009). Grant et al. (2011) measured flux of fecal bacteria across the SWI in a small effluent stream with a turbulent flow and found that the hyporheic exchange controls the transport of the bacteria across the SWI in turbulent streams. By combining duel tracer test results and the transient storage model (Runkel 1998), Shen et al. (2008) showed that a bacteriophage P22 can be successfully used as a tracer in complex surface water environments. When the concentration of free *E. coli* is high in the water column during low flows, the mass exchange between storage zones and the main channel is substantial and thereby ignoring the transient storage may produce significant errors.

The primary objective of this study is to present an alternative approach to modeling bacterial fate and transport in natural streams. The new modeling approach should be applicable to both low flow and high flow (especially flood flow) conditions. To that end, the variable residence time based (VART) model (Deng and Jung 2009) is extended in this study to simulate bacterial fate and transport by taking into account: i) unsteadiness of flow using a hydrograph-
based approach, ii) effect of sediment erosion/deposition on bacterial concentrations, and iii) bacterial growth/decay processes in addition to advection, dispersion, and hyporheic exchange processes included in the original VART model. The extended VART model is applied to simulate bacterial transport in natural streams under different sediment and flow conditions, ranging from steady low flow without sediment transport to flood events with significant sediment transport due to watershed inputs and sediment resuspension from stream bed.

5.2. Model Development

5.2.1. Conceptual Model

Major processes controlling the fate and transport of bacteria in streams include advection, dispersion, transient storage (including hyporheic exchange), inactivation, and resuspension/settling of attached fraction. While the advection and dispersion processes are generally included in mass transport models, other processes are selectively included. Two contrasting flow conditions controlling bacterial fate and transport are often encountered in natural streams: 1) low flow- when a stream has low flow discharge, shallow depth, clearer water, higher residence time, and clear weather with sunshine, and 2) high flow- when a stream has high flow discharge, deep and turbid water, lower residence time and generally cloudy weather with less sunshine. Accordingly, during the low flow there is a likelihood of higher inactivation rates due to longer residence time, clearer water and more sunlight. The exchange due to transient storage may also play an important role during low flow as the flow in the main channel is relatively small and slow. On the other hand, resuspension of sediment associated bacteria from the streambed, particularly during rising flows, and subsequent deposition during receding flows may play a dominant role for storm events. Due to shorter residence time and favorable environment for survival of bacteria in water column, the solar inactivation plays less
important role during such storm events. The transient storage effect is insignificant during storm
events due to the sediment resuspension-induced destruction of the storage zones in the upper
river bed and banks.

![Diagram of key processes in VARTBacT model]

Figure 5-1 Conceptual diagram of key processes in VARTBacT model.

The simulation of bacterial transport during both low flow and high flow events requires the
incorporation of sediment resuspension/deposition processes and bacterial growth/decay into the
VART model. The processes represented in this model are depicted in Figure 5-1.

5.2.2. Hydrograph-Based Approach to Modeling Sediment Resuspension

Flow parameters, including friction slope, flow velocity, and bed shear stress, are
determined using a hydrograph-based method (Ghimire and Deng 2011). The method requires
the use of flow hydrograph as basic input data for computation of the discharge gradient \(\frac{\partial Q}{\partial t}\)
term which is again used to determine bed shear stress for unsteady flow. More specifically, the
bed shear stress, which is considered to be the most important driving factor for sediment
entrainment, is determined as

\[
\tau_b = \rho g R_h \left( S_o + \alpha \frac{\partial Q}{\partial t} \right)
\]  

(5.1)
where \( \tau_b \) = shear stress exerted by flow on the bed \((\text{N/m}^2)\), \( S_o \) = channel bed slope \((-)\), \( R_h \) = hydraulic radius \((\text{m})\) which may be replaced with water depth \( h \) \((\text{m})\) in natural rivers, \( Q \) = flow discharge \((\text{m}^3/\text{s})\), \( t \) = time \((\text{s})\), and \( \alpha \) is a parameter relating the slope of the flood hydrograph with \( \tau_b \) \((\text{m}^{-3}\text{s}^2)\). In case of natural flood events \( \alpha = 1/(BC^2) \), where \( B \) = channel width \((\text{m})\) and \( C \) = average wave celerity \((\text{m/s})\). The parameter \( C \) is normally determined using friction law: \( C = \beta U \), where \( \beta \) is kinematic wave parameter. For a wide rectangular channel \((B >> h)\), Manning’s law gives the maximum value of \( \beta = 1.67 \). Flow parameters, such as flow velocity \( U \) \((\text{m/s})\) and friction slope \( S_f \)(-), were determined following Ghimire and Deng (2011).

The sediment transport during unsteady flows can be calculated using the hydrograph-based approach described in Chapter 3. For non-cohesive sediment, Ackers and White (1973) formula is used. Unlike formulas giving entrainment rates near sediment water interface directly (e.g., Smith and McLean 1977), Ackers and White (1973) formula gives the depth averaged equilibrium sediment concentration \( S_e \) \((\text{kg/m}^3)\). Therefore, the sediment concentration at the reference level \((z = 0.05 h)\) was determined later using Abad and García (2006) formulation for Einstein Integral \( INT_1 \).

The depth averaged net sediment flux \( F_r \) can be calculated following the sediment carrying capacity approach (Gao et al. 2011; RuiJie et al. 2009):

\[
F_r = E - D = \frac{w_s (S_e - \theta S)}{INT_1}
\]

where \( E \) = sediment erosion rate \((\text{kg/m}^2\text{s})\); \( D \) = deposition rate \((\text{kg/m}^2\text{s})\); \( w_s \) = particle settling velocity \((\text{m/s})\); \( S \) = depth-averaged sediment concentration \((\text{kg/m}^3)\); \( S_e \) = equilibrium sediment concentration \((\text{kg/m}^3)\); and \( \theta = S_e/S \) is a sediment concentration profile factor (Gao et al. 2011), \( \theta = 1 \) at the equilibrium sediment transport state and \( \theta \neq 1 \) in a non-equilibrium state. The net
vertical displacement of SWI, $v_{swi}$ is then computed following the approach by Motta et al. (2010) as

$$v_{swi} = \frac{F_r}{(1-p)\rho_s}$$ (5.3)

where $v_{swi} =$ velocity of SWI (m/s), $p =$ porosity of bed sediment (-), and $\rho_s =$ specific density of sediment grains. The settling velocity may be obtained using a relation from Wu and Wang (2006) for naturally worn particles with a Corey shape factor (CSF) of 0.7,

$$w_s = \frac{M'\nu}{N'd} \left( \frac{1}{4} + \left( \frac{4N' \sqrt{3M'^2} D^2}{3M'^2 D^2} \right)^{1/n'} - \frac{1}{2} \right)^{n'}$$ (5.4)

where the coefficients $M' = 33.94$, $N' = 0.98$, and $n' = 1.33$. The kinematic viscosity $\nu$ was calculated as $1.79 \times 10^{-6} / (1 + 0.3368 T + 0.00021 T^2)$ m$^2$/s, where $T$ is in degree Celsius (García 2008). The porosity of the sediment was also estimated as per Wu and Wang (2006).

In case of cohesive sediment transport, the net upward resuspension flux may be written as

$$F_r = \begin{cases} 
E = M \left( \frac{\tau_b}{\tau_{cr,e}} - 1 \right), & \tau_b \geq \tau_{cr,e} \\
0, & \tau_{cr,e} > \tau_b > \tau_{cr,d} \\
-D = -w_s S \left( 1 - \frac{\tau_b}{\tau_{cr,d}} \right), & \tau_b \leq \tau_{cr,d} 
\end{cases}$$ (5.5)

where $M =$ empirical erosion rate constant (kg/m$^2$/s), $\tau_{cr,e}$ $\tau_{cr,d}$ = critical bed shear stress for erosion (Pa), and $\tau_{cr,d}$ $\tau_{cr,e}$ = critical bed shear stress for deposition (Pa). Settling velocity $w_s$ was determined using Eq. (5.4) for CSF = 0.4. The net vertical displacement of SWI can be obtained by dividing $F_r$ by the dry density of the bed sediment, as shown in Eq. (5.3). Herein commonly used Ariathurai-Partheniad relation for surface erosion of cohesive sediment (Ariathurai 1974) and the sediment deposition formula by Krone (1962) were used. A linear erosion formulation with a constant critical shear stress was adopted for simplicity. A more realistic formulation for
natural rivers would have a depth-varying critical stress either determined experimentally or indirectly using measured dry density of bed sediment (Sanford and Maa 2001). In rivers with fine-grained sediment bed, for example, the exponential function given by Parchure and Mehta (1985) with depth-varying critical stress may be used to model the erosion of a soft top layer. No matter which formula is chosen for modeling, accurate determination of site-specific sediment parameters, such as critical shear stress, is very important (Lick 2008; Sanford and Maa 2001).

### 5.2.3. Variable Residence Time Based Bacterial Transport (VARTBacT) Model

The variable residence time (VART) model (Deng and Jung 2009) was originally developed for longitudinal dispersion and transport of solutes in natural streams. This model is based on the concept of varying residence time and is able to produce various types of breakthrough curves commonly observed in natural rivers due to the mass flux across the sediment-water interface (Deng and Jung 2009; Deng et al. 2010).

In order to incorporate the effect of sediment on the bacterial fate and transport, the original VART model is modified by adding appropriate terms for sediment resuspension and deposition and the bacterial die-off/growth in water column and storage zones. The modified VART model for bacterial fate and transport, herein after called VARTBacT model, is given by the following equations:

\[
\frac{\partial C}{\partial t} + U \frac{\partial C}{\partial x} = K_s \frac{\partial^2 C}{\partial x^2} + \frac{A_s}{A T_v} \left( C_s - C \right) + \frac{E}{h} C_b - \frac{D \rho f_p C}{h S} - kC \tag{5.6}
\]

\[
\frac{\partial C_s}{\partial t} = \frac{1}{T_v} \left( C - C_s \right) - k_s C_s \tag{5.7}
\]

\[
T_v = \begin{cases} 
T_{min} & \text{for } t \leq T_{min} \\
T_{min} & \text{for } t \geq T_{min} \quad (T_{min} > 0)
\end{cases} \tag{5.8}
\]
\[
\begin{align*}
t_s = \begin{cases} 
0 & \text{for } t \leq T_{\text{min}} \\
(t - T_{\text{min}}) & \text{for } t \geq T_{\text{min}}
\end{cases}
\end{align*}
\]

(5.9)

where \( A_s = A_{\text{adv}} + A_{\text{dif}} \), \( A_{\text{dif}} = 4\pi D_s t_s \); \( C, C_S \) = bacterial concentration in water column and transient storage zones (CFU/100mL), respectively; \( C_b \) = bacterial concentration in the sediment bed (CFU/0.1g); \( U = \) cross-sectionally averaged flow velocity (m/s); \( h = \) depth of water column (m); \( A, A_S = \) cross-sectional flow areas of main channel and transient storage zones (m\(^2\)), respectively; \( t = \) time (s); \( T_r = \) actual varying residence time of bacteria in storage zones (s); \( A_{\text{adv}}, A_{\text{dif}} = \) areas of advection-dominated transient storage zone and effective diffusion-dominated transient storage zone (m\(^2\)), respectively; \( K_S = \) longitudinal Fickian dispersion coefficient excluding the transient storage effect (m\(^2\)/s); \( D_S = \) effective diffusion coefficient in hyporheic zone (m\(^2\)/s); \( t_s = \) time since the release of bacteria from storage zones to the main stream (s); \( T_{\text{min}} = \) minimum mean residence time for bacteria to travel through the advection-dominated storage zone (s); \( k, k_s = \) bacterial decay/growth rates in the water column and storage zones (1/day), respectively; \( f_p \) = fraction of bacteria attached to suspended sediments (-); and \( f_p \times C \) = volume-specific concentration on the particles (CFU/100 ml). The estimated value of \( f_p \) in the literature is highly variable, ranging from 0.1 (Liu et al. 2006) for Lake Michigan to 0.9 (Steets and Holden 2003) in a coastal lagoon. In this study the approach by Gao et al. (2011) was followed by using a linear partition coefficient assuming instantaneous equilibrium:

\[
f_p = \frac{K_d S}{1 + K_d S}
\]

(5.10)

where \( K_d = \) linear partition coefficient (m\(^3\)/kg). This approach allows us to relate the attached fraction of bacteria to vary depending upon \( S \). The third and fourth terms on the right side of Eq. (5.6) represent bacterial source/sink terms due to sediment resuspension and deposition. The parameters \( E \) and \( D \) represent erosion and deposition rates (kg/m\(^2\)/s) which are calculated
according to Eqs. (5.2) and (5.5) for non-cohesive and cohesive sediments, respectively. In contrast with most of the bacterial transport models which account for sediment-water interaction (e.g., Bai and Lung 2005; Gao et al. 2011; Rehmann and Soupir 2009), this model considers the interaction with the transient storage, reflecting the exchange between surface stream water and subsurface sediment pore water. As there is no separate equation for the concentration of bacteria in the sediment bed, the concentration $C_b$ has to be estimated for computing the resuspension flux of bacteria into the water column. A split-operator method used in the numerical solution of the VARTBacT model is provided in section 5.6.

5.3. Model Applications

To test and demonstrate the use of the model for bacterial fate and transport in natural streams, three case studies with differing flow and sediment conditions were considered. First, the model was applied to a river with low flow, where a duel tracer study was conducted using bacteriophage and a conservative tracer. No sediment resuspension and deposition processes were included in the first case as the tracer test was carried out during a low and relatively steady flow. The second case involved field experiments on resuspension of the bed sediment seeded with bacteria from the upstream boundary of the study reach during artificial storm events. The third case study focused on simulating bacterial transport in a river during natural storm/flood events, characterized with significant sediment transport from the watershed and sediment resuspension from stream bed. Although actual flow and concentration data were used, some important assumptions were made to test the performance of the model during high flows.
5.3.1. Bacterial Transport in Rivers with Low Flow

Shen et al. (2008) carried out a duel tracer study to evaluate the performance of bacteriophage P22 relative to a conservative tracer Rhodamine WT on a 40 km reach of Grand River, MI, USA. The test was done on May 8, 2006 during a low flow period near the end of the recession limb of hydrograph. To accommodate the lateral variability in tracer concentration, the sampling was conducted for both tracers simultaneously at multiple locations including 4.56, 13.69, and 28.38 km downstream of the tracer release station. The study used the conventional transient storage model (Runkel 1998) to describe the solute transport.

The data from the above-mentioned study were first used to test the performance of the VART model, which is the basic model of VARTBacT, for simulating P22 as well as RWT tracer breakthrough curves. With lateral inflow and reaction terms, the VARTBacT model equation (5.6) can be expressed as

\[
\frac{\partial C}{\partial t} + U \frac{\partial C}{\partial x} = K_s \frac{\partial^2 C}{\partial x^2} + \frac{A_s \lambda}{A \bar{T}_v} (C_s - C) + \frac{q_L}{A} (C_L - C) - kC
\]  

(5.11)

where, \(C_L\) = solute concentrations in lateral inflow, \(q_L\) = surface lateral inflow rates per unit channel length. All other symbols have been defined earlier.

The primary flow and channel geometry parameters were taken from Shen et al. (2008) and others were either calculated or estimated. The flow velocity at each site was determined using the distance from the injection site and the time to peak concentration at each sampling site. Lateral flows were included in reaches 2 and 3.

For RWT tracer, Eqs. (5.7)–(5.9), and (5.11) were solved on a reach basis. Decay rates \(k\) and \(k_s\) were set to zero. Four VART model parameters \(K_s, A_{adv}/A, D_s/A,\) and \(T_{min}\) were determined based on best fit with the observed data. The parameter values listed in Table 5-1 were also used to simulate P22 tracer breakthrough curve.
Table 5-1 Estimated common parameters for RWT and P22 in the VART model

<table>
<thead>
<tr>
<th>Reach</th>
<th>$U$ (m/s)</th>
<th>$K_s$ (m$^2$/s)</th>
<th>$A_{adv}/A$</th>
<th>$D_s/A$ (1/s)</th>
<th>$q_L$ (m$^2$/s)</th>
<th>$T_{min}$ (hours)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.43</td>
<td>0.02</td>
<td>0.03</td>
<td>-1.5E-7</td>
<td>0.0</td>
<td>0.23</td>
</tr>
<tr>
<td>2</td>
<td>0.59</td>
<td>1.00</td>
<td>0.27</td>
<td>-7.8E-7</td>
<td>1.3E-4</td>
<td>0.75</td>
</tr>
<tr>
<td>3</td>
<td>0.55</td>
<td>10.00</td>
<td>0.16</td>
<td>-9.3E-8</td>
<td>2.2E-4</td>
<td>1.10</td>
</tr>
</tbody>
</table>

The estimated inactivation rates in the main channel $k$ for P22, and recovery ratios $M^*$ for both tracers along with RMSE values are given in Table 5-2.

Figure 5-2 Comparison between observed and simulated Rodamine WT tracer concentrations at three sampling stations in the Grand River: (a) 4.56, (b) 13.69, and (c) 28.38 km downstream of tracer release. Right, center and left are the sampling locations and Q-wt refers to the flow-weighted concentration at each sampling site.
The comparison between observed and simulated concentrations of RWT and P22 are given in Figure 5-2 and Figure 5-3, respectively.
Figure 5-3 Comparison between observed and simulated bacteriophage P22 concentrations at three sampling stations in the Grand River: (a) 4.56, (b) 13.69, and (c) 28.38 km downstream of tracer release site. Right, center and left are the sampling locations and Q-wt refers to the flow-weighted concentration at each sampling site.
Table 5-2 Estimated tracer specific parameters and RMSE values

<table>
<thead>
<tr>
<th>Tracer type</th>
<th>Reach</th>
<th>(k) (1/s)</th>
<th>(M^*)</th>
<th>RMSE</th>
</tr>
</thead>
<tbody>
<tr>
<td>RWT</td>
<td>1</td>
<td>0.0</td>
<td>1.14</td>
<td>0.46</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.0</td>
<td>1.08</td>
<td>0.19</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>0.0</td>
<td>0.64</td>
<td>0.13</td>
</tr>
<tr>
<td>P22</td>
<td>1</td>
<td>0.0</td>
<td>0.31</td>
<td>1.06</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>2.0E-5</td>
<td>0.27</td>
<td>0.51</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>1.0E-5</td>
<td>0.18</td>
<td>0.33</td>
</tr>
</tbody>
</table>

The results show that the VART model reproduces breakthrough curves for the bacteriophage tracer reasonably well. Slightly higher RMSE was probably due to the use of same parameter values for the conservative tracer RWT and reactive tracer P22. Shen et al. (2008) used different parameter values for two tracers and assumed that the different processes might have contributed to transient storage within the same reach for two tracers, demonstrating the advantage of the VART model.
5.3.2. Sediment Resuspension-Induced Bacterial Transport During High Flows

The numerical model VARTBacT was applied to simulate the resuspension of *E. coli* in Swan Creek, Ontario, Canada using field observations. Jamieson et al. (2005b) seeded a part of streambed with a strain of *E. coli* NAR and monitored the survival and transport of the tracer bacteria for a two-month period covering three storm events. The flow, suspended sediment concentration and bacterial concentration in water column at 10, 100, 500, and 1700 m downstream of the source cell, along with bacterial concentration in bed sediment in the source cell were measured and reported. The river width ranged from 6 to 10 m and the water surface slope was reported to vary from 0.0007 to 0.008 m m$^{-1}$. The depth of sediment sampled for bacteria was 2 cm. The median grain size was 0.11 mm and 32 percent of the sediment particles were reported to be finer than 0.075 mm.

To understand the effect of sediment-water interaction on microbial concentration in the stream, two scenarios were analyzed. The first scenario without sediment-water interaction was simulated using the VART model with a bacterial growth/decay term while the second scenario with sediment-water interaction was modeled using the VARTBacT model given by Eqs. (5.6)–(5.9).

Values of model input parameters related to flow, river geometry, sediment and bacterial concentration were taken from Jamieson et al. (2005b). The average river width of 8 m was used throughout the study reach. The water surface slope during the base flow was set to 0.001 m m$^{-1}$. Manning’s coefficient $n = 0.045$. The hydrograph-based method (Ghimire and Deng 2011) was used to compute flow parameters such as $\tau$ (Figure 5-4), $h$ and $U$. Minimum, mean and maximum values of $\tau$ during the simulation period were 0.41, 1.49, and 3.05 N/m$^2$, respectively. Hourly flow data were used in the calculation. The concentration of bacteria at 10 m downstream from the source cell was used as an upstream boundary condition for the model.
Figure 5-4 Observed flow \((Q)\) and computed bed shear stress \((\tau)\) at 100 m downstream of source cell

The first scenario assumes no sediment resuspension/deposition in the selected reach. Therefore, the third and fourth terms on right-hand side of Eq. (5.6) which are relevant to sediment resuspension and deposition were dropped. The net decay rate in storage zones \(k_s\) is taken as \(-0.12 \text{ d}^{-1}\) as measured by Jamieson et al. (2005b) and the net decay rate in water column \(k\) was adopted as \(8.60 \text{ d}^{-1}\), an average value for three flow events estimated by Rehmann and Soupir (2009). The dispersion coefficient \(K_s\) was calculated based on the formula presented by Deng et al. (2001) for straight rivers:

\[
\frac{K_s}{hU_*} = \frac{0.01\psi}{8M_*} \left(\frac{B}{h}\right)^{5/3} \left(\frac{U}{U_*}\right)^2
\]

(5.12)

where, \(U_* = \text{bed shear velocity (m/s)}\) whose minimum, mean and maximum values were 0.02, 0.038, and 0.055 m/s, respectively; \(\psi = 4.0\) (Deng and Jung 2008) and \(M_*\) is given by
\[ M_* = 0.145 + \frac{1}{3520} \left( \frac{U}{U_*} \right) \left( \frac{B}{h} \right)^{1.38} \]  
(5.13)

Other parameters were either estimated or calibrated as recommended by (Deng and Jung 2009) for VART model. It was assumed that no lateral flow or hyporheic exchange occurred during the simulation period. The estimated parameters for all three reaches are as follows: \( K_s = 4.5 \text{ m}^2/\text{s} \), \( A_{\text{adv}}/A_{\text{av.}} = 0.1 \), \( D_s/A_{\text{av.}} = -7.5 \times 10^{-7} \text{ (1/s)} \), \( T_{\text{min}} = 10 \text{ hrs} \). The results are shown in Figure 5-5. The root mean squared error (RMSE) (Bard 1974) of the fit for reaches 1, 2 and 3 are 1.33, 0.86, and 0.78, respectively.

The second scenario with sediment resuspension and deposition was simulated with all terms in Eq. (5.6). The flow parameters and the decay rates were kept same as in the first scenario. The measured median particle size \( d_{50} \) was set to 0.1 mm (Jamieson et al. 2005b) and \( \tau_{\text{cr,e}} \) was set to 1.5 N/m^2 for sediment resuspension computations. Sediment was considered to be

![Figure 5-5 Simulated concentrations of E. coli in water column at (a) 100, (b) 500, and (c) 1700 m downstream from the source cell without considering sediment-water interactions](image)
cohesive with $\tau_{cr,d} = 0.08 \text{ N/m}^2$, which falls within the commonly observed range of 0.07 and 1.1 N/m$^2$ (Lick 2008). The empirical erosion rate constant $M$ was estimated as $0.12 \text{ g/m}^2\text{s}$. The
settling velocity \( w_s (= 0.0032 \text{ m/s}) \) was determined using Eq. (5.4) and porosity \( p (= 0.46) \) was obtained from a formula by Wu and Wang (2006).

Figure 5-6 Simulated concentrations of *E. coli* in water column at (a) 100, (b) 500, and (c) 1700 m downstream from the source cell considering sediment-water interactions.
The concentration of bacteria in bed sediment $C_b$ was estimated as 1.5 CFU/0.1g which was obtained through model calibrations. The attached fraction of the bacteria was kept as a constant equal to 0.2 which is considerably smaller than unity, assumed by Jamieson et al. (2005b). The RMSE of the fit for reaches 1, 2, and 3 are 1.15, 0.65, and 0.61, respectively.

A comparison of results between Figures 5-5 and 5-6 shows that the prediction of VARTBacT model with the consideration of sediment-water interaction did not produce substantially improved results. While simulated concentrations were slightly overpredicted in the first scenario without sediment interaction, the second scenario with sediment-water interaction slightly underestimated the concentration especially in the downstream reaches. Considering the uncertainties associated with the additional parameters such as settling velocity, sediment erosion and attached fraction, a slightly lower RMSE in the second scenario does not provide a compelling evidence of sediment-water interaction. Rehmann and Soupir (2009) reported that
their model always overpredicted the concentration without including sediment water interaction. When the sediment-water interaction was included, their model results were much better although slightly underpredicted. This slight discrepancy in the result might be due to the use of different approaches in computation of sediment resuspension or due to the consideration of full time series of flow using hydrograph-based method in this study. Rehmann and Soupir (2009) simulated three events with average flows estimated using a steady state model. A comparison of both results is therefore not straightforward, although both studies demonstrated that the impact of sediment-water column interaction on the model may be substantial. Note that Jamieson et al. (2005b) recovered no bacterium in any bed sediment samples collected downstream of the source cell. Moreover, the E. coli concentration did not increase in the downstream direction, leading them to believe that the bacteria, which were resuspended from the source cell during flood events, travelled through the study reach without deposition. It means that the first simulation scenario best describes the experiments by Jamieson et al. (2005b).

5.3.3. E. Coli Transport during High Flow Events with Contribution from Watersheds

The first two cases focus on effects of individual processes, such as transient storage (case 1) and sediment resuspension (case 2), on bacterial fate and transport under controlled experimental conditions. In addition to the individual processes simulated in previous two sections, bacterial fate and transport in streams under natural conditions like flood events are also highly affected by bacterial inputs from watersheds which are rarely taken into account in previous studies. This section is intended to model bacterial fate and transport in streams under natural conditions which may include effects of all individual processes.

E. coli concentration during several natural high flow events in Motueka River, New Zealand were monitored over a period of 13 months from June 2003 (Wilkinson 2008). The
Motueka River has a drainage area of 2180 km$^2$ and a median discharge of 47 m$^3$ at about 8 km upstream of the mouth of river at Woodmans Bend. The discharge and turbidity were also measured continuously at this location. The $E. \ coli$ concentration during storm events at Woodmans Bend were found to be an order of magnitude or more higher than during the base flow (McKergow and Davies-Colley 2010). As routine sampling efforts normally do not cover such events, the monitoring efforts provided excellent data for investigating the controlling factors and sources of $E. \ coli$ in a river under natural conditions. Since the storm events export a major part of the annual load of $E. \ coli$ up to 98% (Chu et al. 2011; McKergow and Davies-Colley 2010), timing and magnitude of $E. \ coli$ concentration is highly important.

In this case study, the VARTBacT model tested in the previous two cases was used to simulate the $E. \ coli$ dynamics in Woodmans Bend of Motueka River using monitored data for twelve storm events between 2003 and 2004. Observed $E. \ coli$ and flow data from Wilkinson et al. (2011), and suspended sediment concentration data from Wild et al. (2006) are used in VARTBacT simulations. The width of the river is about 70 m and the water surface slope during the base flow is about $1.0 \times 10^{-4}$ m m$^{-1}$. Manning’s roughness coefficient was estimated as 0.045 based on Chow (1959). Flow parameters and sediment transport were estimated using the hydrograph-based method. Hourly flow data were used as an input in Eq. (5.1).

As the streambed sediment is mostly sand mixed with gravel (Basher et al. 2011), a non-cohesive formula is more appropriate for sediment transport. Median grain diameter $d_{50}$ of 0.125 mm was used in Ackers and White (1973) formula for sediment transport in the hydrograph-based method. As the sediment size distribution is not available at the study site, a single median grain size was used. The simulated and observed suspended sediment concentrations during two periods are shown in Figure 5-7.
To understand the relative contribution of the streambed and watersheds to *E. coli* transport during storm events and to demonstrate the applicability of the VARTBacT model, a 5-km-long stream reach with its downstream boundary at Woodmans bend was selected. The reach was assumed to have a constant channel geometry and sediment properties. Since there are no measured *E. coli* concentration data available for the flood events upstream of Woodmans bend,
the concentration at the upstream boundary was estimated by assuming a direct correlation of *E. coli* concentration with the simulated depth averaged sediment concentration *S* through a parameter $\xi$. The parameter $\xi$, which represents the contribution of the watershed as well as channel source upstream of the reach, was determined through calibrations. The depth averaged net sediment flux $Fr$ was calculated using Eq. (5.2). Actual average sediment concentration $S$ was set to increase linearly from $0.7 \times S_e$ for the highest bed shear stress ($\tau_{max} = 12.0 \text{ N/m}^2$) to $1.3 \times S_e$ for lowest bed shear stress ($\tau_{min} = 0.5 \text{ N/m}^2$) to ensure net erosion during high flows and net deposition during low flows. As the study reach is not undergoing significant erosion or deposition over time, minimal net erosion was ensured in this way. The equilibrium transport capacity was estimated to occur at bed shear stress $\tau = 5.0 \text{ N/m}^2$. Net vertical displacement was computed using Eq. (5.3) whereas the settling velocity was computed using Eq. (5.4). The porosity was computed using formula by Wu and Wang (2006). The maximum displacement velocity for the events occurred during event no. 2 and was equal to $8 \times 10^{-6} \text{ m/s}$ which is in the lower range of $10^{-7}$ to $10^{-2} \text{ m/s}$ used by Rehmann and Soupir (2009) based on experimental reports by Roberts et al. (1998).

All the terms in equations (5.6)–(5.9) were included and solved. The VART parameters were determined following the same approach described as for the Swan Creek earlier: $K_s = 135 \text{ m}^2/\text{s}$, $A_{adv}/A_{av} = 0.1$, $D_s = -1.0 \times 10^{-5}$ and $T_{min} = 24 \text{ hrs}$. The same parameter values were used for all events. Model results were not very sensitive to these parameters during high flows. The inactivation coefficient of bacteria in water column $k$ and in storage zones $ks$ were set as $1.1 \text{ d}^{-1}$ for all events. Significant die-off of bacteria during storm events is unlikely due to increased turbidity, higher water depth and faster flows (Wilkinson 2008). The linear partition coefficient $K_d$ was set to 4 L/g, which is less than the value ($K_d = 10 \text{ L/g}$) used by Bai and Lung (2005). This
value of $K_d$ gives the fraction of bacteria $f_p = 0.45$ at sediment concentration of 200 g/m$^3$, which is within the range of 0.20 to 0.55 reported by Characklis et al. (2005) for surface waters and storm waters.

The parameter $\xi$ and the concentration of $E. coli$ in bed sediment $C_b$ were primarily determined through calibration (Table 5-3). The $E. coli$ concentration in the bed sediment was set to $5.0 \times 10^2$ CFU/0.1 g in all storm events except event No. 4, in which it was set to $5.0 \times 10^4$ CFU/0.1 g to get the best fit. These values are higher than the average but within the range observed in river sediments: $1-10^5$ CFU/g (Cho et al. 2010b), $10^3-9 \times 10^4$ CFU/g (Smith et al. 2008), $0-10^7$ CFU/100 mL (Tian et al. 2002), $8.3 \times 10^5-3.07 \times 10^5$ MPN/kg (Cho et al. 2010b). Gao et al. (2011) used $10^6$ CFU/g in their model. Storm event No. 4 had the estimated $C_b$ value that is higher than the other events by two orders of magnitude. The concentration in the water column during this event is significantly higher than other events of similar magnitude. McKergow and Davies-Colley (2010) attributed such a high concentration during storm event No. 4 to rainfall distribution and timing between events.

Table 5-3 Parameter $\xi$, and RMSE values for storm events

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Storm Events at Woodmans Bend</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
</tr>
<tr>
<td>$\xi$ (CFU/0.1g)</td>
<td>1.5</td>
</tr>
<tr>
<td>RMSE</td>
<td>0.55</td>
</tr>
</tbody>
</table>

A comparison between modeled and observed $E. coli$ concentration for flood events shows that the model generally captured observed concentration patterns well, as shown in Figure 5.8. The simulated peak concentration mostly occurred during the rising limb of the hydrograph, matching the trend of the observed data well. A slight under-estimation during low flows might
Figure 5-8 Observed and simulated *E. coli* concentrations in the Motueka River at Woodmans Bend during storm events 1 (a) to 12 (k) which occurred between June 2003 and June 2004.
(Figure 5-8 cont’d)

- **Event 4 (9/28/03 - 10/1/03)**
  - Graph showing E. coli (CFU/100 mL) over time (hour) with a notable peak.

- **Event 5 (10/03/03 - 10/06/03)**
  - Graph showing E. coli (CFU/100 mL) with a lesser peak compared to Event 4.

- **Event 6 (11/1/03 - 11/5/03)**
  - Graph showing E. coli (CFU/100 mL) with a significant drop post-peak.
Figure 5-8 cont’d

Event 7 (11/12/03 - 11/15/03)

Event 8 (2/27/04 - 3/2/04)

Event 9 (5/27/04 - 5/31/04)
be due to the higher settling velocity. This is usually the case when a single median particle size is used.

The best fit was achieved primarily with the two parameters $\zeta$ and $C_b$. Inactivation rate or other VART parameters were not very sensitive during these short high flow events although their roles can be important during moderate to low flows for an extended period. While resuspension of *E. coli*, represented by $C_b$, was important to simulating peak concentrations correctly, the parameter $\zeta$ has the greatest impact on the model output ($\zeta \times S = \text{assumed upper boundary concentration}$). Without the resuspension component, the simulated concentration gradient during rising or falling hydrographs would be low and underestimate the peak
concentration. Although it was not possible to quantify the contribution of channel sources, this simulation demonstrated that the remobilization of channel sources of *E. coli* can be an important part of the total *E. coli* load during peak flows. With measured concentrations at two locations or with better estimates of $C_b$, the contribution from the channel source can be determined using this method.

5.4. General Discussion

One of the distinctive features of the hydrograph-based modeling approach in comparison with other models for bacterial transport is that the new approach incorporates the transient storage effect and the effect of sediment resuspension and deposition into the VART model as external sources or sinks, respectively. In contrast, many other models (e.g., Gao et al. 2011; Rehmann and Soupir 2009; Steets and Holden 2003) consider the mass balance between water column and sediment bed, and neglect the transient storage effect. Therefore, the present model is suitable to use when  

i) the transient storage exchange is important for bacterial transport such as when the hyporheic exchange is the main cause for bacterial flux (e.g., Grant et al. 2011), and there is no significant sediment resuspension, and

ii) when the sediment resuspension is significant but the duration is relatively short such as during flood events. In the first case, the resuspension term in Eqs. (5.6) is irrelevant, whereas in the second case, the transient storage effect is often negligible and become irrelevant due to longer time scale and destruction of storage zones by high flow-induced sediment resuspension. However, if the resuspension and deposition occur for longer time or in succession, this model may not be efficient as it does not consider the change in concentration in bed sediment $C_b$ in response to resuspension/deposition to and from the water column directly. When it is applied for individual flow events where the settling does not significantly affect the concentration, this model should work well.
With the resuspension term in the VARTBacT model, $C_b$ has to be estimated together with the rate of resuspension $E$. This was done while modeling for Swan Creek and Motueka storm events in this study. The resuspension was first estimated and then $C_b$ was determined through calibrations. As uncertainties related to each term is usually high, one of the options is to use resuspension flux ($E \times C_b$) as a single parameter to determine through calibrations as proposed by Cho et al. (2010b). This approach is helpful when estimating $E$ is difficult due to lack of knowledge on sediment properties or sediment transport data. This is usually the case when the sediment is cohesive and no measured erosion rate is available. In view of the possibility of decreasing values of both $E$ and $C_b$ vertically in the streambed, the calibration appears to be the only viable option.

In addition to resuspension, two processes related to the sediment-water interaction are important: settling and attachment to particles. Proper understanding of uncertainties associated with these processes is important for bacterial modeling. For example, the particle size of bottom sediment vary over 2–3 orders of magnitude and the settling velocity varies over 4–6 orders of magnitude. Yet a single representative d50 size was adopted in this study for simplicity of computation. Similarly, the effect of flocculation on settling velocity for cohesive sediment was ignored, despite the fact that the concentration of bacteria is highest in flocs dominating in the suspended sediment (Droppo et al. 2009). Most of the existing models for cohesive sediment transport use Stokes law although the law is only applicable for a steady and laminar flow (Lick 2008) and gives higher settling velocity for cohesive sediments (Camenen and Larson 2009). Another uncertainty is due to assumed fraction of attached bacteria $f_p$ or partition coefficient $K_d$ in the suspended sediment. Attachment ratio greatly influences the settling rate of the bacteria from the water column. The attached fractions vary very widely from about 0.10 (Liu et al. 2006)
to more than 0.80 (Hipsey et al. 2006; Steets and Holden 2003) in natural waters. Characklis et al. (2005) found that 20 to 55% of *E. coli* was associated with settleable solids in surface waters and storm waters. Reported values of *f*<sub>p</sub> in rivers and estuaries are mostly in this range, e.g., 0.22 to 0.44 (Jamieson et al. 2005a), 0.30 (Dorner et al. 2006), 0.38 (Fries et al. 2006), and 0.50 (Cho et al. 2010b; Wu et al. 2009). In this study, *k*<sub>d</sub> was set to 4 L/g for Motueka river floods, which gives *f*<sub>p</sub> = 0.45 at a concentration of about 200 g/m³. In case of Swan Creek, the value of 1.0 was used in Swan Creek as assumed by Jamieson et al. (2005b) in their study. Unattached bacteria were assumed to have zero settling velocity following Hipsey et al. (2006).

Although less important for simulating short storm events, the bacterial decay/growth terms *k*, *k*<sub>s</sub> may be important for overall model results. The factors that influence the decay of *E. coli* in aqueous environment include exposure to sunlight, temperature, salinity, pH and nutrients. Excellent literature is available on the fate of bacteria in aquatic (Hipsey et al. 2008; John and Rose 2005) and sediment (Davies et al. 1995; Garzio-Hadzick et al. 2010) environments. In general the survival rate of *E. coli* bacteria in sediment is much longer than in surface waters. Jamieson et al. (2004) found out that *E. coli* may survive in streambed sediments for more than 6 weeks where they are protected against predation and sunlight exposure. Evidence of growth in the sediment has also been reported (Hipsey et al. 2008).

Finally, the performance of the model for high flow events is greatly enhanced by the use of hydrograph-based method (Ghimire and Deng 2011), which provides better estimates of bed shear stress and other flow variables comparing with the conventional steady uniform flow approach. This method takes into account the unsteadiness of flow by including the discharge gradient (∂Q/∂t) for correcting friction slope during unsteady flows. The *E. coli* concentration is found to be more closely correlated with turbidity than with the flow (Muirhead et al. 2004) and
often found to exhibit hysteretic relationships (Nagels et al. 2002) similar to those of suspended sediment concentration (Asselman 1999). It is obvious that the prediction of the model, particularly in timing and magnitude of the peak, is improved for flood events.

5.5. Conclusion

A new modeling approach to prediction of the fate and transport processes of bacteria in streams is presented. The model extends VART model by including resuspension and deposition processes during high flow events. It was developed using hydrograph-based method for flow, sediment transport and sediment-water interface displacement. Sediment fluxes were computed using sediment carrying capacity approach for non-cohesive sediment and critical bed shear stress approach for cohesive sediment. The model was applied to three different rivers with distinct characteristics, ranging from steady flow with significant transient storage exchange to highly unsteady storm flows where the sediment resuspension plays an important role for bacterial fate and transport in streams. The main findings from this study are as follows:

- The VART model is able to simulate bacteriophage P22 tracer breakthrough curves in a low flow river without significant sediment resuspension/deposition.
- The VARTBacT model is able to predict the field measurements relatively accurately using time series of hourly flow data generated from the hydrograph-based method. This model is appropriate for studying effect of sediment-water column interaction on the bacterial concentration during flood events in rivers.
- While transient storage and settling may be important during low and relatively steady flows, resuspension and deposition processes become dominant during high storm flows and the transient storage effect is negligible.
• The model can be applied effectively for simulating individual flood events. The
demonstration application in Motueka River of New Zealand shows that while the
resuspension from bed sediment can be a significant source of *E. coli* bacteria, watershed
inputs are by far the most important contributor to instream bacterial transport during flood
events. More efforts are needed in estimating watershed loading of bacteria.

5.6. Numerical Solution of VARTBacT Model

A split-operator method is employed to split the Eq. (5.6) into a pure advection and a
dispersion equation with the transient storage, erosion/deposition and decay terms:

\[
\frac{\partial C}{\partial t} + U \frac{\partial C}{\partial x} = 0, \quad t \in \left(t^n, t^{n+1/2}\right) \tag{A1}
\]

\[
\frac{\partial C}{\partial t} = K_s \frac{\partial^2 C}{\partial x^2} + \frac{A_v}{A} \frac{1}{T_v} (C_s - C) + \frac{E}{h} C_b - \frac{D f_p C}{h} \frac{1}{S} - kC, \quad t \in \left(t^{n+1/2}, t^{n+1}\right) \tag{A2}
\]

where \(n\) stands for time step. The Eq. (A1) is solved using semi-Lagrangian approach (Deng et
al. 2006), whereas Eq. (A2) in conjunction with Eq. (5.7)–(5.9) is solved using forward time
scheme and fully implicit F.3 central finite-difference scheme presented by Deng et al. (2004).

The discretized form of the Eq. (A2) may be written as

\[
\frac{C_i^{n+1} - C_i^{n+1/2}}{\Delta t/2} = \frac{K_s}{(\Delta x)^2} \left(C_{i+1}^{n+1} - 2C_i^{n+1} + C_{i-1}^{n+1}\right) + \frac{\varepsilon}{T_v} R \left(\frac{C_{sl}^{n+1} + C_{sl}^{n+1/2}}{2} - \frac{C_i^{n+1} + C_i^{n+1/2}}{2}\right) \tag{A3}
\]

\[
- \left(\frac{D f_p}{h S} + k\right) \left(\frac{C_i^{n+1} + C_i^{n+1/2}}{2}\right) + \frac{E}{h} C_b
\]

Where \(\varepsilon = 1/T_v, R = A_s/A\). The Eq. (5.7) is discretized as
Rearranging terms in Eq. (A4) yields

\[
\frac{C_{si}^{n+1} - C_{sl}^{n+1/2}}{\Delta t/2} = \varepsilon \left( \frac{C_{i}^{n+1} + C_{i}^{n+1/2}}{2} - \frac{C_{si}^{n+1} + C_{si}^{n+1/2}}{2} \right) - k_s \left( \frac{C_{si}^{n+1} + C_{si}^{n+1/2}}{2} \right) \quad (A4)
\]

Substituting Eq. (A5) into Eq. (A3) and rearranging terms to have all known terms on the right side gives

\[
-\alpha C_{i+1}^{n+1} + (1 + 2\alpha + \gamma + \beta)C_{i}^{n+1} - \alpha C_{i-1}^{n+1} = (1 - \gamma - \beta)C_{i}^{n+1/2} + 2\beta C_{si}^{n+1/2} + \eta \quad (A6)
\]

where the following definitions are used:

\[
\alpha = \frac{K_s \Delta t}{2(\Delta x)^2}, \quad \gamma = \left(\frac{D f_p}{h} + k\right)\frac{\Delta t}{4}, \quad \beta = \frac{(\varepsilon R \Delta t)(1 + k_s \Delta t/4)}{4(1 + \varepsilon \Delta t/4 + k_s \Delta t/4)}, \quad \eta = \frac{E \Delta t}{2h} C_b \quad (A7)
\]

The parameters \( \gamma, \beta \) and \( \eta \) are either known or calculable. The concentration \( C_{i}^{n+1/2} \) is computed from the solution of Eq. (A1) whereas the concentration \( C_{si}^{n+1/2} \) is assumed to be equal to \( C_{i}^{n} \).

The Eq. (A7) then may be grouped as follows

\[
\Omega C_{i+1}^{n+1} + PC_{i}^{n+1} + \Omega C_{i-1}^{n+1} = W^{n+1/2} \quad (A8)
\]

Where, \( \Omega = -\alpha, \ P = 1 + 2\alpha + \gamma + \beta, \) and

\[
W^{n+1/2} = (1 - \gamma - \beta)C_{i}^{n+1/2} + 2\beta C_{si}^{n+1/2} + \eta \quad (A9)
\]

The left-hand side of Eq. (A8) may be assembled into a tridiagonal matrix and solved to determine the concentration at time level \( n+1 \).

**5.7. References**


Krone, R. B. (1962). "Flume studies of the transport of sediment in estuarial shoaling processes." Hydraulic Engineering Laboratory and Sanitary Engineering Research Laboratory, University of California, Berkeley, CA.


CHAPTER 6. MAJOR FINDINGS AND DISCUSSIONS

6.1. Major Findings

This dissertation presents a practical approach, called hydrograph-based approach, to modeling fate and transport of fecal indicator bacteria in coastal lowland streams. The coastal streams are commonly characterized by fine-grained nutrient-rich sediment and high variability in flow ranging from low flow without sediment to flood events carrying high concentrations of sediment and associated bacteria. Dominant processes responsible for bacterial fate and transport vary with flow and sediment transport. The hydrograph-based approach is characterized by the important features: (1) Low data requirement: The basic data used in this approach are flow hydrographs which are the most commonly available flow data; (2) High efficiency: The new hydrograph-based approach is simple and efficient because the results from the hydrograph-based models are better or at least comparable with those from more complicated models; and (3) High effectiveness: The new hydrograph-based approach is effective in terms of its capability in predicting bacterial concentrations for a wide range of flow conditions from low flow without sediment to flood events carrying high concentrations of sediment, from watersheds and resuspension from stream bed, and associated fecal indicator bacteria. The new features of the hydrograph-based approach are obtained from the following major contributions of this study:

1. A flow-hydrograph based method for bed shear velocity was developed for mild-sloped rivers. Derived from St. Venant equations, this method uses the discharge gradient from flow hydrographs and flood wave celerity from the friction law to modify the friction slope during unsteady flow. Comparisons with other established methods using experiment data showed that this method is able to produce reasonable results. The accuracy was much better during rising limb of the hydrograph than during falling limb, when it was slightly underpredicted.
Similar results were also observed when this method was applied for river flood events and compared with the Hydrologic Engineering Centers River Analysis System (HEC-RAS) software.

2. A method for simulating sediment transport during flood events was developed. Using shear velocity and other flow parameters such as friction slope, flow velocity etc., derived from the hydrograph-based method, this method uses a selected empirical formula from the literature to obtain sediment concentration. Application of this method in two rivers covering several flood events showed that this method is comparable with more complicated numerical models such as HEC-RAS in terms of overall accuracy and gives relatively better results during rising as well as falling phases of large flood events. This method was able to reproduce clockwise hysteresis of sediment concentration frequently observed in rivers.

3. Investigations into solute transport process in medium and large rivers using tracer test data showed that the VART model is able to reproduce any type of solute breakthrough curves commonly observed. Solute residence time distributions (RTDs) were found to be channel-size dependent. Large rivers, dominated by instream advection and dispersion processes, tend to exhibit lognormal distributions. Small streams may display various types of the distributions, as they are affected more significantly by hyporheic exchange. The influence of hyporheic exchange on RTDs was found to increase with decreasing channel size. Moderate-sized rivers exhibited both lognormal and power-law RTDs. The effect of water and solute losses/gains on RTDs was found to be important for simulation results. Finally, the ratio of the time to peak to the minimum mean residence time in the VART model was found to be approximately equal to the recovery ratio of tracer. This relation provided an estimate of minimum mean residence time, thus greatly simplifying the application of the model.
4. VART model was extended to the simulation of bacterial transport and fate by adding resuspension/deposition processes of sediment and associated bacteria and adopting hydrograph-based method for flow and sediment transport. The extended VART model for bacteria transport (VARTBacT) uses sediment carrying capacity method for non-cohesive sediment and critical bed shear stress approach for cohesive sediment for computing sediment fluxes. The performance of the VARTBacT model was tested in three different rivers with distinct characteristics, ranging from steady flows with significant transient storage exchange to highly unsteady storm events where the sediment resuspension played an important role in bacterial fate and transport. The model without sediment terms was able to simulate bacteriophage P22 tracer breakthrough curves, whereas in two other case studies with sediment resuspension terms included, the model predicted the field measurements reasonably well. The application of the VARTBacT model for several storm events in Motueka River in New Zealand indicated that the resuspension from bed sediment can be a significant source of *E. coli* bacteria during flood events.

6.2. Discussions

The model (VARTBacT) includes all key processes responsible for bacterial transport and fate: advection, dispersion, growth/decay, resuspension/deposition, and transient storage exchange. Among these processes, resuspension and transient storage exchange are unlikely to play significant roles simultaneously. In fact, the transient storage effect should be negligible during high flow events, when resuspension is dominant. The opposite may be true during low flows, especially in sandy or gravel bed rivers. With selection of appropriate parameters, the VARTBacT model may be applied to both situations. Case studies for bacterial modeling in this study exemplify the applicability of the model to different flow and sediment conditions.
Another, probably the most important, feature of the hydrograph-based approach to bacterial modeling is that it requires far less input data as compared with other models for unsteady flows. In terms of flow data, the input data necessary are flow hydrograph, channel width and bed slope; all other parameters are determined using the method developed in this study. Modeling sediment transport based on a single median particle size for non-cohesive sediment also is straightforward as outlined in this study. The major assumption here is that the shear stress is the primary driving force for sediment transport, and that there is no limitation to the supply of sediment. With cohesive sediment, however, it is more difficult. The critical bed shear stresses for erosion and deposition are very site specific and must be known for reliable calculations.

In contrast to other models that consider sediment-water column interaction, the resuspension and deposition terms are an external source or sink to bacteria in this model. Due to this feature, the present model is suitable to use: i) when the transient storage exchange is important for bacterial transport and there is no significant sediment resuspension and, ii) when the sediment resuspension is significant but the duration is relatively short such as during flood events. However, if the resuspension and deposition occur for a longer time or in succession, this model may not be efficient as it does not consider the change in concentration in bed sediment $C_b$ in response to resuspension/deposition to and from the water column directly. The hydrograph-based approach is best to apply in mild-sloped rivers with regular geometry and may not be suitable for highly irregular rivers with flow over flood plains.

This study has provided a tool for researchers or practitioners who deal with sediment and bacterial transport problems during natural floods but are inclined to use steady flow formula either for simplicity or due to lack of data. With the approach developed in this research, there
should be no need to go for steady formula for those reasons or compromise seriously with the accuracy due to unsteadiness of the flow during natural flood events in lowland rivers.

Finally, this study focused on in-channel processes concerning fate and transport modeling of bacteria and did not include watershed modeling. Quantifying channel contribution for bacteria load during storm events is important to understanding transport processes, but requires additional data not available for this study. Therefore future studies are recommended to determine the contribution of bed sediments to the total bacteria load by either using output from a watershed model or collecting bacterial concentration data at upstream and downstream points of the considered reach. As some of the parameters involved in the model are difficult to estimate, sensitivity and uncertainty analysis will also be helpful to determining the importance of model parameters.
APPENDIX: LETTERS OF PERMISSION

Title: Event flow hydrograph-based method for shear velocity estimation
Author: Bhuban Ghimire, Zhi-Qiang Deng
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VITA

Bhuban Ghimire was born in 1966 in Gulmi, Nepal. He received his Bachelor of Science in Industrial and Civil Engineering degree from Odessa State Academy of Civil Engineering and Architecture (then Odessa Civil Engineering Institute), Odessa, Ukraine, in 1992. He worked as a civil engineer for nine years before he joined UNESCO-IHE for Water Education, Delft, Netherlands, where he was awarded the Master of Science in Hydraulic Engineering degree in 2003. Upon graduation, he continued working as a civil engineer for the Department of Water Induced Disaster Prevention, Nepal. He joined water resources program of the Department of Civil and Environmental Engineering, Louisiana State University in August 2006.